PRELIMINARY GEOTECHNICAL DESIGN REPORT
U.S. ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

by

Haley & Aldrich, Inc.
Portland, Maine

for

Maine Department of Transportation
Augusta, Maine

File No. 38751-030
17 September 2013

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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Preliminary Geotechnical Design Report
U.S. Route 1 Viaduct Bridge Rehabilitation
MaineDOT WIN 19273.00
Bath, Maine

Ladies and Gentlemen:

This preliminary geotechnical design report (PGDR) presents the results of limited subsurface investigation and geotechnical laboratory testing programs, and also provides preliminary geotechnical design recommendations and geotechnical factors that could affect U.S Route 1 Viaduct Bridge (viaduct bridge) replacement/rehabilitation alternatives currently being considered. This PGDR has been prepared in support of Figg Bridge Engineers, Inc. (Figg) Feasibility Study submission to the Maine Department of Transportation (MaineDOT).

This work has been completed based on our mutually agreed upon work scope and in accordance with the provisions of our GCA Agreement with MaineDOT, No. CT20110614000000006492, and with our project-specific assignment letter dated 15 March 2013.

ELEVATION DATUM

Elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88). Please note that the historic viaduct bridge information presented and referenced herein references an assumed vertical datum. The relationship between the assumed vertical datum and the NAVD 88 datum is discussed in subsequent sections of this report.

PROJECT UNDERSTANDING

Project Location and Existing Viaduct Bridge Structure

The existing viaduct bridge carries U.S. Route 1 traffic and serves as the west approach to the Sagadahoc Bridge over the Kennebec River in Bath, Maine (see Figure 1, Project Locus).

Based on our review of historic bridge plans provided by MaineDOT, it is our understanding that the existing viaduct bridge was originally constructed in 1957 and consists of an approximate 1,300-ft long, 34-ft wide (two travel lanes, one in each direction), 20-span structure supported on two abutments and

19 piers. The foundation support type for the abutments and piers is variable and consists of one of the following:

- Cast-in-place concrete spread footings bearing on bedrock (i.e., Abutment 1, Piers 1, 2, 4, 8, 9 and 13)
- Cast-in-place concrete spread footings constructed on seal concrete, bearing on bedrock (i.e., Piers 3 and 14)
- HP10x42 steel H-piles driven to glacial till or bedrock (i.e., Piers 5-7, 10-12, 15-19 and Abutment 2)

A summary of existing viaduct bridge substructure types is provided for reference in Tables I and II, respectively.

Based on our review of historic bridge plans it is unclear what allowable bearing pressure was used to design the footings that support the existing abutments/piers. In addition, HP10x42 steel H-piles supporting the abutments/piers listed in Table II were designed based on "allowable loads" equal to 67.6 kips (Abutment 2) and 92.6 kips, respectively. Estimated pile lengths range from approximately 15 to 55 ft. Select piles supporting the viaduct bridge piers and Abutment 2 are battered at 4V:1H and 5V:1H, respectively.

Viaduct Bridge Replacement/Reconstruction Alternatives and Work Scope

Based on discussions with you, it is our understanding that Figg has been retained by MaineDOT to evaluate bridge rehabilitation and replacement alternatives and to prepare conceptual drawings along with potential construction schedule options and estimated construction costs. Based on our discussions with MaineDOT and Figg, it is our understanding that the Feasibility Study will evaluate the following:

- Need to replace superstructure concrete and steel beams.
- Concept for bridge deck replacement, including cast-in-place concrete and precast panels.
- Need for pier repair and/or replacement and reuse potential of existing foundations.
- Potential for shortening the bridge by removing existing spans and replacing them with earthfill. Specifically, evaluating the potential for eliminating the superstructure between Abutment 1 and Pier 6, Pier 7 and Pier 14 and Pier 18 and Abutment 2 and replace with a vertical sided embankment to reduce access beneath the bridge, minimize viaduct bridge replacement costs or minimize future viaduct bridge maintenance costs.

Depending on the alternative, we understand that staged construction and maintenance of U.S. Route 1 traffic will be necessary during construction.

SUBSURFACE EXPLORATION PROGRAMS

Historic Explorations

The plan locations of the historic explorations (rod soundings and test borings) as well as logs providing information on subsurface conditions encountered in rod soundings and test borings are provided on historic drawings included in Appendices D and A, respectively. Additional details are provided in the following sections of this report.

A. Rod Soundings

A preliminary subsurface investigation, consisting of rod soundings, was completed in late 1956 and early 1957. Rod soundings were completed at the following existing substructure locations: Pier Nos. 1, 3, 5, 9, 11, 16, 17 and 19. No specific information was provided on the historic viaduct bridge drawings describing the means and methods used to advance the rod soundings. However, based on our past project experience, it is likely that rod soundings were advanced by driving steel rods into the ground by dropping a standard-size weight from a known height. The number of blows required to advance the steel rods in 1-ft increments is often recorded until a refusal surface is encountered. All rod soundings were driven to refusal as shown on the historic bridge plans however, no information was provided that defines what refusal criterion was used to terminate the explorations. Rod soundings were advanced from depths ranging from approximately 2.5 to 40.0 ft below historic ground surface (BGS), prior to existing viaduct bridge construction.

B. Test Borings

Upon completion of the preliminary subsurface investigation (i.e., the rod soundings), a supplemental investigation was conducted, which consisted of a minimum of 23 test borings, designated 1 through 23. Please note that the historic viaduct bridge plans indicate that more than 23 test borings were drilled however, logs were only provided for the first 23 test borings. The test borings were drilled to confirm findings from the preliminary subsurface investigation and to collect samples of compressible soils. Test borings were drilled at the following substructure locations: Abutment Nos. 1 and 2 and Pier Nos. 2, 4, 6-8, 10, 12-15 and 18.

The test borings were drilled to depths ranging from approximately 4 to 60 ft BGS using steel casing of unknown diameter. In-situ vane shear testing was conducted within and relatively undisturbed Shelby tube samples were obtained the marine silt/clay deposit. In addition, each test boring, with the exception of boring nos. 3 and 21, 22 and 23, was advanced up to approximately 9 ft into bedrock.

Recent Explorations

Haley & Aldrich conducted a limited subsurface investigation at the site to provide general subsurface information at specific substructure locations that were identified as critical locations relative the viaduct bridge replacement/reconstruction alternatives being evaluated by Figg. Three test borings, designated BB-BROV-101 through BB-BROV-103, were drilled at the following locations:

- BB-BROV-101: immediately northwest of Pier 6, drilled to investigate the potential for shortening the viaduct bridge by removing the superstructure between Abutment 1 and Pier 6 and replacing it with normal-weight earthfill retained by vertical-sided retaining walls.

- BB-BROV-102: immediately northeast of Pier 11, drilled to investigate the potential for shortening the viaduct bridge by removing the superstructure between Pier 7 and Pier 14 and replacing it with normal-weight earthfill retained by vertical-sided retaining walls.

- BB-BROV-103: midway between Pier 18 and Pier 19, drilled to investigate the potential for shortening the viaduct bridge by removing the superstructure between Pier 18 and Abutment 2 and replacing it with normal-weight earthfill retained by vertical-sided retaining walls.

The as-drilled plan location and ground surface elevation of each test boring was determined upon completion of drilling by MaineDOT using GPS and/or optical survey equipment. The as-drilled test boring locations are shown on Figure 2.

Test borings were drilled by Maine Test Borings of Hermon, Maine and were monitored by a Haley & Aldrich geologist on 26 and 27 March 2013 using a truck-mounted Mobile Drill B-53 drill rig. Test borings were drilled to depths ranging from approximately 17 to 54 ft BGS using 3.0-in. (NW-size) and 4.0-in. (HW-size) inside diameter (ID) steel casing using cased/washed drilling methods. Soil samples were collected continuously and/or at standard 5-ft increments by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., using a standard rope and cathead and safety hammer.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval (blows per foot, bpf). The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.60 (i.e., 60 percent theoretical hammer efficiency).

A total of three, relatively undisturbed samples of cohesive soil (marine silt/clay) were obtained in test boring BB-BROV-103. The samples were collected at various depths/elevations and were obtained by advancing a 3-in. outside diameter (OD) thin-wall Shelby Tube into the soil using a piston sampler. Please note that relatively undisturbed samples of marine silt/clay could not be obtained from test borings BB-BROV-101 and BB-BROV-102 due to the nature and consistency of the marine silt/clay encountered.

In-situ vane shear tests were conducted in test borings BB-BROV-101 and BB-BROV-103. Please note that in-situ vane shear testing could not be conducted in test boring BB-BROV-102, as judged by our field geologist, based on the nature and consistency of the marine silt/clay encountered. In-situ vane shear tests were conducted with either a 55 mm by 110 mm or 65 mm by 130 mm Geonor rectangular vane (per MaineDOT requirements) attached to a 2-ft long, 12-mm diameter rod extension, attached to a string of 5/8-in. outside diameter (OD) hollow chrome-moly rods. At each in-situ vane shear test location, the vane was pushed (by hand) until the bottom of the vane was approximately 1 to 2 ft below the bottom of the borehole. The vane was then rotated at a rate of about 90 degrees per minute using a

calibrated torque wrench. Results of the vane shear testing, including raw torque values and calculated shear strengths, are provided on the test boring logs in Appendix A and are shown graphically in Figure 3.

Each of the three test borings was advanced approximately 5 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

EXISTING ROCK SLOPE MAPPING

Haley & Aldrich collected rock mass data and photographed the exposed rock slope on the north side of the existing Route 1 Viaduct Bridge between Abutment 1 and Pier 6 on 4 April 2013. While onsite, data on structural geologic properties (e.g., discontinuity dip and dip direction, infilling, visible seepage, persistence, aperture) and general rock mass properties (e.g. weathering/alteration, intact rock compressive strength) were collected. As an integral part of our field evaluations, Haley & Aldrich utilized industry-accepted technical guidance criteria produced by the International Society for Rock Mechanics (ISRM). The rock mass data that was collected was used to support our preliminary technical evaluations and develop the subsequent conclusions and preliminary design recommendations included herein. The rock mass data that was collected is summarized in subsequent sections of this report and is also provided for reference in Appendix B.

GENERALIZED SUBSURFACE CONDITIONS

Subsurface soil, bedrock and groundwater conditions encountered in recent test borings are described in the following sections of this report.

Soil and Bedrock Conditions

A. Bituminous Concrete/Man-Placed Fill

A surficial layer of bituminous concrete was encountered at the ground surface in each test boring. The thickness of the bituminous concrete ranged from 0.1 to 1.0 ft.

A 2 to 3-ft thick (approximate) layer of man-placed fill was encountered in each test boring directly beneath the surficial layer of bituminous concrete. In general, the man-placed fill soil consisted of the medium to fine SAND (SW) with varying amounts of silt and coarse sand and gravel. Please note that historic boring no. 7 (drilled at the center of pier 12) encountered approximately 8 ft of coal and ash fill at the ground surface.

The fill was medium dense to dense with SPT N60 values ranging from 13 to 50 blows per foot (bpf).

B. Organic Deposit

An approximate 5.5-ft thick layer of organic soil was encountered beneath the man-placed fill in test boring BB-BROV-103. The soil consisted of SILT (ML) with varying amounts of fine sand and

woody/fibrous organic matter and was soft to medium stiff with SPT N60-values ranging from 2 to 5 bpf.

C. Marine Silt/Clay Deposit

Marine silt/clay was encountered directly beneath the man-placed fill or organic soil in each test boring. The deposit ranged in thickness from approximately 5 ft (BB-BROV-101) to 38 ft (BB-BROV-103) and generally increased from west to east, towards the Kennebec River.

In test borings BB-BROV-101 and BB-BROV-102 (thickness ranging from 5 to 7.5 ft), the deposit consisted of either mottled SILT (ML) with varying amounts of clay and fine sand or silty CLAY (CL) with fine sand. The soil was medium stiff to hard with SPT N60 values ranging from 6 to 44 bpf.

In test boring BB-BROV-103 (approximate 38-ft thickness) the upper 8.5 ft consisted of stiff to very stiff mottled clayey SILT (ML) to silty CLAY (CL). The remaining portion of the deposit consisted of soft to medium stiff silty (CL). In-situ vane shear test results within the lower portion of the deposit ranged from approximately 500 to 850 pounds per square foot (psf) and typically decreased with depth to approximately El. -15, as shown on the test boring logs and in Figure 3. Below El. -15 the undrained shear strength is generally uniform with depth and equal to approximately 500 psf.

D. Glacial Till

A limited thickness of glacial till was encountered in test borings BB-BROV-101 and BB-BROV-103 directly beneath the marine silt/clay layer and overlying bedrock. The deposit ranged in thickness from approximately 2 to 3 ft and consisted of clayey, fine to medium SAND (SC/SM) with little coarse sand and gravel.

Due to the limited thickness of glacial till encountered and pre-determined sampling intervals, limited SPT data was obtained. Therefore, an accurate assessment of the density/consistency of the soil layer could not be determined.

E. Bedrock

As described above, bedrock was encountered and sampled in each test boring. The top of bedrock surface varied from approximately 10 ft (BB-BROV-102) to 49 ft (BB-BROV-103) BGS, corresponding to El. 7 to El. -41. As shown on the historic viaduct bridge drawings, the top of bedrock surface is variable and could be considered erratic along the length of the viaduct bridge. The type of bedrock encountered was variable and generally consisted of the following:

- Moderately hard, fresh to slightly weathered SCHIST of the Cape Elizabeth Formation. Primary joints were oriented at low to moderate angles, very close to closely spaced and were generally tight to open. Joint surfaces were occasionally moderately weathered and silt coated.

- Moderately hard, slightly weathered GNEISS of the Cape Elizabeth Formation. Primary joints were oriented at low to moderate angles, very close to moderately closely spaced and were generally tight to open. Joint surfaces were occasionally silt coated.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of recovered bedrock. RQD values for bedrock encountered at the site ranged from 52 to 85 percent.

Disclosed joint sets observed along the rock slope along the northerly side of the viaduct bridge and between Abutment 1 and Pier 6 generally trended in a northwest to northeast (primarily northeast) orientation with the strike varying from approximately N75°W to N87°E. The dip of the rock mass varied between 22° and 90° (average of 69°) to the northeast, southeast, southwest and northwest (dip direction varied between 27° and 348°). Water was visible seeping out of some of the exposed joints.

Groundwater Conditions

Water levels were measured in the recently completed boreholes at depths ranging from approximately 12 to 14 ft BGS, prior to backfilling, as indicated on the test boring logs. Please note that observation wells were not installed in any of the recently completed boreholes. The water depths measured in the test borings may not be representative of actual groundwater levels and could have been influenced by drilling means/methods. In addition, given the proximity of test boring BB-BROV-103 to the Kennebec River, we anticipate that the water level at this location may be tidally influenced. In general, groundwater levels can be expected to fluctuate, subject to seasonal variation, local soil conditions, tide, topography and precipitation. Water levels encountered during construction may differ from those summarized above.

LABORATORY TESTING PROGRAMS

Historic Laboratory Testing

A laboratory testing program was undertaken on soil samples collected from the historic test borings drilled to support the original design and construction of the existing viaduct bridge. The available laboratory test results indicate that the primary purpose of the previous testing was to assist in soil classification/identification and to determine engineering properties of representative cohesive (marine silt/clay) soil samples. In general, laboratory testing was performed on disturbed and undisturbed marine silt/clay soil samples collected during drilling. It is our understanding that all laboratory soil testing was performed by either the Maine State Highway Commission (MaineDOT) or the University of Maine Soils Engineering Lab in Orono, Maine.

The testing of samples of marine silt/clay included the following: natural water content, Atterberg Limits, vane shear strength and incremental consolidation tests (used to determine compressibility and stress history characteristics).

Historic index testing and incremental consolidation test reports are provided on the test boring reports provided in Appendix A and in Appendix C, respectively.

Recent Laboratory Testing

A preliminary phase laboratory testing program was undertaken to assist in soil classification/identification and to determine engineering properties of representative cohesive (marine silt/clay) soil samples and bedrock samples collected during the field investigation. In general, laboratory testing was performed on disturbed and undisturbed marine silt/clay soil samples collected during SPT and Shelby Tube sampling, respectively as well as on intact rock samples collected during rock core sampling. All laboratory soil and rock testing was performed by GeoTesting Express of Acton, Massachusetts in accordance with applicable American Society for Testing Material (ASTM) testing procedures. A summary of laboratory testing conducted for the different soil and rock types encountered in the test borings is provided below.

The testing program on samples of marine silt/clay included three natural water content tests, three Atterberg Limits tests and three incremental consolidation tests (used to determine compressibility and stress history characteristics). A summary of laboratory test results completed on collected samples of marine silt/clay is provided below.

- Classification: CL
- Natural Water Content: 40% to 48%
- Atterberg Limits:
  - Liquid Limit (LL): 37% to 41%
  - Plastic Limit (PL): 19%
  - Plasticity Index (PI): 18% to 22%
- Total Unit Weight: 110 pounds per cubic foot (pcf) to 113 pcf

The laboratory testing program also included two unconfined compressive strength tests on samples of bedrock core. The results of these laboratory tests are summarized below.

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Peak Compressive Stress (psi)	Bulk Density (pcf)
BB-BROV-101	R1	14.6	15.0	2,321	174
BB-BROV-102	R1	14.0	14.4	1,522	173

Results of individual laboratory tests on soil and rock are provided for reference in Appendix C.

COMPARISON OF HISTORIC AND RECENT TEST BORINGS

As discussed in previous sections of this report, the viaduct bridge elevation information shown on the historic drawings references an assumed vertical datum and the relationship between the assumed datum and the NAVD 88 datum (current project vertical datum) is not known. Given the amount of in-situ and laboratory testing that was conducted within and from samples collected from the historic test borings, determining the relationship between the two elevation datums was considered critical and

would enable the use of laboratory test results to supplement recent laboratory findings in the preliminary geotechnical engineering evaluations discussed herein.

As a result, we were able to conduct a side-by-side comparison between test boring BB-BROV-103 and other historic test borings drilled in the immediate vicinity of Piers 18 and 19. This area was selected due to the number of different soil units present, the thickness of overburden soil encountered (greater than at test borings BB-BROV-101 and BB-BROV-102) and the number of test borings drilled in the area. Considering these factors increased the likelihood that the difference between the elevation datums could be determined by looking at individual soil unit thicknesses and material type and consistency assuming nothing has changed from the time the historic test borings were drilled to the present. A comparison of soil stratigraphy between recent test boring BB-BROV-103 and historic test borings 17 and 21 (historic test borings nearest to BB-BROV-103) as interpreted from the test boring logs is provided below.

Test Boring No.[1]	Ground Surface Elevation[2]	Approximate Strata Thickness[3] (ft)					Depth to Top of Bedrock BGS (ft)
		Man-Placed Fill	Organic Deposit	Marine Clay "Crust"	Non-dessicated Marine Clay	Marine Sand/ Glacial Till	
BB-BROV-103	7.8	3.0	5.5	8.5	29.8	2.2	49.0
Boring No. 17	56.2	3.0	5.5	10.5	31.0	10.5	57.5
Boring No. 21	55.5	4.0	5.0	10.0	30.0	11.0	60.0

[1] – See Figure 2 for plan location of test borings.
[2] – Ground surface elevation at test boring BB-BROV-103 references NAVD 88 vertical datum; ground surface elevation at test borings 17 and 21 reference an assumed vertical datum.
[3] – Strata thicknesses are approximate and are based on Haley & Aldrich interpretation of historic test boring logs.

As shown above, the soil unit thicknesses interpreted from the historic test boring logs closely match those encountered in recent test boring BB-BROV-103 with the exception of the thickness of the granular soils encountered beneath the marine clay and the depth to top of bedrock BGS. Based on the information summarized above, Haley & Aldrich judged that the subsurface conditions encountered in historic test boring 17 more closely matched the subsurface conditions encountered in recent test boring BB-BROV-103 and as a result, the ground surface elevation at each test boring location were approximately equal. Therefore, the approximate relationship between the historic assumed vertical datum and the NAVD 88 datum is as follows:

Assumed Historic Vertical Datum – 48 ft = Approximate NAVD 88 Vertical Datum

Using the relationship shown above, the elevation of historic in-situ and laboratory vane shear strength and incremental consolidation test data was converted to the NAVD 88 vertical datum and compared to recent data as shown in Figure 3. Given that the observed trends in undrained shear strength and

overconsolidation with depth were similar between historic and recent data, Haley & Aldrich judged that the relationship between the two elevation datums was appropriate and reliable for the intended use.

STRENGTH AND COMPRESSIBILITY CHARACTERISTICS OF MARINE SILT/CLAY

Please note that the stress history and compressibility characteristics of marine silt/clay soil discussed in this section of the report are relative to test boring BB-BROV-103 and the existing Abutment 2 area only.

The stress-strain or compressibility characteristics of clay deposits are highly dependent upon their stress history. Overconsolidation is a condition that results from the clay deposit having been exposed, at some time in the geologic past, to stresses greater than the present in-place stresses. If the clay deposit is stressed within the limits of the maximum previous stress (i.e., maximum past pressure), the magnitude of consolidation settlement will be a function of the recompression ratio (RR) of the clay. If the applied stress exceeds the maximum previous stress, the magnitude of consolidation settlement will be a function of the virgin compression ratio (CR). Measured values of CR are typically 10 to 25 times greater than RR, and consolidation settlement is directly correlated with the value of CR or RR. Therefore, the estimated consolidation settlement for normally consolidated clay would be 10 to 25 times greater than that of overconsolidated clay for the same stress increase. A summary of the stress history and compressibility characteristics determined from incremental consolidation tests is provided below and is shown graphically in Figure 3.

Test Boring No.[1]	Sample No.	Sample Elevation (ft, NAVD88)	Stress History			Compressibility[2]	
			Existing Vertical Effective Stress (psf)	Maximum Previous Stress[2] (psf)	OCR[2]	CR	RR
Boring No. 19	4U	-20.7	2,040	2,600	1.3	0.224	0.025
Boring No. 21	5U	-18.9	1,680	1,680	1.0	0.277	0.030
	6U	-23.9	1,900	1,900	1.0	0.189	0.042
BB-BROV-103	1U	-14.2	1,970	3,672	1.9	0.234	0.025
	2U	-23.2	2,396	2,525	1.1	0.274	0.023
	3U	-34.2	2,916	2,984	1.0	0.246	0.027

[1] – See Figure 2 for plan location of test borings.
[2] – Values shown are considered approximate and are based on interpretation of laboratory test results.

The preliminary design overconsolidation profile is shown in comparison to the laboratory consolidation data and the existing effective overburden pressure in Figure 3. The data indicates that the marine silt/clay in the vicinity of Abutment 2 is heavily overconsolidated (>1,500 psf) above El. -15 and becomes moderately overconsolidated between El. -15 and El. -20 (approximate). The marine silt/clay deposit is judged to be normally consolidated below approximately El. -20. The information shown in Figure 3 was used as the basis for the settlement analysis and results that are presented in subsequent sections of this report.

The undrained shear strength profile of the marine clay stratum was determined using in-situ vane shear tests conducted during drilling of the historic and recent test borings., The design undrained shear strength profile used in our global stability and settlement evaluations in the vicinity of Abutment 2 is shown in Figure 3. This information was used as the basis for the slope stability evaluations and results that are presented in subsequent sections of this report.

PRELIMINARY GEOTECHNICAL EVALUATIONS

Preliminary geotechnical design recommendations for the subject project, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Sixth Edition, 2012 with Errata dated June 2012.
- MaineDOT Bridge Design Guide (BDG), August 2003 with Interim Revisions through August 2008.

Furthermore, the preliminary design recommendations are based, in part, on the subsurface conditions encountered in the recently completed test borings. It should be noted that only three test borings were drilled at select existing viaduct bridge substructure locations. Variations in subsurface conditions are likely to exist transverse to the existing viaduct bridge structure. Our evaluations are based on the assumption that subsurface conditions are somewhat uniform in these areas and similar to those encountered in the test borings.

Existing Viaduct Bridge Foundations and Reuse Potential

The viaduct bridge replacement/reconstruction alternatives presented in previous sections of the report assume that the existing substructures and foundation soils/rock can accommodate increases in loading, which could vary depending on the replacement/rehabilitation alternative selected, while still meeting minimum acceptable design requirements. The following sections of this report summarize technical evaluations completed to assess the adequacy of the existing viaduct bridge foundations.

A. Preliminary Substructure Loading

Preliminary factored axial compressive loads (loads) for each existing viaduct bridge pier were developed by Figg and provided to Haley & Aldrich on 12 April 2013. Please note that loads were not provided for Abutment 1 or Abutment 2. The loads were developed for use in evaluating reuse potential of existing spread footing and steel H-pile abutment/pier foundations. Based on the information provided by Figg we understand that the loads were developed based on the following:

- Concrete segmental superstructure (9.33 kips/linear foot)
- Two lanes of vehicular traffic with HL-93 truck loading at each pier
- Live load (LL) factor = 1.75; dead load (DL) factor = 1.25

In general, the loads ranged from approximately 900 to 1,100 kips. We understand that the loads provided by Figg are considered conservative, primarily based on the assumption of a replacement concrete segmental superstructure (versus rehabilitation of the existing superstructure).

It should be noted that tensile loads, lateral loads and moments were not developed by Figg and therefore were not considered as part of the existing viaduct bridge foundation evaluations summarized in the subsequent sections of this report.

B. Spread Footings

As discussed in previous sections of the report and summarized in Table I, select viaduct bridge abutments and piers are supported by cast-in-place concrete spread footings bearing directly on bedrock or on seal concrete, bearing on bedrock. Based on our review of historic bridge plans it is unclear what allowable bearing pressure (i.e., factored bearing resistance) was used to design the footings that support substructures listed in Table I.

Based on the bedrock conditions encountered in the recent test borings and observed along the rock slope north of existing viaduct bridge and between Abutment 1 and Pier 6, we estimate that the adequacy of the existing spread footings can be evaluated based on a nominal bearing resistance equal to 44 kips per square foot (ksf). In addition, the footings should be evaluated at the service limit state based on a presumptive bearing resistance equal to 60 ksf (AASHTO LRFD Table C10.6.2.6.1-1). Footings should also be evaluated at the strength limit state based on a factored bearing resistance equal to 20 ksf (ϕ=0.45). Please note that resistance factor for extreme event limit state bearing resistance is equal to 1.0. Therefore, the factored bearing resistance at this limits state is equal to the recommended nominal bearing resistance (44 ksf).

Please note that the methodology outlined in AASHTO LRFD for determining bearing resistance of footings on bedrock is partially dependent on interpretive grading assigned to the recovered bedrock core in accordance with the Rock Mass Rating (RMR) System. The collected rock mass data (discussed in previous sections of this report) was used to adjust the RMR to account for strike and dip orientations relative to the existing spread footing foundations.

Based on the preliminary substructure loading information provided by Figg and the existing spread footing dimensions shown on the historic bridge plans and summarized on Table I, contact pressures were calculated and compared to the factored bearing resistance (20 ksf) at each pier location (please recall that loads were not provided for Abutment 1). The resulting contact pressures were less than the factored bearing resistance and ranged from approximately 7 to 8 ksf. As a result, from a geotechnical standpoint, it is our opinion that the existing spread footing foundations bearing on bedrock are capable of supporting the rehabilitated superstructure and meet minimum design requirements.

Please note that tensile loads, lateral loads and moments were not considered as part of the existing viaduct bridge spread footing evaluations. Therefore, we recommend that the adequacy of the existing spread footing foundations be re-evaluated once a preferred replacement/rehabilitation alternative has been selected and foundation loading has been developed and/or refined. We recommend that the existing abutment/pier footings be evaluated for sliding and eccentricity during subsequent phases of

design in accordance with AASHTO LRFD Sections 10.6.3.4 and 11.6.3.3, respectively. In addition, we recommend that the load and resistance factors provided in AASHTO LRFD Tables 3.4.1-1, 3.4.1-2, and 10.5.5.2.2-1 be used when evaluating external stability.

Depending on the magnitude of the loads, it may be possible that the existing spread footing foundations will not meet minimum acceptable design requirements and may require modification.

C. Steel H-piles

As discussed in previous sections of the report and summarized in Table II, select viaduct bridge abutments and piers are supported by HP10x42 steel H-piles. In general, the existing piles were evaluated by 1) reviewing historic geotechnical reports, 2) comparing estimated pile tip elevations to soil stratigraphy and top of bedrock elevation at each pile-supported substructure location, and 3) comparing maximum pile loads shown on the historic bridge plans to the estimated pile loads based preliminary factored axial compressive loads provided by Figg.

Based on our review of historic geotechnical reports we understand that piles were likely designed as end-bearing piles in/on bedrock. In general, due to the nature and consistency of the overburden soils at each abutment/pier location, as indicated by steel drill casing resistance during historic test boring drilling, it was determined that the piles could be driven to/into bedrock. The authors of the report specifically noted the potential for hard driving through surficial granular soil (believed to be man-placed fill soil) in the vicinity of Abutment 2. Based on our review of the historic test boring logs, we concur that the piles could have likely been driven to bedrock.

In addition, pile cutoff elevation and estimated pile length information shown on the historic viaduct bridge drawings were used to calculate pile tip elevations and were compared to the top of bedrock elevations determined from historic rod soundings and test borings. In general, the calculated pile tip elevations were found to be equal to or lower than the top of bedrock elevation and each abutment/pier location. Please note that the estimated pile tip elevations are based on estimated pile lengths that were likely developed prior to construction. We do not believe that as-built information (estimated pile lengths) is shown on the historic viaduct bridge drawings. Therefore, the drawings likely reflect the design intent, as summarized above and any changes or difficulties that may have occurred during construction were not documented or not included in the reference reports and drawings.

As discussed in previous sections of this report, historic viaduct bridge drawings suggest that abutment and pier piles were designed to resist maximum pile loads equal to 67.6 and 92.4 kips, respectively. Based on the preliminary substructure loading information provided by Figg (exclusive of Abutment 2) and the number of piles located within each abutment/pier pile group shown on the historic bridge plans and summarized on Table II, pile loads were calculated and compared to the existing maximum pile loads (67.6 and 92.4 kips). The resulting pile loads ranged from approximately 55 to 69 kips. As a result, from a geotechnical standpoint, it is our opinion that the existing pile-supported abutment/pier foundations are capable of supporting the rehabilitated superstructure and meet minimum design requirements. Please note that corrosion was not considered as part of the preliminary evaluations.

Similar to the existing spread footing foundation evaluations summarized above, tensile loads, lateral loads and moments were not considered during evaluation of existing pile-supported pier substructures. Therefore, we recommend that the adequacy of individual pile groups be re-evaluated once a preferred replacement/rehabilitation alternative has been selected and foundation loading has been developed and/or refined to determine the impacts of combined loading. Depending on the magnitude of the loads, it may be possible that the existing pile-supported abutment/pier foundations will not meet minimum acceptable design requirements and may require modification. We recommend that non-destructive testing and dynamic pile testing on a representative number of piles be considered for use in assessing existing pile integrity and estimating pile lengths and compression resistances during subsequent phases (preliminary/final design) of the project.

Historic Settlement and Global Stability Evaluations

Global stability and settlement evaluations were completed in the vicinity of Abutment 2 to support the original design and construction of the existing viaduct bridge.

Settlement evaluations were completed for the existing retaining walls and the embankment behind Abutment 2 using the historic viaduct bridge plans included in Appendix D and the consolidation test data (stress history and compressibility characteristics) on samples of marine silt/clay obtained from test boring nos. 21 and 19, respectively. The results of the historic settlement evaluations are summarized below.

- Retaining Wall(s): settlement ranging from approximately 3 to 8.5 in. along the length of the wall(s)
- Embankment: settlement ranging from approximately 2.5 (3 months after fill placement) to 4.5 in. ("ultimate")

Based on the magnitude of predicted settlement, it was recommended that the retaining wall be supported on piles. In addition, recommendations were made for embankment fill to be left in-place for 6 to 9 months before final paving in an attempt to minimize post-construction settlement.

Global stability evaluations were also completed modeling the approximate 15-ft raise in grade and utilizing the undrained shear strength data from test boring nos. 19 and 20. The calculated factor of safety against rotational failure of the embankment was equal to 2.9 and was judged to "be sufficiently high to allow for even softer material which may exist between the borings." The proposed embankment was judged to be stable.

Recent Global Stability and Settlement Evaluations

Based on discussions with you, it is our understanding that Figg is evaluating the potential for shortening the existing viaduct bridge by removing existing spans and replacing them with earthfill retained by near vertical-sided mechanically-stabilized earth (MSE) walls. Specifically, eliminating the superstructure between Abutment 1 and Pier 6, between Pier 7 and Pier 14, and between Pier 18 and

Abutment 2 are being investigated. As part of the feasibility evaluations, global stability and settlement evaluations were completed at each location. The evaluations and results are summarized below.

The subsurface soil conditions along the existing viaduct bridge alignment will affect the planning and design of the rehabilitation and/or replacement alternatives being considered. Removal of portions of the existing superstructure and replacement with retained earthfill could cause excessive vertical and lateral strains eventually resulting in a shear failure of the foundation soil and subsequent failure of the roadway (U.S. Route 1) as well as cause excessive post-construction vertical deformation (consolidation settlement) of the roadway.

As a result, a series of two-dimensional and one-dimensional global stability and consolidation settlement evaluations, respectively, were completed to assess the feasibility of shortening the viaduct bridge in select areas by removing existing spans and replacing them with earthfill without having to utilize lightweight fill, staged construction or other ground improvement techniques. Global stability evaluations were performed using the computer program Slide 6.0 to evaluate the likelihood of global stability failures at the site. In addition, settlement evaluations in the vicinity of Pier 18 and Abutment 2 were conducted using the computer program Settle 3D.

Typical soil profiles were developed based on the subsurface conditions encountered in both the historic and recent test borings. Soil properties used in the evaluations were developed from in-situ and/or laboratory test results (in the vicinity of Abutment 2; see Figure 3) or were assumed based on our experience (Abutment 1 to Pier 6 and Pier 7 to Pier 14). Please recall that only three test borings were drilled at select existing viaduct bridge substructure locations and variations in subsurface conditions are likely to exist transverse to the existing viaduct bridge structure (direction that global stability evaluations were completed). The evaluations are based on the assumption that subsurface conditions are somewhat uniform in these areas and similar to those encountered in the test borings.

Results of global stability and settlement evaluations completed along different portions of the existing viaduct bridge and associated feasibility-level recommendations are summarized separately, below.

A. Abutment 1 to Pier 6

- Global Stability

Stability evaluations were conducted parallel to the existing viaduct bridge baseline immediately downstation of Pier 5 and 6 where, based on our review of historic bridge plans, maximum raises in grade would be anticipated. In general, maximum raises along this portion of the existing viaduct bridge alignment would vary from approximately 13 ft (Abutment 1) to 26 ft (Pier 6). Please note that it is our opinion that conducting the stability evaluations parallel to the baseline at this location will result in a lower calculated factor of safety as compared to the transverse direction since the thickness of overburden generally increases from west to east, along the viaduct bridge alignment.

Since laboratory or in-situ shear strength data within the marine clay was not available in this area, our evaluations assumed the undrained shear strength was equal to 1,000 psf, which was

based on SPT N60 values within the deposit obtained from test boring BB-BROV-101. In addition, a 250 psf live load surcharge within the limits of U.S. Route 1 was included in our model.

The calculated factor of safety assuming that the existing viaduct bridge superstructure is replaced between Abutment 1 and Pier 6 or Pier 5 is approximately 1.3 and 1.7, respectively. The minimum required factor of safety as specified by both AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG) for "embankments" under static conditions, which support structures is 1.5.

- Settlement

Test boring BB-BROV-101, drilled immediately northwest of Pier 6, encountered approximately 5 ft of medium stiff to stiff SILT with sand and silty CLAY with sand. In addition, historic test borings drilled at existing substructure locations between Abutment 1 and Pier 5 did not encounter marine silt/clay soil. No historic or recent stress history or compressibility characteristic data was available for the marine silt/clay deposit present along this portion of the existing viaduct bridge alignment. As a result, settlement evaluations were not completed at this location.

- Feasibility-Level Recommendation

As a result of the stability and settlement evaluations summarized above, we consider replacement of the existing viaduct bridge superstructure between Abutment 1 and Pier 5 with a vertical-sided normal weight earthfill embankment to be technically feasible. We do not however consider replacement of the existing viaduct bridge superstructure between Pier 5 and Pier 6 with a vertical-sided normal-weight earthfill embankment to be technically feasible at this time without the use of ground modification or lightweight fill. We recommend that filling between Pier 5 and Pier 6 be re-evaluated during preliminary and final design once additional subsurface information is collected and the strength, stress history and compressibility characteristics of the marine silt/clay deposit are better defined along this portion of the existing viaduct bridge alignment.

B. Pier 7 to Pier 14

- Global Stability

Stability evaluations were conducted transverse to the existing viaduct bridge baseline immediately upstation (towards the river) of Pier 10 where, based on our review of historic bridge plans, maximum raises in grade would be anticipated. Please note that global stability evaluations were not conducted at other locations between Pier 7 and Pier 14. In general, the maximum raise in grade along this portion of the existing viaduct bridge alignment would be approximately 28 ft. Please note that it is our opinion that conducting the stability evaluations transverse to the baseline at this location will result in a lower calculated factor of safety as

compared to the longitudinal direction since the existing foundations supporting Piers 9 and 11, which provide additional resistance, are not considered.

Since very little undrained shear strength data was available within the marine clay deposit along this portion of the existing viaduct bridge alignment, we conducted a "sensitivity" analysis where the undrained shear strength of the marine clay was varied to see the range in calculated factors of safety and to understand how sensitive the factor of safety is to changes in undrained shear strength. The undrained shear strengths used in the evaluations ranged from 500 to 1,000 psf. In addition, a 250 psf live load surcharge within the limits of U.S. Route 1 was included in the model.

The calculated factor of safety for the condition summarized above ranged from approximately 0.7 to 1.3 for an upper layer of marine clay having an undrained shear strength equal to either 500 or 1,000 psf. The minimum required factor of safety as specified by both AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG) for "embankments" under static conditions, which support structures is 1.5.

- Settlement

As described above, the stress-strain or compressibility characteristics of marine clay deposits are highly dependent upon their stress history. No historic or recent stress history or compressibility characteristic data was available for the marine silt/clay deposit present along this portion of the existing viaduct bridge alignment. As a result, a range in the potential magnitude of consolidation settlement was calculated for varying degrees of overconsolidation ($1.0 < OCR < 4.0$) between Pier 10 and Pier 12. Please note that settlement evaluations were not conducted at other locations between Pier 7 and Pier 14. The magnitude of stress increase within the marine clay deposit, as a result of the proposed fill placement, was estimated by assuming a maximum fill height equal to 28 ft with normal weight earthfill weighing 125 pounds per cubic foot (pcf). Since the combination of existing vertical effective stress and the stress increase caused by fill placement will be greater than the maximum past pressure within the marine clay deposit, depending on which OCR is used, the magnitude of consolidation settlement will be a function of both the recompression ratio (RR) and the virgin compression ratio (CR) of the clay.

Estimates of settlement were at the center of the proposed fill area. Estimates of the magnitude of ground surface settlement using normal weight earthfill range from approximately 1 ft to 2.5 ft. As discussed above, the estimated magnitude of settlement will occur as both recompression and virgin compression since the combination of existing vertical effective stress and estimated stress increase within the marine clay layer is greater than the maximum past pressure within the lower portions of the marine clay deposit. As a result, we anticipate that some amount of post-construction settlement will occur without the use staged construction, lightweight fill and/or ground modification.

- Feasibility-Level Recommendation

As noted above, global stability and settlement evaluations were only conducted in the vicinity of Pier 10 only. Based on the stability and settlement evaluations summarized above, we do not consider replacement of the existing viaduct bridge superstructure in the vicinity of Pier 10 with a vertical-sided normal-weight earthfill embankment to be technically feasible without the use of ground modification or lightweight fill. Additional feasibility-level recommendations for replacing other viaduct bridge spans between Pier 7 and Pier 14 are provided below and are based on inspection of the subsurface conditions encountered in the historic rod soundings and test borings. We recommend that the feasibility-level recommendations summarized below be re-evaluated during the preliminary and/or final design phase(s) of the project.

- Pier 7 to Pier 8 – replacement of viaduct bridge superstructure with normal weight earthfill is not considered technically feasible without the use of lightweight fill, staged construction or ground modification at this time.
- Pier 8 to Pier 9 – replacement of viaduct bridge superstructure with normal weight earthfill is considered technically feasible at this time.
- Pier 11 to Pier 13 – replacement of viaduct bridge superstructure with normal weight earthfill is not considered technically feasible without the use of lightweight fill, staged construction or ground modification at this time.
- Pier 13 to Pier 14 – replacement of viaduct bridge superstructure with normal weight earthfill is considered technically feasible at this time.

C. Pier 18 to Abutment 2

- Global Stability

Stability evaluations were conducted transverse to the existing viaduct bridge baseline immediately upstation (towards the river) of Pier 18 where, based on our review of historic bridge plans, maximum raises in grade would be anticipated. In general, maximum raises in grade along this portion of the existing viaduct bridge alignment would vary from approximately 15 ft (Abutment 2) to 22 ft (Pier 18). Please note that it is our opinion that conducting the stability evaluations transverse to the baseline at this location will result in a lower calculated factor of safety as compared to the longitudinal direction since the existing foundation piles supporting Piers 18 and 19, which provide additional resistance, are not considered.

Organic and marine clay soils were modeled with the undrained shear strengths shown in Figure 3. In addition, a 250 psf live load surcharge within the limits of U.S. Route 1 was included in the model.

The calculated factor of safety for the condition summarized above is approximately 0.7. The minimum required factor of safety as specified by both AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG) for “embankments” under static conditions, which support structures is 1.5.

- Settlement

As described above, the stress-strain or compressibility characteristics of marine clay deposits are highly dependent upon their stress history. Based on the historic and recent laboratory test results, the maximum previous stress (i.e., maximum past pressure) within the marine clay deposit exceeds the existing vertical effective stress ($1.0 < OCR < 3.5$) as shown in Figure 3. The magnitude of stress increase within the marine clay deposit, as a result of the proposed fill placement, was estimated by assuming a maximum fill height equal to 22 ft with normal weight earthfill weighing 125 pounds per cubic foot (pcf). Since the combination of existing vertical effective stress and the stress increase caused by fill placement will be greater than the maximum past pressure within the lower portions of the marine clay deposit, the magnitude of consolidation settlement will be a function of both the recompression ratio (RR) and the virgin compression ratio (CR) of the clay.

Estimates of settlement were computed along the perimeter and within the "core" of the proposed fill area. Estimates of the magnitude of ground surface settlement using normal weight earthfill range from approximately 9 in. at the corners up to 22 in. within the central portion of the fill area.

As discussed above, the estimated magnitude of settlement will occur as both recompression and virgin compression since the combination of existing vertical effective stress and estimated stress increase within the marine clay layer is greater than the maximum past pressure within the lower portions of the marine clay deposit. As a result, we anticipate that some amount of post-construction settlement will occur without the use staged construction, lightweight fill and/or ground modification.

- Feasibility-Level Recommendation

As a result of the stability and settlement evaluations summarized above, we do not consider replacement of the existing viaduct bridge superstructure between Pier 18 and Abutment 2 with a vertical-sided normal-weight earthfill embankment to be technically feasible without the use of ground modification or lightweight fill.

CLOSURE

We trust this information meets your present needs. Please do not hesitate to contact us if you have any questions, comments or concerns related to information summarized in this report.

Sincerely yours,
HALEY & ALDRICH, INC.

Bryan C. Steinert, P.E.
Senior Geotechnical Engineer

Wayne A. Chadbourne, P.E.
Lead Geotechnical Engineer/Vice President

Enclosures:

Table I -	Existing Abutment/Pier Footing Summary
Table II -	Existing Abutment/Pier Pile Summary
Figure 1 -	Project Locus
Figure 2 -	Site and Subsurface Exploration Location Plan
Figure 3 –	Compressibility and Shear Strength Data for Marine Silt/Clay
Appendix A -	Test Boring Logs
Appendix B -	Existing Rock Slope Rock Mass Data
Appendix C -	Laboratory Test Results
Appendix D -	Historic Bridge Plans
Appendix E -	Calculations

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REFERENCES

1. Report entitled, "Subsurface Investigations For Carlton Bridge and West Approach in Bath, Maine," prepared by the Maine State Highway Commission, Soil Division, dated April 1957.

2. Report entitled, "Detailed Study in the Vicinity of the Abutment at Station 398+78, Carlton Bridge, Carlton, Maine," prepared by the Maine State Highway Commission, Soils Division, dated 8 May 1957.

3. Drawings entitled, "Carlton Bridge Widening and West Approach between Bath and Woolwich, Sagadahoc County, Maine, Federal Aid Project No. U-026-1(8)," prepared by the State of Maine State Highway Commission, dated January 1957, Sheet Nos. 1, 10-14, 33-39 of 148.

4. Report entitled, "Supplemental Field and Laboratory Investigation, Bath/Woolwich Bridge, Bath and Woolwich, Maine," prepared by Haley & Aldrich, Inc. for the Maine Department of Transportation, dated 30 May 1997.

TABLE I
EXISTING ABUTMENT/PIER FOOTING SUMMARY
ROUTE ONE VIADUCT BRIDGE REPLACEMENT
MAINEDOT WIN. 19273.00
BATH, MAINE

Substructure Location[1]	Footing Dimensions[2]			Bottom of Footing Elevation[2]	Approx. Top of Bedrock Elevation[3]	Vert. Distance Above/Below Bedrock (ft)	Construction Considerations
	Width (B, ft)	Length (L, ft)	Thickness (t, ft)				
Abutment 1	37	11	3	36.1	35.2	0.9	rock excavation/concrete fill required
Pier 1	13	10	3	34.3	37.6	-3.3	rock excavation required
Pier 2	13	10	3	31.5	36.0	-4.5	rock excavation required
Pier 3	13	10	3	29.1	28.1	1.0	concrete fill required
Pier 4	13	10	3	26.3	29.1	-2.8	rock excavation required
Pier 8	13	10	3	17.5	18.6	-1.1	rock excavation required
Pier 9	13	10	3	15.5	15.1	0.4	rock excavation/concrete fill required
Pier 13	13	10	3	9.7	10.3	-0.6	rock excavation required
Pier 14	13	10	3	8.0	6.7	1.3	concrete fill required

Notes:
[1] - See Figure 2 for the plan location of each substructure location.
[2] - Dimensions and elevations taken from historic bridge plans, sheet nos. 33, 36, 37 and 38 of 148, dated January 1957 and adjusted to NAVD 88 vertical datum; width (B) measured transverse to viaduct bridge alignment; length (L) measured parallel to viaduct bridge alignment.
[3] - Taken from referenced historic test boring logs. Elevations at rod sounding locations were scaled from historic profile. Elevations adjusted to NAVD 88 vertical datum.

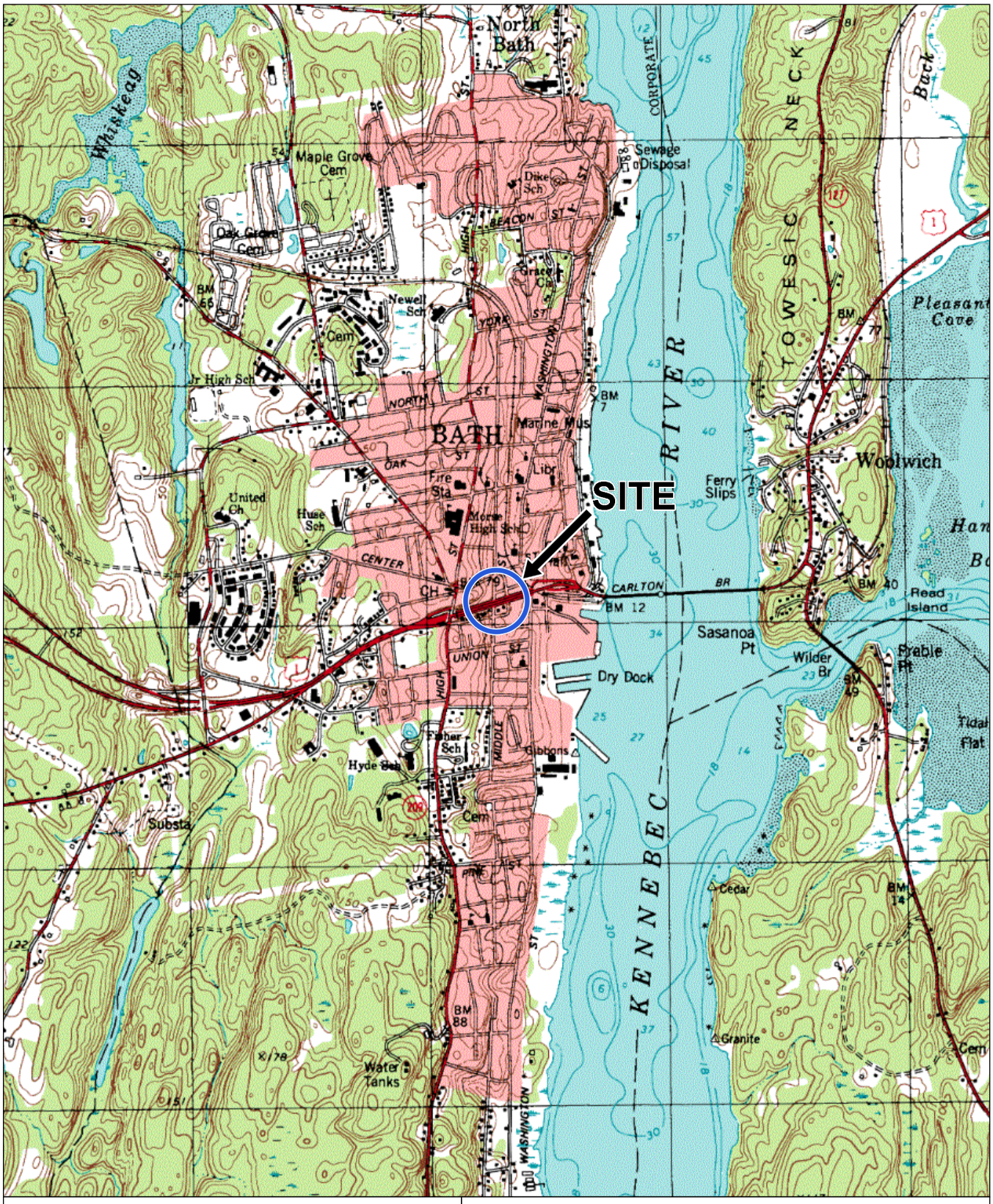
	Individual	Date
Prepared By:	BCS	4/17/2013
Checked By:	EFW	7/2/2013
Reviewed By:	WAC	9/6/2013

TABLE II
EXISTING ABUTMENT/PIER PILE SUMMARY
ROUTE ONE VIADUCT BRIDGE REPLACEMENT
MAINEDOT WIN. 19273.00
BATH, MAINE

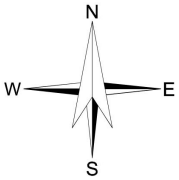
Substructure Location[1]	Pile Type[3]	Total No. of Piles[3]	Pile Cap Dimensions[2] (ft)			Pile Cutoff Elevation[2]	Estimated Pile Length[3] (ft)	Estimated Pile Tip Elevation[2]	Approximate Top of Bedrock Elevation
			Width (B, ft)	Length (L, ft)	Thickness (t, ft)				
Pier 5	HP10x42	16	16	14	4.0	23.9	15	8.9	14.6
Pier 6	HP10x42	16	16	14	4.0	21.5	25	-3.5	4.4
Pier 7	HP10x42	16	16	14	4.0	19.5	30	-10.5	-2.1
Pier 10	HP10x42	16	16	14	4.0	14.0	20	-6.0	2.1
Pier 11	HP10x42	16	16	14	4.0	12.5	15	-2.5	7.1
Pier 12	HP10x42	16	16	14	4.0	10.8	25	-14.2	-8.2
Pier 15	HP10x42	16	16	14	4.0	6.0	25	-19.0	-12.9
Pier 16	HP10x42	16	16	14	4.0	4.1	35	-30.9	-13.2
Pier 17	HP10x42	16	16	14	4.0	3.4	50	-46.6	-17.2
Pier 18	HP10x42	16	16	14	4.0	2.5	55	-52.5	-49.9
Pier 19	HP10x42	16	16	14	4.0	3.3	40	-36.7	-14.9
Abutment 2	HP10x42	19	35.3	10	4.0	4.6	35	-30.4	-22.9

Notes:
[1] - See Figure 2 for the plan location of each substructure location.
[2] - Dimensions and elevations taken from historic bridge plans, sheet nos. 33, 36, 37 and 38 of 148, dated January 1957 and adjusted to NAVD 88 vertical datum; width (B) measured transverse to viaduct bridge alignment; length (L) measured parallel to viaduct bridge alignment.
[3] - Taken from historic bridge plans, sheet nos. 36-38 of 148, dated January 1957.

	Individual	Date
Prepared By:	BCS	4/17/2013
Checked By:	EFW	7/2/2013
Reviewed By:	WAC	9/6/2013



SITE COORDINATES: 43°54'40"N,69°49'6"W



U.S.G.S. QUADRANGLE: BATH, ME

HALEY & ALDRICH

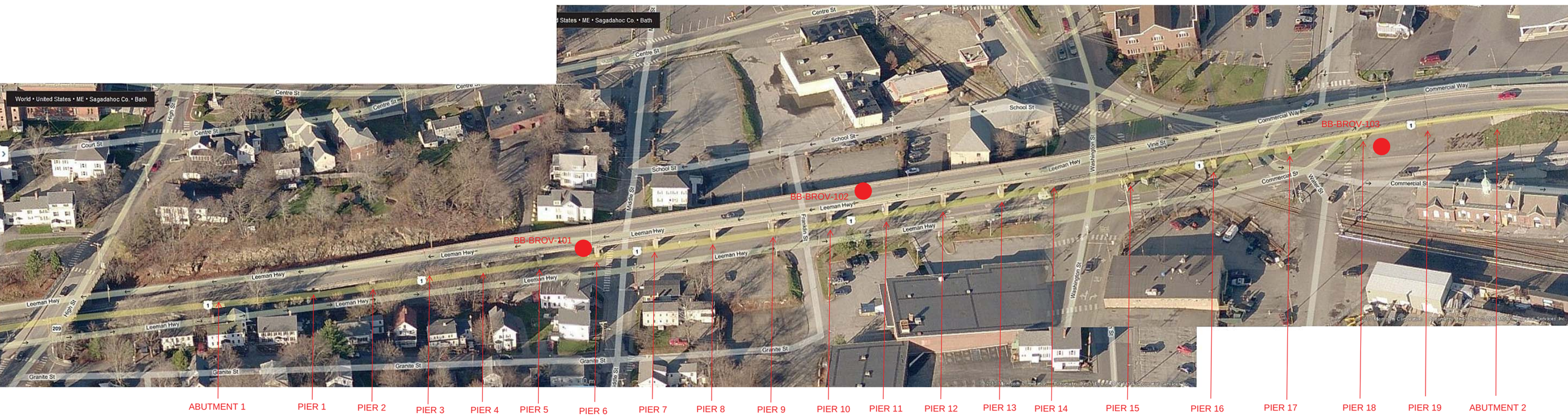
ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

PROJECT LOCUS

SCALE: 1:24,000
AUGUST 2013

FIGURE 1

38751-030 1.PDF



NOTES:

1. LOCATIONS OF TEST BORINGS ARE APPROXIMATE AND WERE DETERMINED BY TAPING/PACING DISTANCES FROM EXISTING SITE FEATURES.

2. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH, INC.

3. REFER TO APPENDIX A FOR TEST BORING LOGS AND APPENDIX C FOR HISTORIC BRIDGE DRAWINGS.

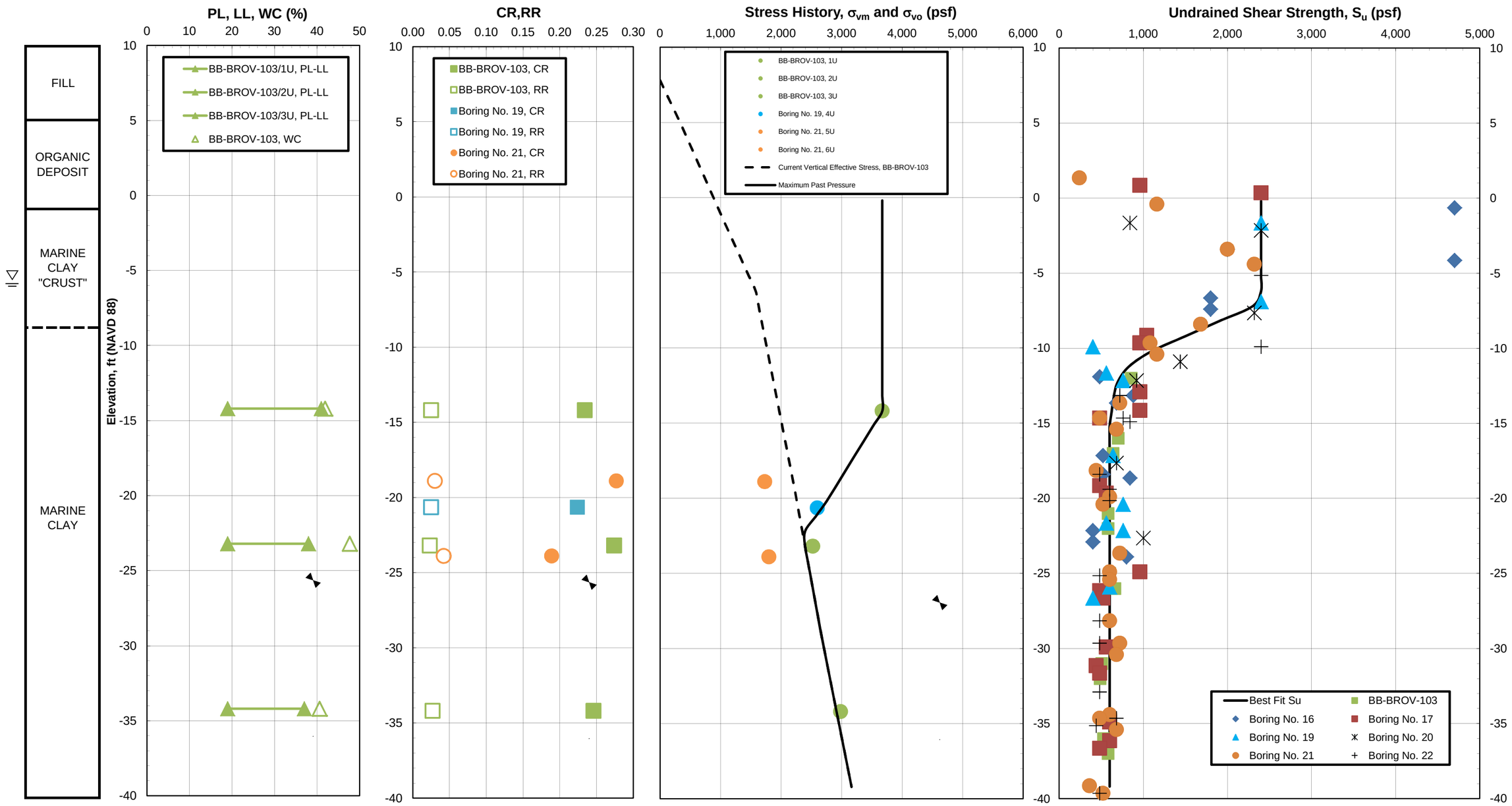
BB-BROV-101 — DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY MAINE TEST BORINGS OF HERMON, MAINE ON 26 AND 27 MARCH 2013

ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

SITE AND SUBSURFACE EXPLORATION
LOCATION PLAN

SCALE: NOT TO SCALE
SEPTEMBER 2013

FIGURE 2



NOTES:

1. Typical soil profile and in-situ vane test results from test boring BB-BROV-103 and Boring 17.
2. Compressibility data from laboratory incremental consolidation tests performed on specimens of marine clay trimmed from Shelby tube samples obtained from test borings BB-BROV-103, 19 and 21.
3. Design undrained shear strength profile developed by establishing a best-fit curve through in-situ vane shear test data shown.
4. Design preconsolidation pressure profile established using estimated values of preconsolidation pressure (Pp) from incremental consolidation tests.
5. RR = Recompression Ratio; CR = Virgin Compression Ratio; Po' = Existing Effective Overburden Pressure

COMPLETED BY:	BCS
CHECKED BY:	EFW
REVIEWED BY:	WAC

HALEY & ALDRICH

PROPOSED ROUTE 1 VIADUCT BRIDGE REPLACEMENT
MAINEDOT WIN 19237.00
BATH, MAINE

COMPRESSIBILITY AND SHEAR STRENGTH DATA FOR MARINE SILT/CLAY IN THE VICINITY OF ABUTMENT 2

NOT TO SCALE
APRIL 2013

FIGURE 3

G:\PROJECTS\38751 - MaineDOT Multi-PIN Support\030 - Sagadahoc Bridge Viaduct\Evaluations\[2013_0326_HAI_19273 Compressibility Data_rev1.xlsx]Compressibility Figure

APPENDIX A

Test Boring Logs

Recent Test Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 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)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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 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 thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*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)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-101
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.4	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	NW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	NW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131000, N393314 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.8
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							1		-BITUMINOUS CONCRETE-	
	1D	24/12	1.0 - 3.0	6/7/6/25	13	13	SSA	26.4	1.0 Brown, dry, medium dense, medium to fine SAND, little coarse sand, trace silt, little gravel -FILL-(SW) Note: Probable cobble from approximately 3.0 to 4.3 ft.	
	2D	24/22	4.3 - 6.3	4/6/6/5	12	12		23.1	4.3 Brown-gray, dry to moist, stiff mottled SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5	3D	24/22	6.3 - 8.3	6/6/6/5	12	12		21.1	6.3 Brown-gray, moist, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
	4D V1A	24/22	8.3 - 10.3	1/2/3/3 ~~Su>1940 psf~~	5	5	4	18.1	Brown, moist, medium stiff, silty CLAY with sand layers 55x110 mm vane raw torque reading: V1A: >500 in/lbs	
10							13		9.3 Brown, clayey, medium to fine SAND, little coarse sand, little gravel -GLACIAL TILL-(SC/SM)	
							42	15.4	12.0	
	R1	59/59	12.3 - 17.2	RQD = 59%			26(2.0")	15.1	-WEATHERED BEDROCK- 12.3 Top of Bedrock at El. 15.1 Gray, medium to fine-grained chlorite, biotite, SCHIST with frequent quartz and pegmatitic intrusions. Moderately hard, slightly weathered with occasional moderately weathered joints, low angle to moderately dipping, one steep secondary joint, very close to close, tight to open Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 12.3-13.3' (1:45); 13.3-14.3' (2:33); 14.3-15.3' (1:53); 15.3-16.3' (2:26); 16.3-17.2' (3:04)	GTX#300397 qp=2,321 psi
15								10.2	17.2 **Bottom of Exploration at 17.2 feet below ground surface.**	
20										
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-102
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	17.2	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131318, N393408 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.5 - 2.5	18/16/7/6	23	23	SSA	16.8	-BITUMINOUS CONCRETE- 0.4	
									Brown, dry, medium dense, fine to medium SAND, some coarse sand, trace silt, little gravel -FILL-(SW) 2.5	
	2D	24/14	2.5 - 4.5	5/10/11/16	21	21		14.7	Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	3D	24/22	4.5 - 6.5	9/10/10/10	20	20			Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5										
	4D	24/20	6.5 - 8.5	13/14/20/21	34	34			Brown-gray, dry to moist, hard, clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	5D	20/2	8.5 - 10.2	9/16/28/40(3.0")	44	44			Note: Recovered 2-in. brown, dry, dense, weathered rock.	
10	R1	62/62	10.7 - 15.9	RQD = 85%				7.2	10.0	
								6.5	-WEATHERED BEDROCK- 10.7	
									Top of Bedrock at El. 6.5 Tan-white-gray, medium to fine-grained GNEISS. Moderately hard, slightly weathered, joints low angle to moderately dipping, few steep secondary joints, very close to moderately close, tight to open, occasional silt coatings on joint surfaces. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 10.7-11.7' (2:08); 11.7-12.7' (1:08); 12.7-13.7' (1:19); 13.7-14.7' (1:31); 14.7-15.9' (1:00)	GTX#300397 qp=1,522 psi
15								1.3	15.9	
20										
25										
30										

Bottom of Exploration at 15.9 feet below ground surface.

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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

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

Boring No.: BB-BROV-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.5 - 2.5	6/20/30/6	50	50	SSA	7.7	-BITUMINOUS CONCRETE- 0.1	
									Brown, dry, dense, medium to fine SAND, little coarse sand, little gravel, trace silt -FILL-(SW)	
	2D	24/10	2.5 - 4.5	3/2/2/1	4	4		4.8	3.0	
									Dark brown, moist, soft, SILT, little organics (woody/fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	3D	24/24	4.5 - 6.5	1/1/1/1	2	2			Dark brown, moist, soft, SILT with organics, trace fine sand -ORGANIC DEPOSIT-(ML)	
5										
	4D	24/24	6.5 - 8.5	2/2/3/5	5	5			Dark brown, moist, medium stiff, SILT with organics (fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	5D	24/24	8.5 - 10.5	3/5/7/8	12	12		-0.7	8.5	
									Brown-gray, moist, stiff, mottled clayey SILT with organics (fibrous) -MARINE DEPOSIT-(ML)	
10	6D	24/24	10.5 - 12.5	13/12/14/12	26	26	36		Brown-gray, moist, very stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							35			
	7D	24/12	12.5 - 14.5	4/5/6/7	13	13	34		Note: Attempted field vane shear test at 12.0 ft, could not push. Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
							49			
15	8D	24/24	15.0 - 17.0	3/4/5/4	9	9	45		Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
	9D	24/18	17.0 - 19.0	1/2/2/2	4	4	40	-9.2	17.0 Gray, moist to wet, medium stiff, silty CLAY (CL) -MARINE DEPOSIT-(CL)	
							38			
	10D	24/24	19.0 - 21.0	WOH/WOH/WOH/ WOH			40		Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
20	V1A		19.6 - 20.0	Su=855/20 psf			51		55x110 mm vane raw torque readings: V1A: 220/6 in-lbs	
	U1		21.0 - 23.0				45			C#IP-1 WC=41.9% LL=41 PL=19, PI=22
							39			
	V2A		23.6 - 24.0	Su=700/95 psf			40		55x110 mm vane raw torque readings: V2A: 178/26 in-lbs	
	V2B		24.6 - 25.0	Su=640/115 psf			34		V2B: 165/30 in-lbs	
25	11D	24/24	25.0 - 27.0	WOR/WOR/WOR/ WOR			44		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							40			
							27			
	V3A		28.6 - 29.0	Su=580/80 psf			19		55x110 mm vane raw torque readings: V3A: 150/21 in-lbs	
30	V3B		29.6 - 30.0	Su=580/80 psf			13			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

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: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	U2		30.0 - 32.0				27		V3B: 150/20 in-lbs Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-3 WC=47.7% LL=38 PL=19, PI=19
							19			
	V4A		32.6 - 33.0	Su=600/40 psf			19		55x110 mm vane raw torque readings: V4A: 154/12 in-lbs V4B: 170/32 in-lbs	
	V4B		33.6 - 34.0	Su=660/115 psf			21			
							24			
35	12D	24/24	35.0 - 37.0	WOR/WOR/WOR/ WOR			23		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							25			
							22			
	V5A		38.6 - 39.0	Su=505/135 psf			25		55x110 mm vane raw torque readings: V5A: 130/34 in-lbs V5B: 127/11 in-lbs	
	V5B		39.6 - 40.0	Su=485/40 psf			27			
40							30			
	U3		41.0 - 43.0				30		Gray, wet, soft to medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-2 WC=40.6% LL=37 PL=19, PI=18
							25			
	V6A		43.6 - 44.0	Su=525/115 psf			30		55x110 mm vane raw torque readings: V6A: 137/31 in-lbs V6B: 150/33 in-lbs	
	V6B		44.6 - 45.0	Su=580/135 psf			35			
45							41			
							41			
							51	-39.0	46.8	
							70		Note: Sand and gravel observed in wash water at approximately 46.8 ft. -PROBABLE GLACIAL TILL-	
	13D	0/0	49.0 - 49.0	50(0")				-41.2	49.0	
50	R1	60/60	49.0 - 54.0	RQD = 52%					Top of Bedrock at El.-41.2 Gray, black and white, coarse to medium-grained pegmatitic SCHIST. Moderately hard, fresh to slightly weathered, joints low angle to moderately dipping, very close to close, tight to open, occasional weathered joint surfaces and silt coatings. Rock Mass Quality=Fair Recovery=100% R1 Core Times (min:sec): 49.0-50.0' (2:25); 50.0-51.0' (3:25); 51.0-52.0' (1:58); 52.0-53.0' (2:00); 53.0-54.0' (2:08)	
								-46.2		
55									54.0 **Bottom of Exploration at 54.0 feet below ground surface.**	
60										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

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

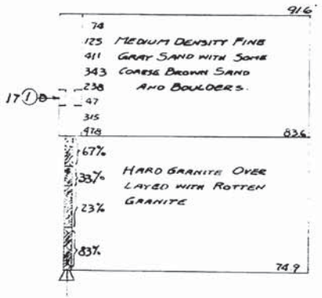
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Historic Test Boring Logs

F. A. P. No.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
	MAINE	U-026-1(8)	10	148

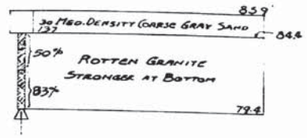
BORING NO. 1
STA 385+95 CENTER ABUT. #1

Elevation



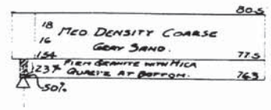
BORING NO. 2
STA. 387+15 CENTER PIER #2

Elevation



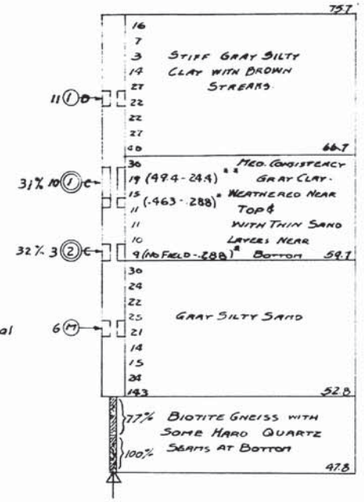
BORING NO. 3
STA. 388+37 CENTER PIER #4

Elevation



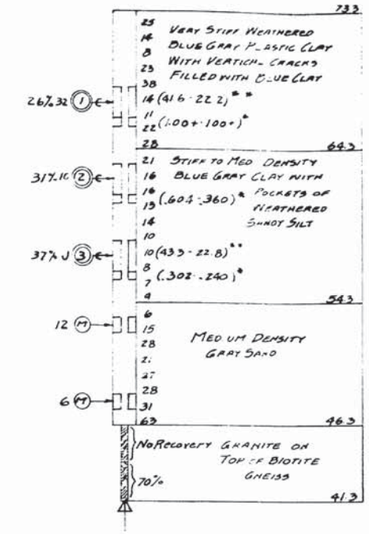
BORING NO. 4
STA 389+51 CENTER PIER #6

Elevation



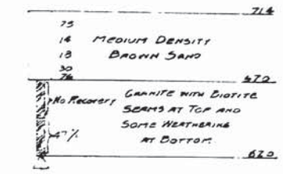
BORING NO. 5
STA 391+22 CENTER PIER #7

Elevation



BORING NO. 6
STA 392+68 CENTER PIER #8

Elevation



BORING NOTES

1. Scales as Noted on Drawings
2. Casing Size as Noted on Drawings.
3. Ground Water Table indicated thus:
4. Number of Blows of 275# Hammer falling 18 inches required to drive Extra Heavy Casing One Foot thus:
5. W, S, or V indicate respectively that the Hole was Washed, Sampled or Vaned ahead of Casing.
6. Location and designation of dry Samples taken in S.&H. sampler #1290s indicated thus:
7. Location and designation of dry samples taken in 2" O.D. 18 Ga. seamless Tubing indicated thus:
8. Location and designation of dry samples taken in 3½ O.D. 16 Ga. seamless tubing indicated thus:

LOG SHEETS — DETAIL SHEETS

40 — Graphical

(1) D — 1D

(1) C — 1C

(1) U — 1U

BORING NOTES

9. Location and designation of 3½ O.D. dry samples taken with piston sampler indicated thus:
10. Location and designation of wash samples indicated thus:
11. Unsuccessful attempts to secure dry sample indicated thus:
12. Location of field vane test indicated thus:
13. Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus
14. Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:
15. Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:

LOG SHEETS — DETAIL SHEETS

(1) P — 1P

(1) W — 1W

R — M

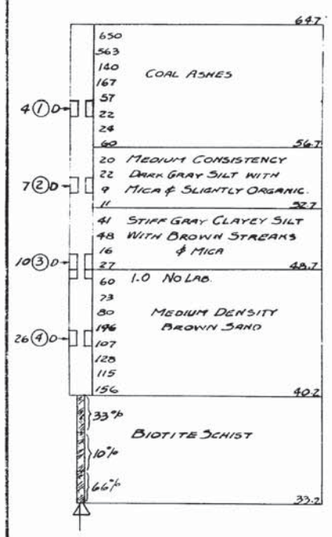
20 — GRAPHICAL

J — J

H — H

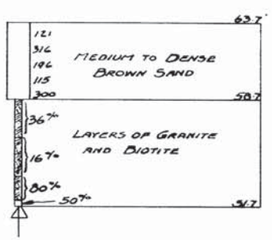
BORING NO. 7
STA 393+40 CENTER PIER #12

Elevation



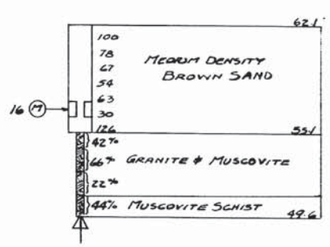
BORING NO. 8
STA 394+04 CENTER PIER #13

Elevation



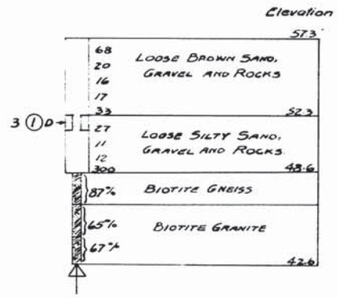
BORING NO. 9
STA 394+68 CENTER PIER #14

Elevation



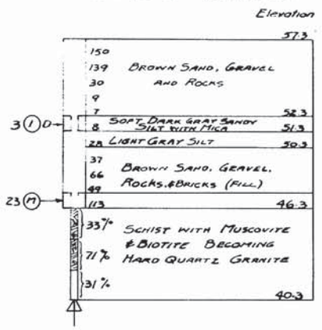
BORING NO. 10
LOCATION "O"
STA 403+12 15' RT. & RAMP

Elevation



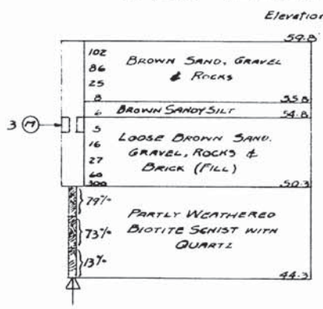
BORING NO. 11
LOCATION "M"
STA 402+59 14.5' RT. & RAMP

Elevation



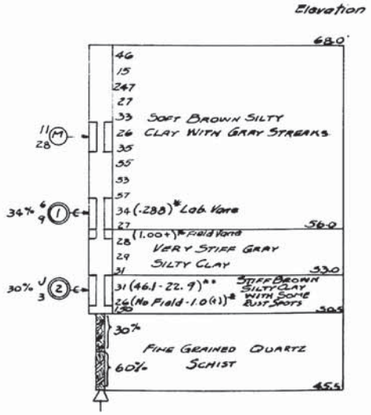
BORING NO. 12
LOCATION "K"
STA 402+01 14' RT. & RAMP

Elevation



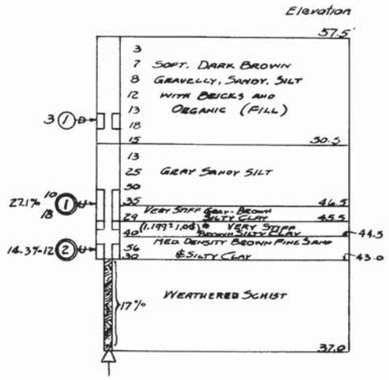
BORING NO. 14
STA 392+13 CENTER PIER #10

Elevation



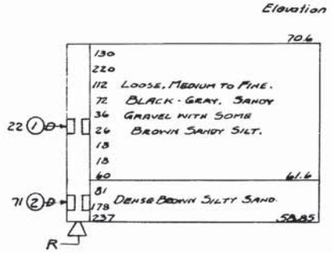
BORING NO. 15
LOCATION "E"
STA 400+34 47' RT. & TRESTLE

Elevation



BORING NO. 23
STA. 401+00 15' RT. & TRESTLE

Elevation



BORING NOTES

16. Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.
17. Natural water contents, given as percent of dry weight are indicated thus:
18. Bottom of boring indicated thus:
19. Refusal of drill rods or casing indicated thus:
20. % recovery of rock core by size E diamond bit thus:

LOG SHEETS — DETAIL SHEETS

31%-20 — Graphical

R — R

71% — 71%

NOTES:

* Field Vane Shear Strength Followed by Laboratory Vane Shear Strength in Tons/sq. ft.

** Liquid Limit Followed By Plastic Limit (Per Cent of Water Content)

SCALE 1" = 5 FEET.

DESIGN - Garrett
TRACE - JONES
CHECK - Langford

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

BORINGS

SHEET 10 OF 148 AUGUSTA, MAINE JAN. 1957

BORING NOTES

		LOG SHEETS	DETAIL SHEETS
1.	Scales as noted on drawings.		
2.	Casing size as noted on drawings.		
3.	Ground water table indicated thus:		
4.	Number of blows of 275# hammer falling 18 inches required to drive extra heavy casing one foot thus:	40	(graphical)
5.	W, S, or V indicate respectively that the hole was washed, sampled, or vaned ahead of casing.		
6.	Location and designation of "dry" samples taken in S&H sampler #1290s indicated thus:	I D	ID
7.	Location and designation of "dry" samples taken in 2" O.D. 18 ga. seamless tubing indicated thus:	I C	IC
8.	Location and designation of "dry" samples taken in 3½" O.D. 16 ga. seamless tubing indicated thus:	I U	IU
9.	Location and designation of 3½" O.D. "dry" samples taken with piston sampler indicated thus:	I P	IP
10.	Location and designation of wash samples indicated thus:	I W	IW
11.	Unsuccessful attempts to secure dry sample indicated thus:	M	M
12.	Location of field vane test indicated thus:		
13.	Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus:	20	(graphical)
14.	Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:	J	J
15.	Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:	H	H
16.	Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.		
17.	Natural water contents, given as percent of dry weight are indicated thus:	31% 20	graphical
18.	Bottom of boring indicated thus:		
19.	Refusal of drill rods or casing indicated thus:	R	R
20.	% recovery of rock core by size "E" diamond bit thus:	71%	71%

SHEAR AND WATER CONTENT NOTES

Shear Notes:

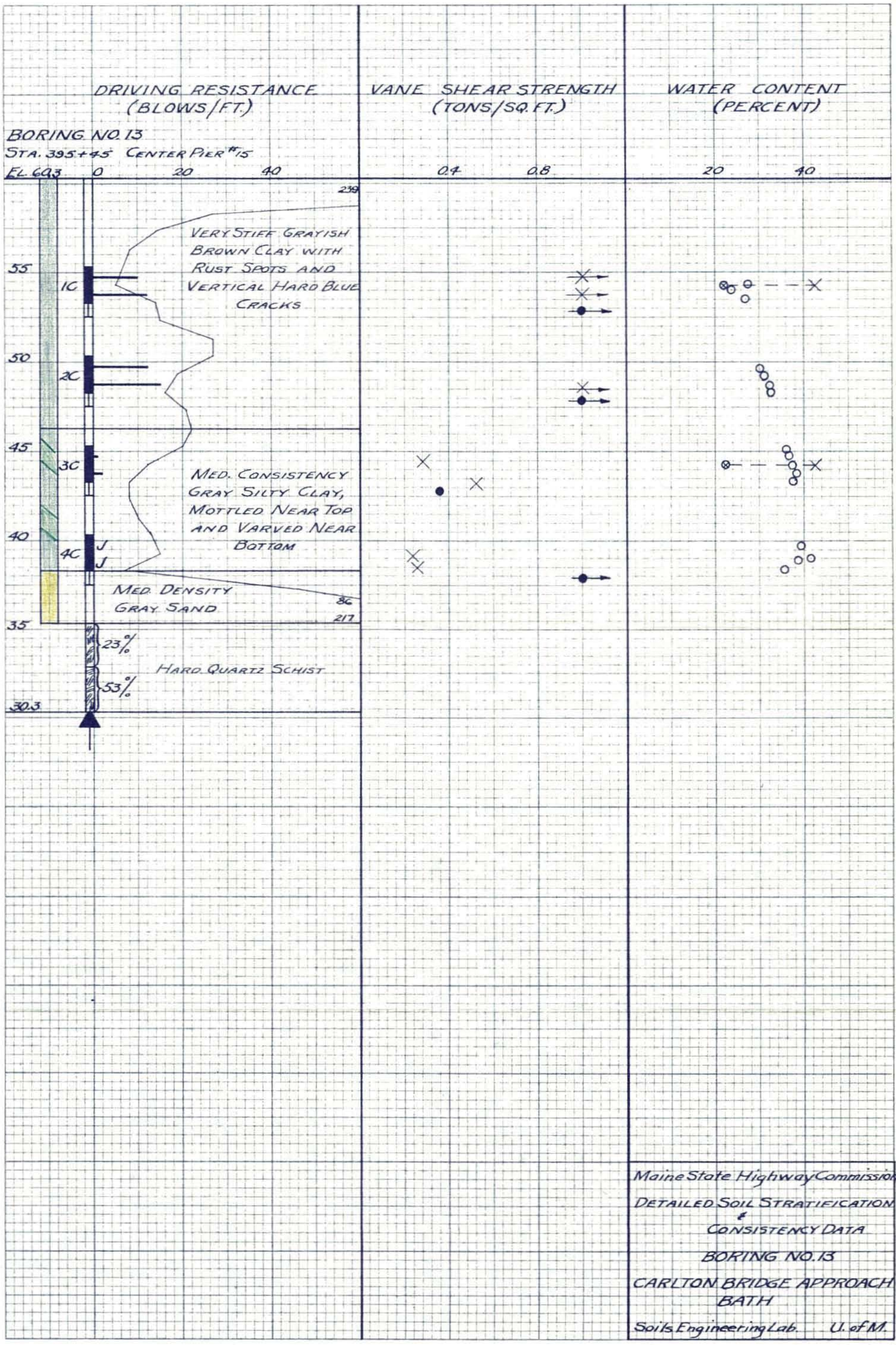
1. Field vane shear strengths indicated thus: ●
2. Laboratory vane shear strengths indicated thus: x
3. One half unconfined compressive strengths indicated thus: o
4. Strengths beyond range of plot indicated at right edge of plot by numerical values and symbols thus: 1.62●
5. Field vane shear strengths in excess of capacity of equipment indicated thus: —●→
6. Laboratory vane shear strengths in excess of capacity of equipment (1.0 T/sf) indicated thus: —x→
7. Field vane shear strengths in excess of capacity of equipment and beyond range of plot indicated at right edge of plot thus: 1.50(+)●
8. Laboratory vane shear strength in excess of capacity of equipment (1.0T/sf) and beyond range of plot indicated at right edge of plot thus: 1.00 (+)x

Water Content Notes:

1. Natural water contents, given as percent of dry weight, are indicated thus: o
2. Plastic and liquid limits are indicated thus: ⊗— — —x
3. Ignition losses are given as percent of dry weight.

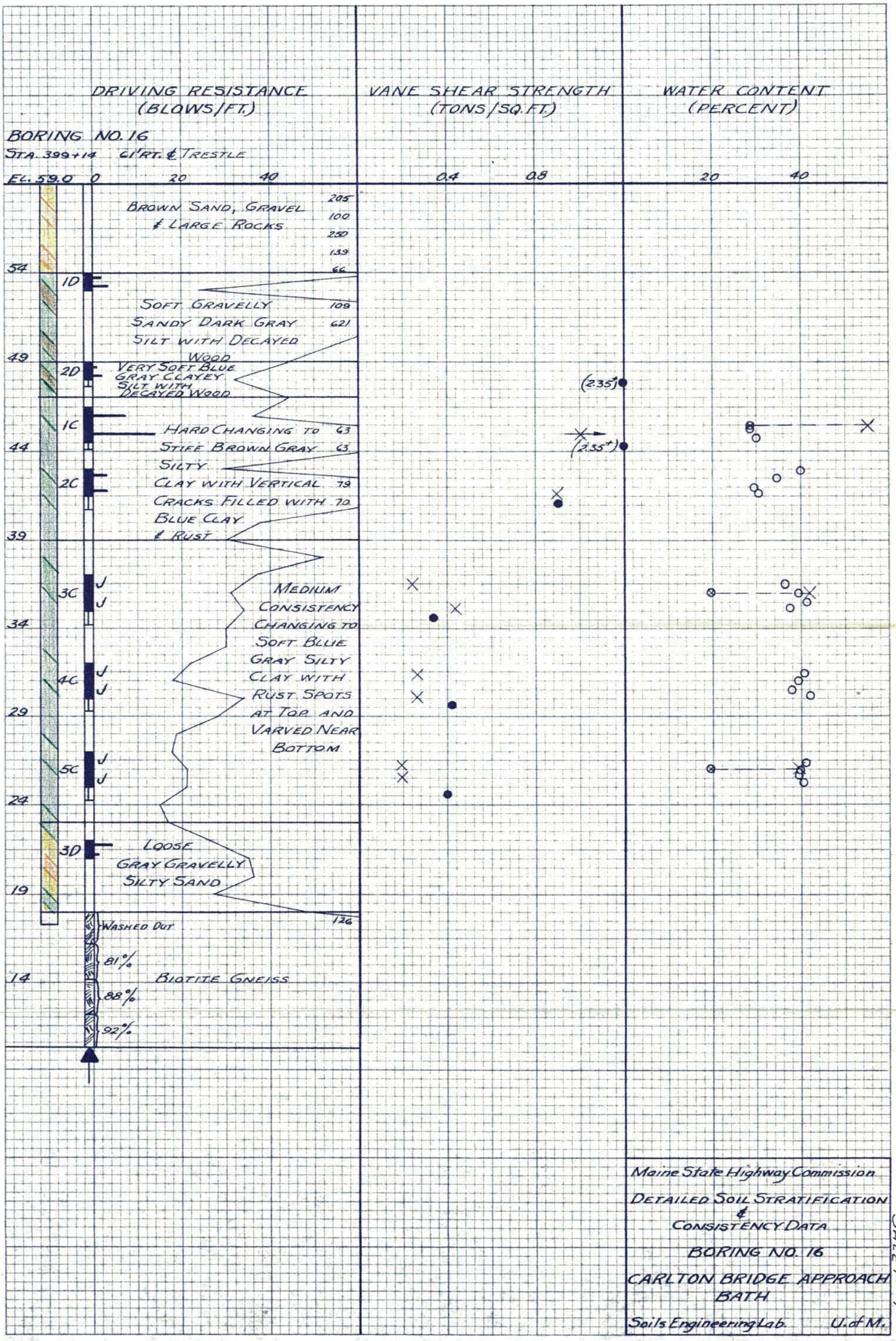
NO. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



10. 340D-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



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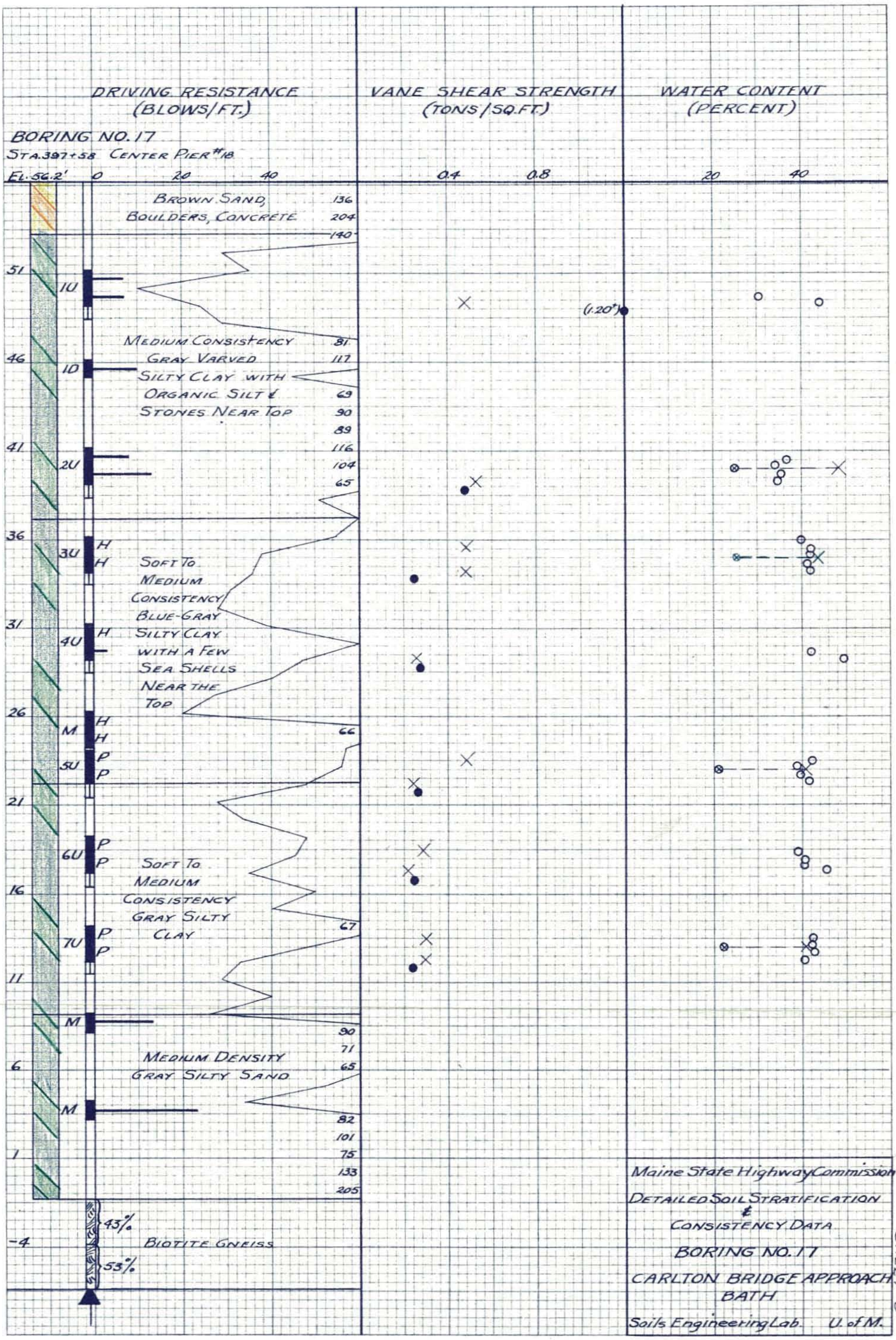
EUGENE DIETZGEN CO.
MADE IN U. S. A.



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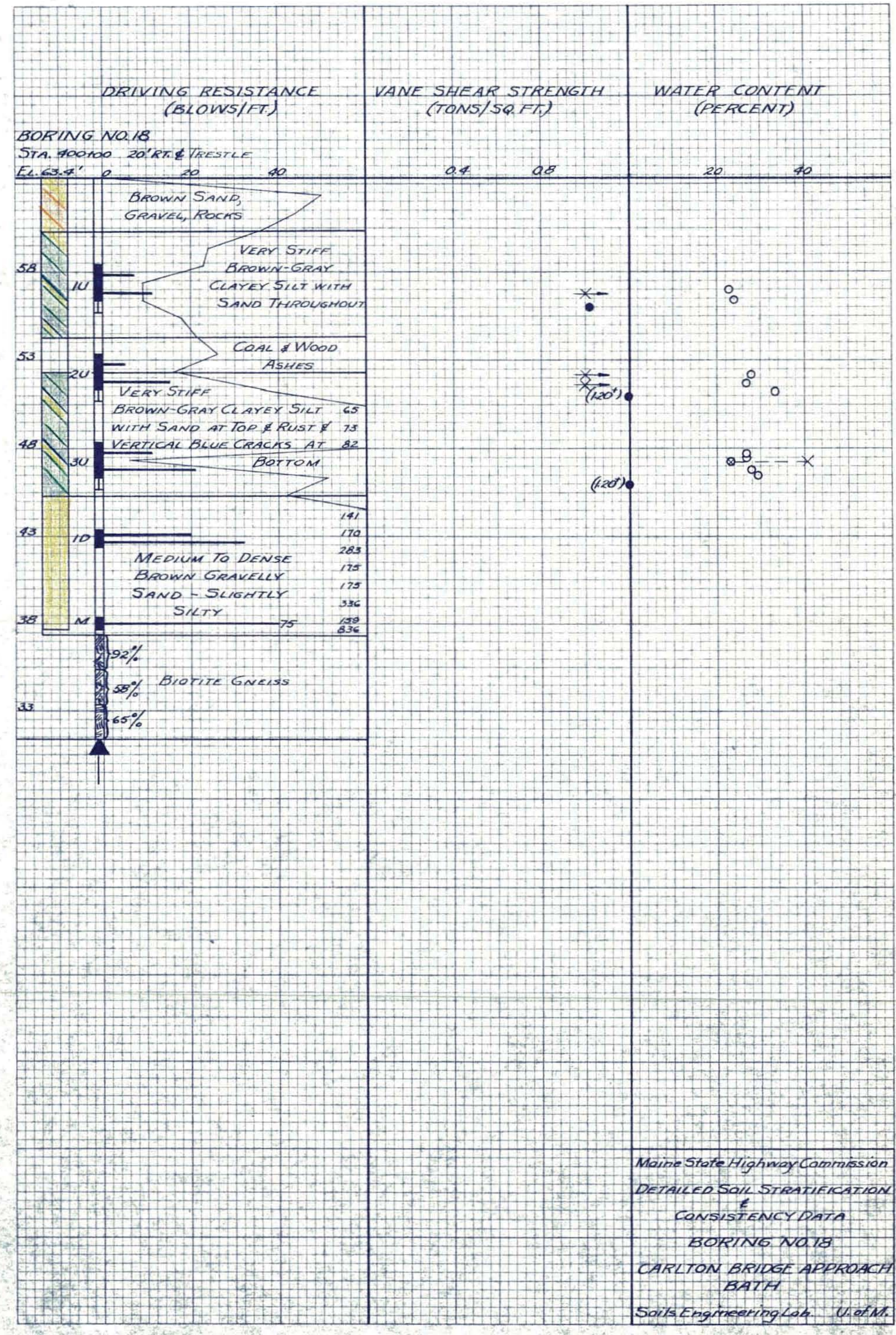
MADE IN U.S.A.



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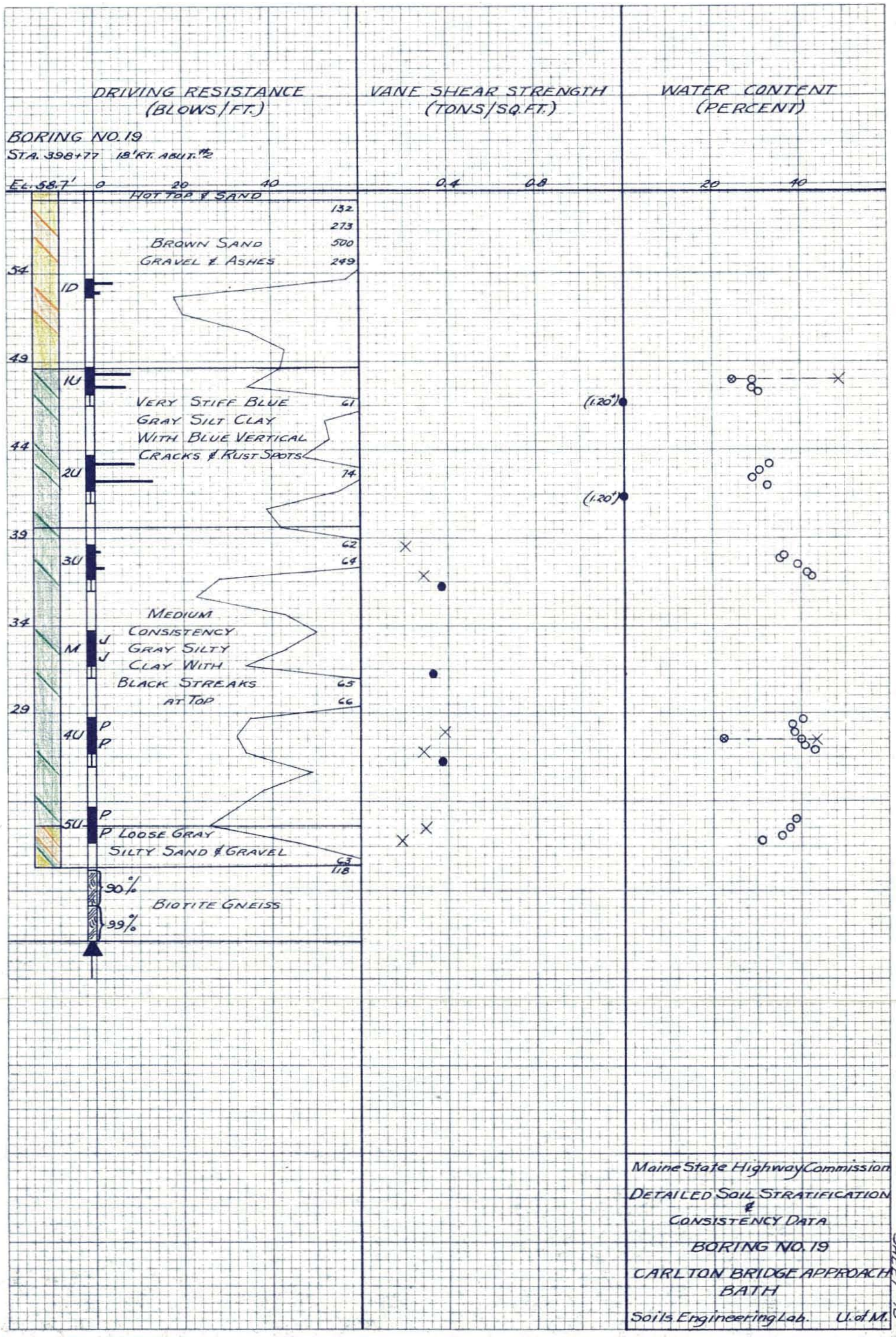
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING NO. 19

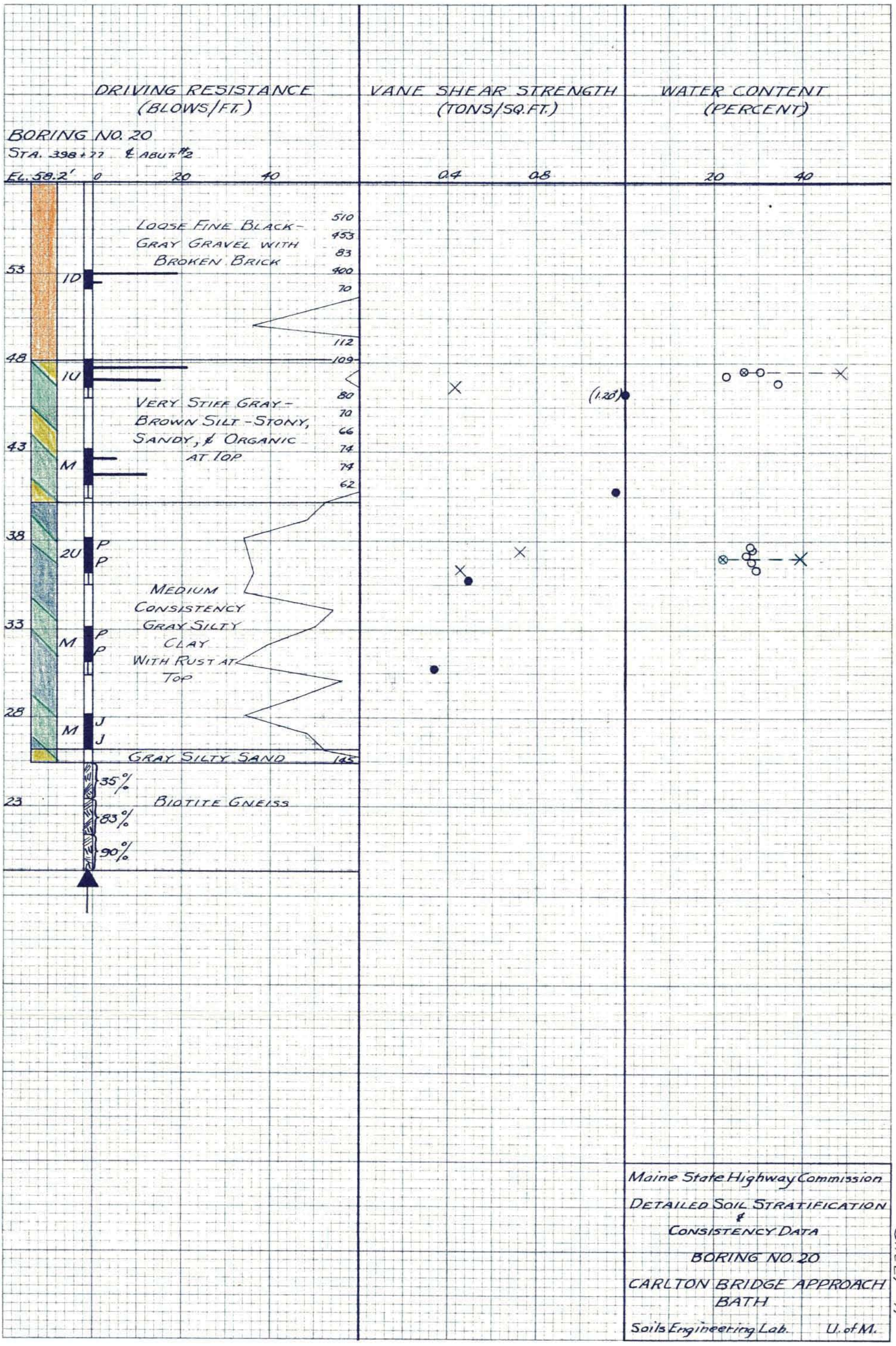
CARLTON BRIDGE APPROACH
BATH

Soils Engineering Lab. U. of M.

SHEET #10

10. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



NO. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

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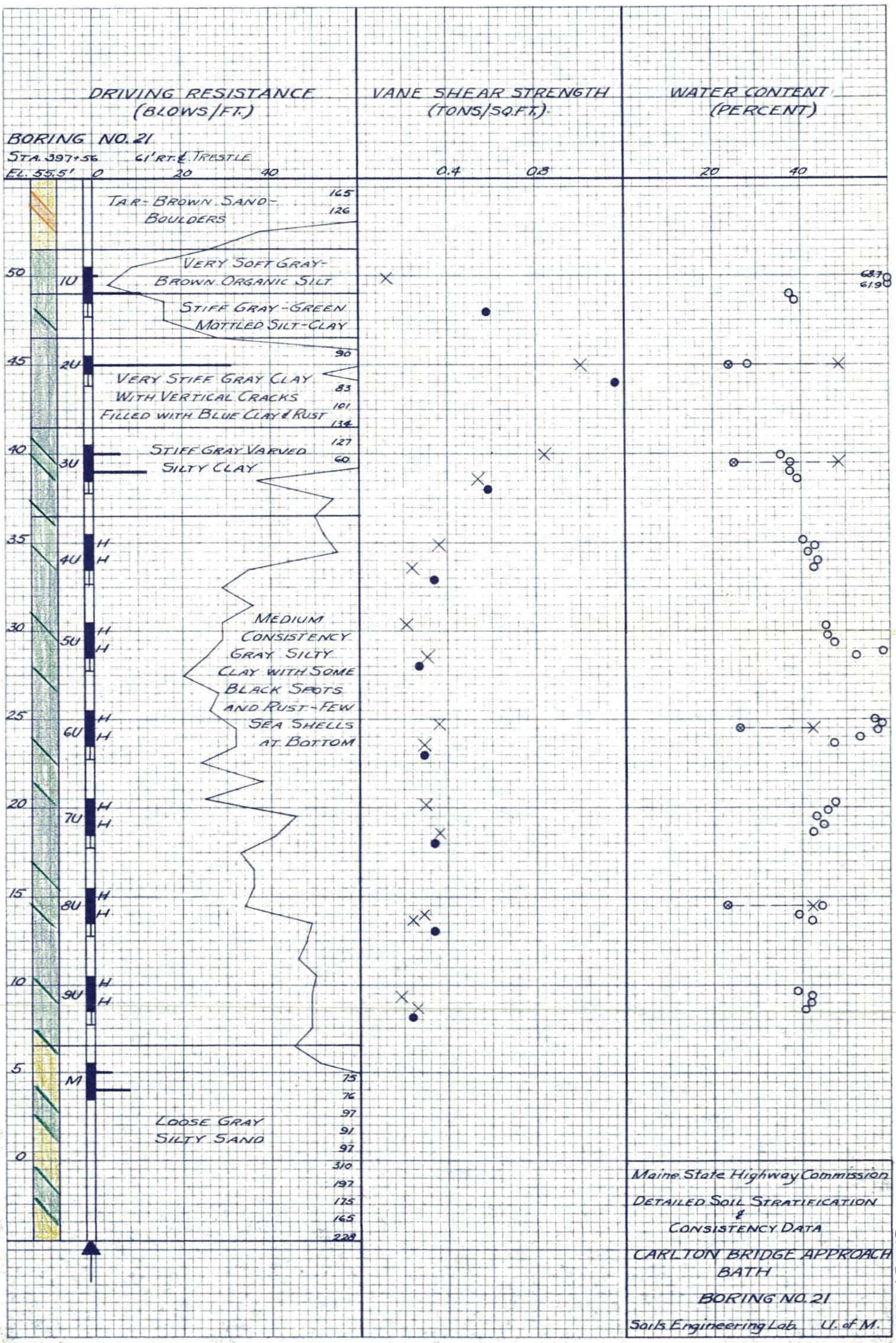
MADE IN U.S.A.



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

CARLTON BRIDGE APPROACH
BATH

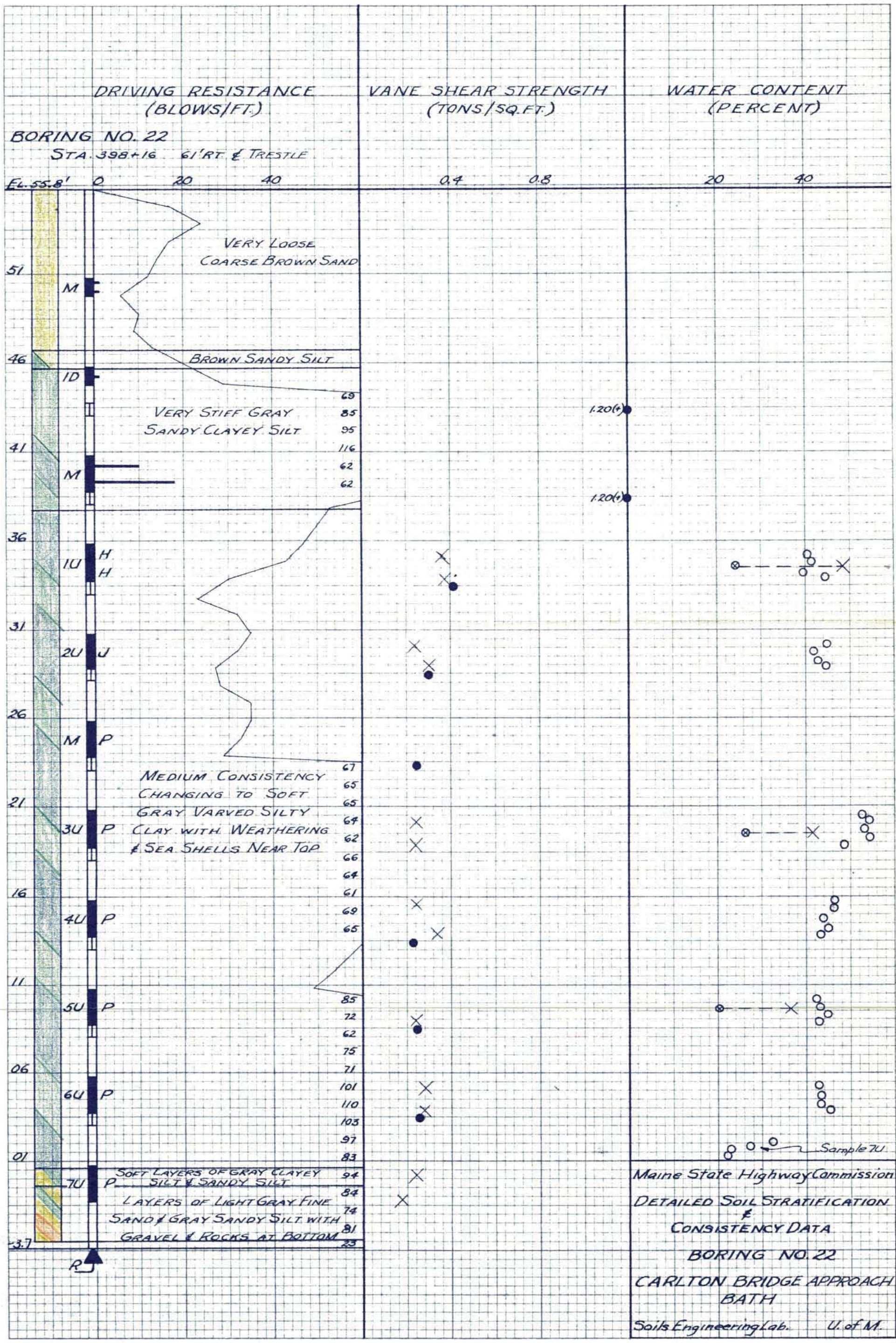
BORING NO. 21

Soils Engineering Lab. U. of M.

SHEET #12

10. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



APPENDIX B

Existing Rock Slope
Rock Mass Data

04/04/13 - 1

Western end of the existing rock slope, looking north.

04/04/13 - 2

Near the western end of the existing rock slope, looking north.

04/04/13 - 3

Central portion of the existing rock slope, looking northeast.

04/04/13 - 4

Central portion of the existing rock slope, looking northeast.

04/04/13 - 5

Near the eastern end of the existing rock slope, looking northeast.

04/04/13 - 6

Eastern end of the existing rock slope, looking northeast.

04/04/13 - 7

Eastern half of the existing rock slope, looking northwest.

04/04/13 - 8

View along the existing rock slope near the eastern end, looking west.

Discontinuity Survey Data Sheet

General Information

Location: BATH, MAINE

Project Name: RTE 1 VIADUCT

Project Number: 38751-030

Slope Height: ~20-~25'

Slope Length: ~350'

Slope Dip-Direction (South): Ground surface sloping east along alignment; Rock outcrop facing south

Domain: _____ of _____

GPS Coord: Northing: _____ Easting: _____

Compass Declination: 0

Associated Rock Mass Sheet: _____

Date: 4 (Month) 4 (Day) 2013 (Year)

Inspector(s): M. SNOW

_____ page of _____ page

HALEY & ALDRICH

A Start of Alignment = 50' from bridge underpass wall (tunnel) on southbound side (seep/low) of Rte. 1 (west end of outcrop)

Reference: FHWA-HI-99-007

Nature and Orientation of Discontinuity

BASE LINE (West to East)	~~Chainage or Depth~~	STRIKE	Type	Dip	Dip Direction	Persistence	Apperture Width	Nature of Filling	Strength of Filling	Surface Roughness	Surface Shape	JRC	Water Flow	Spacing	Remarks
0+		N5°5°E	Joint	79°	325°	4'	1/8"	—	—	R	P	6-8	None	6"-2'	Hard, slightly weathered, medium to coarse
50'		N77°E	"	54°	347°	5'	1/8"	—	—	R	P to S	8-10	"	6"-2'	grained GRANITE
		N335W	J	56°	257°	8'	0.25-0.5mm	—	—	R	U to S	10-12	"	8'	0'-50'- massive to blocky
		N325W	Joint (block)	62°	53°	12'	1"-4"	—	—	R	U to S	14-16	"	2' wide block w/ parallel joints	
	*	N84°E	B	22°	174°	15'	1"-3"	broken rock fragments / open	R4	R	U	12-14	"	3'-4'	50-100'- horizontal beds, low angle dip,
50'-		N296°W	J	66°	206°	10'	1/4"-1/2"	—	—	R	U	12-14	"	2' (292)	parallel, undulating beds
100'		N285°W	J	54°	195°	35'	1/4"	—	—	R	P	6-8	"	2'	
	*	S105°E	B	17°	195°	>20'	1"-3"	broken rock fragments	R4	R	U	8-10	"	2.5'-4'	water: slight seep from soil above
100'-		N62°E	J	83°	332°	>20'	0.5mm	—	—	R	P to U	8-10	"	6"-2'	outcrop in occasional areas (~50' apart)
150'		N64°E	J	83°	334°	>20'	0.5mm to open face	—	—	R	P to S	8-10	"	6"-2'	
		N78°E	J	82°	348°	10'	1/8"	—	—	R	P to S	8-10	"	2.5'	100-150'- steep, parallel joints 6"-1.5' apart
		N78°E	J	88°	348°	10'	0.1-0.5mm	—	—	R	P to S	12-14	"	2.5'	140'- seep @ 4' + 10' above ground surface
		S92°E	B	14°	182°	5'	not visible soil covered bed			R	S	14-16	"	10'	observed on rock face.
155' wedge rt. side		N337°W	J	40°	247°	5'	0.5mm	—	—	R	U	8-10	"	12'	~130'- seep @ 7' above ground surface
		N75°E	J	78°	345°	8'	open face	—	—	R	P to S	8-10	"	5'-6'	
150'-		N37°E	J	74°	307°	14'	0.5-1/8"	—	—	R	P to S	8-10	seepage, no flow	2.5'	150'- wedge feature (see photo)
200'		N73°E	J	22°	343°	4'	0.5-2.5mm	—	—	R	U	8-10	slight seep no flow	2.5'	
		N42°E	J	78°	312°	6'	" "	—	—	R	P to S	6-8	" "	2.5'	~180'- water seeping to flowing (several drops per second)
200'		N56°E	B	25°	146°	5'	0.5-2.5mm	—	—	R	P	8-10	—	6'	(see photo)
250'		N87°E	B	11°	177°	14'	1/8"	—	—	R	P to U	8-10	slight seep	6'	
		N53°E	J	90°		25'	1/8"	—	—	R	P to S	8-10	—	4"-3'	~250' becoming more massive toward
		N82°E	B	9°	172°	8'	soil covered bed	—	—	R	P	10-12	—	3'	east end of alignment
250'-		N297°W	J	59°	27°	4'	"	—	—	R	P	8-10	—	2.5'-3.5'	~285'- large, open face vertical joint
350'		N309°W	J	52°	38°	15'	1/8"	—	—	R	P to U	10-12	—	4"-2'	continuous (>20')
		N75°E	B	24°	165°	20'	1/8"-3/8"	—	—	R	U	12-14	—	3'-10'	
		N304°W	J (large)	90°		>20'	0.5mm-1/8"	—	—	R	P	6-8	—	>10'	
		N72°E	J	90°		10'-12'	open face	—	—	R	P to U	8-10	—	2'	

Type
1. Bedding — 0-90 NE
2. Small Joint — 90-180 SE
3. Large Joint — 180-270 SW
4. Sealed Joints — 270-360-NW
5. Foliations
6. Faults
7. Not defined

Aperture Width
1. Very Tight (<0.1 mm)
2. Tight (0.1-0.25 mm)
3. Partly open (0.25-0.5 mm)
4. Open (0.5-2.5 mm)
5. Moderately wide (2.5-10mm) - 1/8" - 3/8"
6. Wide (> 10 mm)
7. Very wide (1-10 cm) 3/8" - 3 7/8"
8. Extremely wide (10-100 cm) 3 7/8" - 39 3/8"
9. Cavernous (> 1 m) > 39 3/8"

Persistence

1. Very low persistence	< 1 m	< 3.3 ft
2. Low persistence	1 - 3 m	3.3 - 10 ft
3. Medium persistence	3 - 10 m	10 ft - 33 ft
4. High persistense	10 - 20 m	33 - 66 ft
5. Very high persistence	> 20 m	> 66 ft

Surface roughness
1. Rough
2. Smooth
3. Polished
4. Slickensided

Surface shape
1. Stepped
2. Undulating
3. Planar

Nature of Infilling
1. Clean
2. Surface Staining
3. Non-cohesive
4. Inactive clay or clay matrix
5. Swelling clay or clay matrix
6. Cemented
7. Chloride, talc, or gypsum
8. Calcite

Termination
0. Neither end visible
1. One end visible
2. Both ends visible

Spacing

1. Extremely close spacing	< 20 mm	< 0.8 in
2. Very close spacing	20 - 60 mm	0.8 - 2.4 in
3. Close spacing	60 - 200 mm	2.4 - 8.0 in
4. Moderate spacing	200 - 600 mm	8.0 in - 2.0 ft
5. Wide spacing	600 - 2,000 mm	2.0 - 6.6 ft
6. Very wide spacing	2,000 - 6,000 mm	6.6 - 20.0 ft
7. Extremely wide spacing	> 6,000 mm	> 20.0 ft

Compressive Strength of Infilling

	Mpa	Psf
S1 Very soft clay	< 0.025	500 psf
S2 Soft Clay	0.025-0.05	500-1,000 psf
S3 Firm clay	0.05-0.10	1,000-2,000 psf
S4 Stiff clay	0.10-0.25	2,000-5,000 psf
S5 Very stiff clay	0.25-0.50	5,000-10,000 psf
S6 Hard clay	>0.50	>10,000 psf
R0 Extremely weak rock	0.25-1.0	36 - 145 psi
R1 Very weak rock	1.0-5.0	145 - 725 psi
R2 Weak rock	5.0-25	725 - 3,625 psi
R3 Medium strong rock	25-50	3,625 - 7,250 psi
R4 Strong rock	50-100	7,250 - 14,500 psi
R5 Very strong rock	100-250	14,500 - 36,250 psi
R6 Extremely strong rock	>250	> 36,250 psi

Water Flow (open)
0. The discontinuity is very tight and dry: water flow along it does not appear possible.
1. The discontinuity is dry with no evidence of water flow.
2. The discontinuity is dry but shows evidence of water flow. i.e. rust staining.
3. The discontinuity is damp but no free water is present.
4. The discontinuity shows seepage, occasional drops of water, but no continuous flow.
5. The discontinuity shows a continuous flow of water. (Estimate l/min and describe pressure i.e. low medium, high)

Water Flow (filled)
6. The filling materials are heavily consolidated and dry: significant flow appears unlikely
7. The filling materials are damp, but no free water is present.
8. The filling materials are wet, occasional drops of water.
9. The filling materials show signs of outwash, continuous flow of water (estimate l/,lm).
10. The filling materials are washed out locally; considerable water flow along out-wash channels (estimate l/min and describe pressure, i.e.low, medium, high).

APPENDIX C

Laboratory Test Results

Recent Laboratory Test Results

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

Atlanta
Boston
Chicago
New York
San Francisco

www.geotesting.com

Transmittal

TO:

Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

DATE: 5/3/2013	GTX NO: 300397
RE: U.S. Route 1 Viaduct Bridge Replacement	

COPIES	DATE	DESCRIPTION
	5/3/2013	**April 2013 Laboratory Test Report**

REMARKS:

SIGNED: Joe Tomei, Laboratory Manager

CC:

APPROVED BY: Nancy Hubbard, Project Manager

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

Atlanta
Boston
Chicago
New York
San Francisco

www.geotesting.com

May 3, 2013

Bryan Steinert
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617

RE: U.S. Route 1 Viaduct Bridge Replacement, Bath, ME (GTX-300397)

Dear Bryan:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received six samples from you between 3/28/2013 and 4/2/2013. These samples were labeled as follows:

Boring	Sample	Depth
BB-BROV-101	R1	14.4-15.2 ft.
BB-BROV-102	R1	13.8-14.5 ft.
BB-BROV-103	R1	52.8-53.5 ft.
BB-BROV-103	U1	21.0-23.0 ft.
BB-BROV-103	U2	30.0-32.0 ft.
BB-BROV-103	U3	41.0-43.0 ft.

GTX performed the following tests on these samples:

3 ASTM D2216 - Moisture Contents
3 ASTM D4318 - Atterberg Limits
3 ASTM D2435 - Incremental Consolidation tests
2 ASTM D7012D- Elastic Moduli of Rock in Uniaxial Compression

A copy of your test request is attached.

The results presented in this report apply only to the items tested. This report shall not be reproduced except in full, without written approval from GeoTesting Express. The remainder of these samples will be retained for a period of sixty (60) days and will then be discarded unless otherwise notified by you. Please call me if you have any questions or require additional information. Thank you for allowing GeoTesting Express the opportunity of providing you with testing services. We look forward to working with you again in the future.

Respectfully yours,

Joe Tomei
Laboratory Manager

Atlanta
Boston
Chicago
New York
San Francisco

www.geotesting.com

GeoTesting EXPRESS

Technologies to manage risk for infrastructure

Geotechnical Test Report

5/3/2013

GTX-300397
U.S. Route 1 Viaduct Bridge Replacement

Bath, ME

Client Project No.: 38751-030

Prepared for:

Haley & Aldrich, Inc.

Client:	Haley & Aldrich, Inc.				
Project:	U.S. Route 1 Viaduct Bridge Replacement				
Location:	Bath, ME			Project No:	GTX-300397
Boring ID:	---	Sample Type:	---	Tested By:	jek
Sample ID:	---	Test Date:	04/09/13	Checked By:	jdt
Depth :	---	Test Id:	263125		

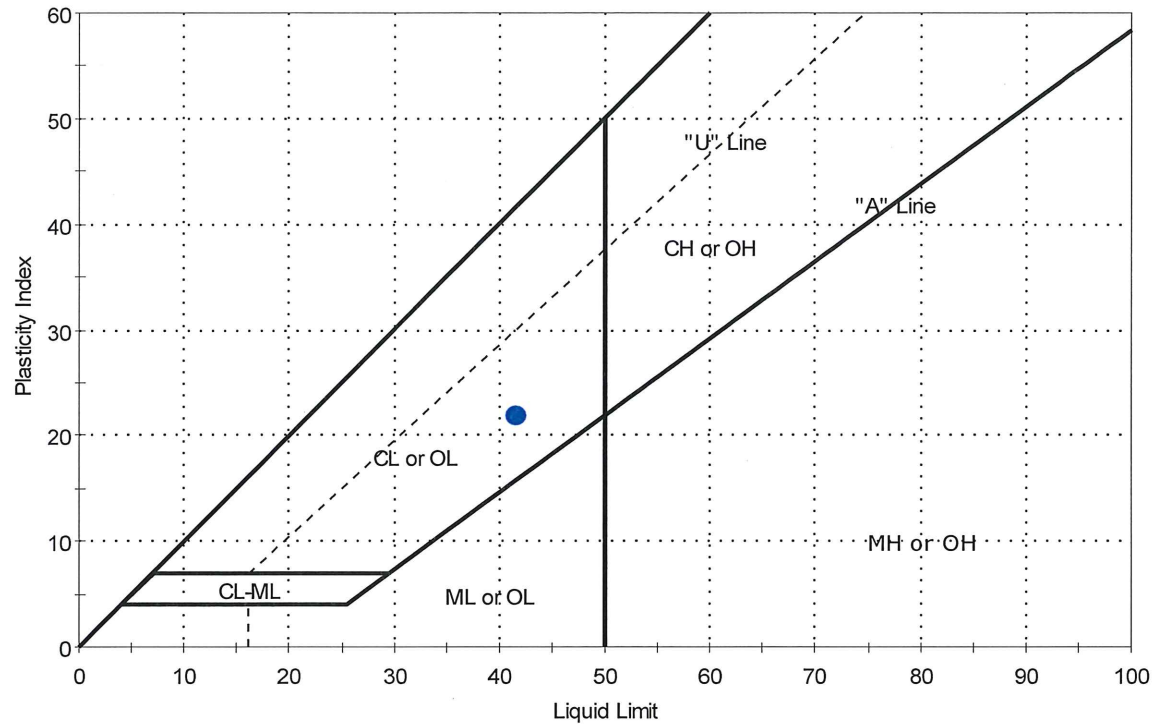
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BROV-103	U1	21.0-23.0 ft.	Moist, gray clay	41.9
BB-BROV-103	U2	30.0-32.0 ft.	Wet, gray clay	47.7
BB-BROV-103	U3	41.0-43.0 ft.	Wet, gray clay	40.6

Notes: Temperature of Drying : 110° Celsius

Client:	Haley & Aldrich, Inc.				
Project:	U.S. Route 1 Viaduct Bridge Replacement				
Location:	Bath, ME			Project No:	GTX-300397
Boring ID:	BB-BROV-103	Sample Type:	tube	Tested By:	cam
Sample ID:	U1	Test Date:	04/05/13	Checked By:	jdt
Depth :	21.0-23.0 ft.	Test Id:	263120		
Test Comment:	---				
Sample Description:	Moist, gray 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
●	U1	-BROV-1	21.0-23.0 ft.	42	41	19	22	1	

Sample Prepared using the WET method

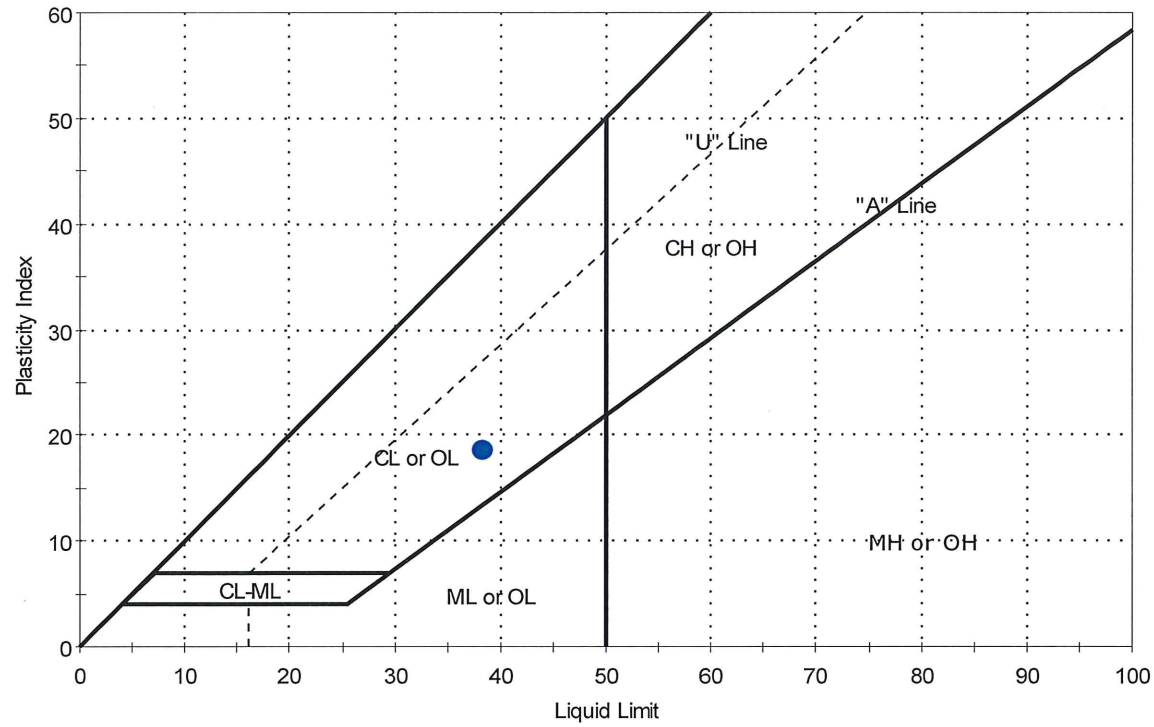
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	U.S. Route 1 Viaduct Bridge Replacement				
Location:	Bath, ME			Project No:	GTX-300397
Boring ID:	BB-BROV-103	Sample Type:	tube	Tested By:	cam
Sample ID:	U2	Test Date:	04/04/13	Checked By:	jdt
Depth :	30.0-32.0 ft.	Test Id:	263121		
Test Comment:	---				
Sample Description:	Wet, gray 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
●	U2	-BROV-1	30.0-32.0 ft.	48	38	19	19	1	

Sample Prepared using the WET method

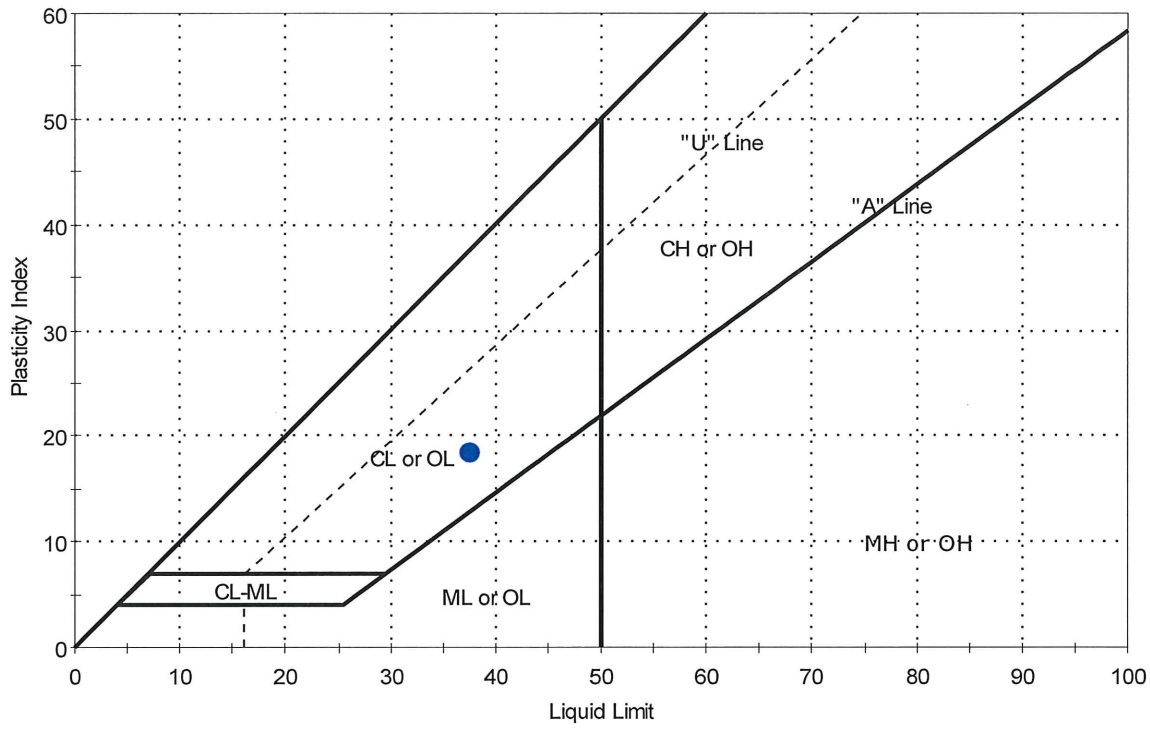
Dry Strength: VERY HIGH

Dilatancy: NONE

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	U.S. Route 1 Viaduct Bridge Replacement				
Location:	Bath, ME			Project No:	GTX-300397
Boring ID:	BB-BROV-103	Sample Type:	tube	Tested By:	cam
Sample ID:	U3	Test Date:	04/05/13	Checked By:	jdt
Depth :	41.0-43.0 ft.	Test Id:	263122		
Test Comment:	---				
Sample Description:	Wet, gray 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
●	U3	-BROV-1	41.0-43.0 ft.	41	37	19	18	1	

Sample Prepared using the WET method

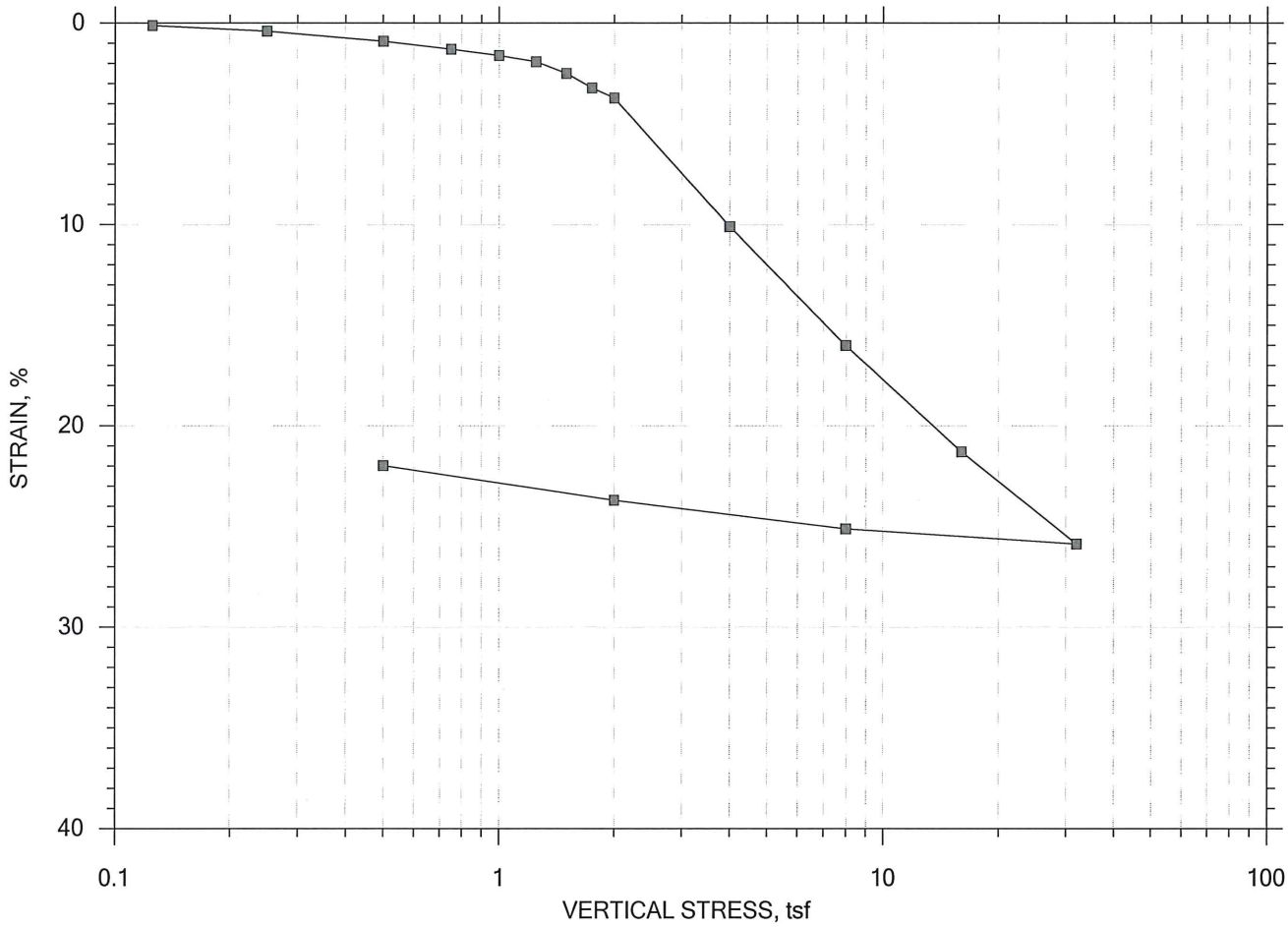
Dry Strength: VERY HIGH

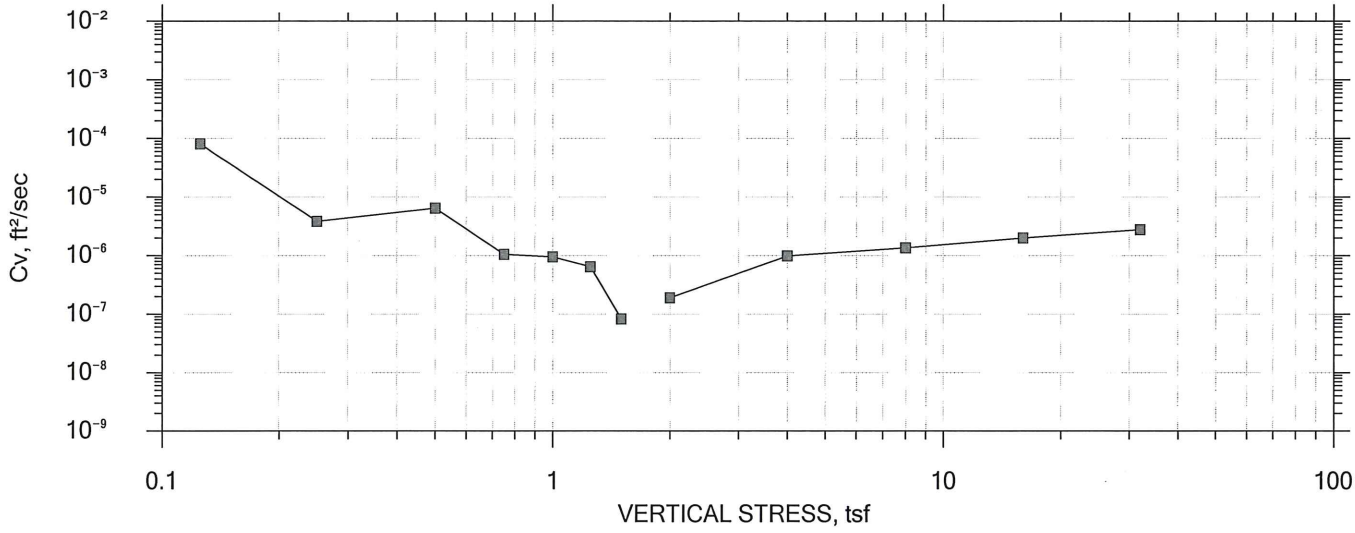
Dilatancy: SLOW

Toughness: LOW

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

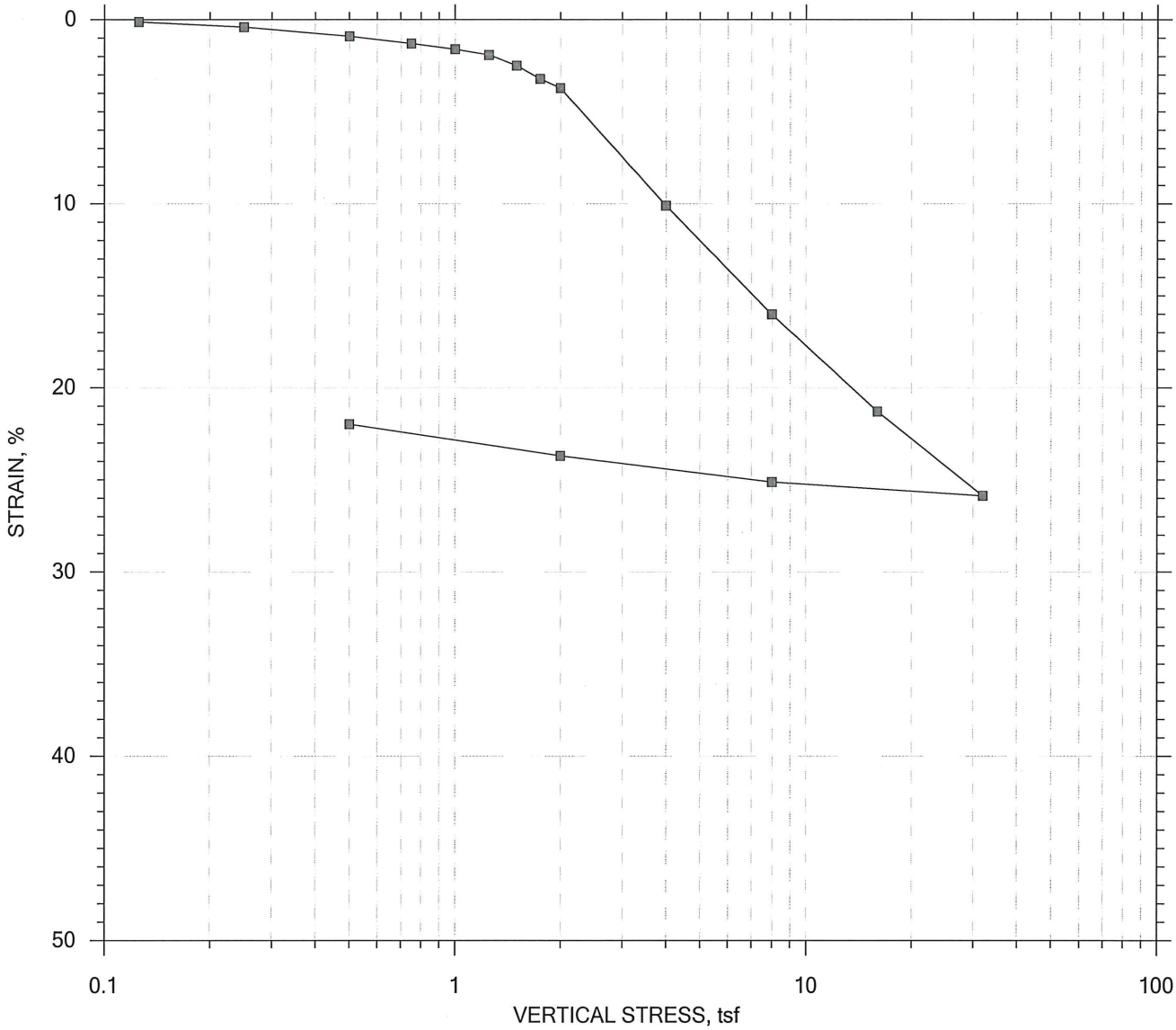




GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		
	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, %	41.90	23.22
Preconsolidation Stress: ---				Dry Unit Weight, pcf	79.566	106.09
Compression Ratio: ---				Saturation, %	97.81	100.00
Diameter: 2.5 in		Height: 1 in		Void Ratio	1.20	0.65
LL: 41	PL: 19	PI: 22	GS: 2.81			

GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: U.S. Rt. 1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U1
Test No.: IP-1

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: remolded

Project No.: GTX-300397
Checked By: jdt
Depth: 21.0-23.0 ft
Elevation: ---

Soil Description: Moist, gray clay
Remarks: System X

Estimated Specific Gravity: 2.81
Initial Void Ratio: 1.20
Final Void Ratio: 0.652

Liquid Limit: 41
Plastic Limit: 19
Plasticity Index: 22

Specimen Diameter: 2.50 in
Initial Height: 1.00 in
Final Height: 0.75 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	10851	RING	EE	11316
Wt. Container + Wet Soil, gm	182.43	254.34	235.19	134.77
Wt. Container + Dry Soil, gm	130.24	211.38	211.38	110.80
Wt. Container, gm	7.5200	108.86	108.86	7.5800
Wt. Dry Soil, gm	122.72	102.52	102.52	103.22
Water Content, %	42.53	41.90	23.22	23.22
Void Ratio	---	1.20	0.652	---
Degree of Saturation, %	---	97.81	100.00	---
Dry Unit Weight, pcf	---	79.566	106.09	---

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: U.S. Rt. 1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U1
Test No.: IP-1

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: remolded

Project No.: GTX-300397
Checked By: jdt
Depth: 21.0-23.0 ft
Elevation: ---

Soil Description: Moist, gray clay
Remarks: System X

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	0.125	0.001202	1.20	0.120	0.103	2.38e-004	9.61e-003	6.18e-003
2	0.250	0.004049	1.19	0.405	6.873	3.55e-006	2.28e-002	2.18e-004
3	0.500	0.008938	1.18	0.894	4.114	5.89e-006	1.96e-002	3.11e-004
4	0.750	0.01283	1.17	1.28	21.966	1.09e-006	1.56e-002	4.59e-005
5	1.00	0.01597	1.17	1.60	23.697	1.01e-006	1.26e-002	3.41e-005
6	1.25	0.01916	1.16	1.92	34.966	6.77e-007	1.28e-002	2.33e-005
7	1.50	0.02487	1.15	2.49	238.338	9.85e-008	2.28e-002	6.06e-006
8	1.75	0.03216	1.13	3.22	0.000	0.00e+000	2.92e-002	0.00e+000
9	2.00	0.03707	1.12	3.71	106.135	2.15e-007	1.97e-002	1.14e-005
10	4.00	0.1009	0.980	10.1	19.993	1.06e-006	3.19e-002	9.16e-005
11	8.00	0.1601	0.850	16.0	13.927	1.33e-006	1.48e-002	5.31e-005
12	16.0	0.2129	0.734	21.3	8.240	1.97e-006	6.60e-003	3.51e-005
13	32.0	0.2587	0.633	25.9	4.813	2.98e-006	2.86e-003	2.30e-005
14	8.00	0.2512	0.649	25.1	2.276	5.98e-006	3.12e-004	5.03e-006
15	2.00	0.2370	0.681	23.7	6.324	2.22e-006	2.37e-003	1.42e-005
16	0.500	0.2198	0.718	22.0	33.375	4.38e-007	1.14e-002	1.35e-005

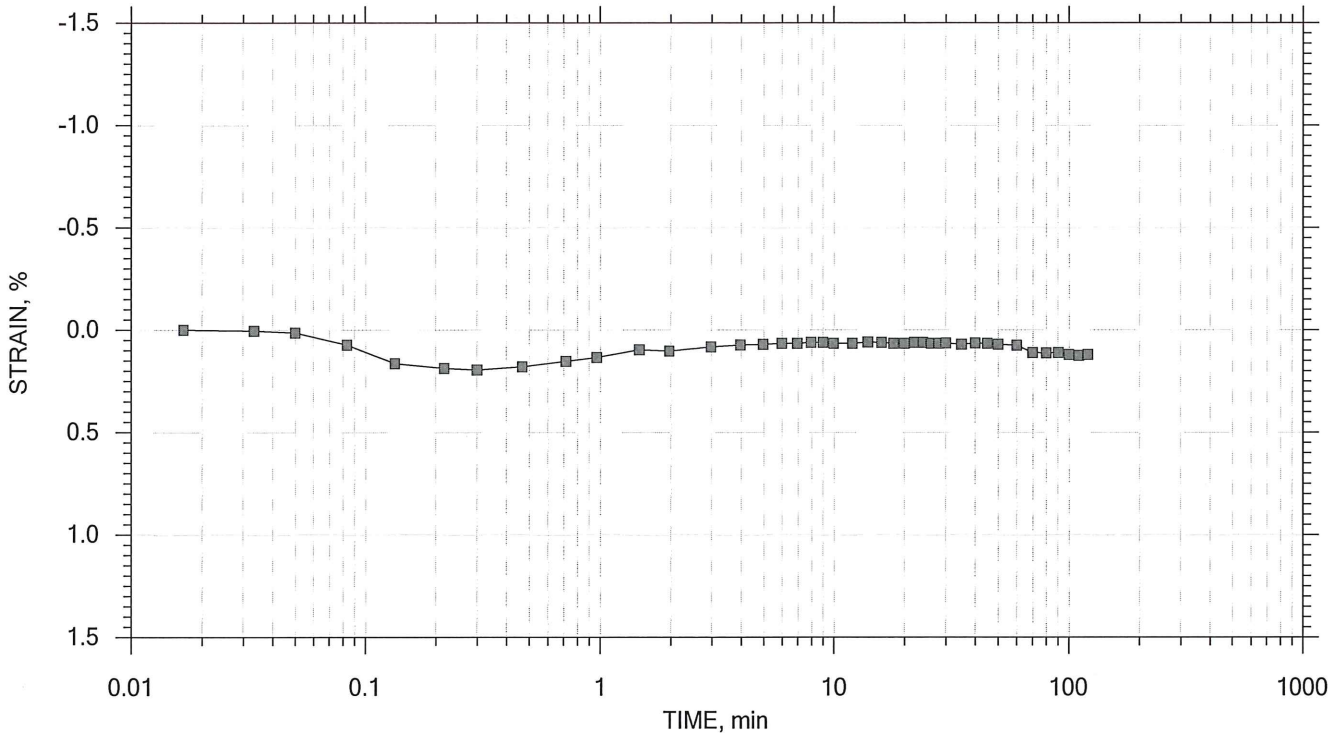
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.125	0.001202	1.20	0.120	0.000	0.00e+000	9.61e-003	0.00e+000	0.00e+000
2	0.250	0.004049	1.19	0.405	1.356	4.18e-006	2.28e-002	2.57e-004	0.00e+000
3	0.500	0.008938	1.18	0.894	0.000	0.00e+000	1.96e-002	0.00e+000	0.00e+000
4	0.750	0.01283	1.17	1.28	0.000	0.00e+000	1.56e-002	0.00e+000	0.00e+000
5	1.00	0.01597	1.17	1.60	0.000	0.00e+000	1.26e-002	0.00e+000	0.00e+000
6	1.25	0.01916	1.16	1.92	9.182	5.99e-007	1.28e-002	2.06e-005	0.00e+000
7	1.50	0.02487	1.15	2.49	0.000	0.00e+000	2.28e-002	0.00e+000	0.00e+000
8	1.75	0.03216	1.13	3.22	0.000	0.00e+000	2.92e-002	0.00e+000	0.00e+000
9	2.00	0.03707	1.12	3.71	0.000	0.00e+000	1.97e-002	0.00e+000	0.00e+000
10	4.00	0.1009	0.980	10.1	5.416	9.12e-007	3.19e-002	7.86e-005	0.00e+000
11	8.00	0.1601	0.850	16.0	3.293	1.31e-006	1.48e-002	5.22e-005	0.00e+000
12	16.0	0.2129	0.734	21.3	2.039	1.85e-006	6.60e-003	3.29e-005	0.00e+000
13	32.0	0.2587	0.633	25.9	1.329	2.50e-006	2.86e-003	1.93e-005	0.00e+000
14	8.00	0.2512	0.649	25.1	0.495	6.39e-006	3.12e-004	5.37e-006	0.00e+000
15	2.00	0.2370	0.681	23.7	0.000	0.00e+000	2.37e-003	0.00e+000	0.00e+000
16	0.500	0.2198	0.718	22.0	0.000	0.00e+000	1.14e-002	0.00e+000	0.00e+000

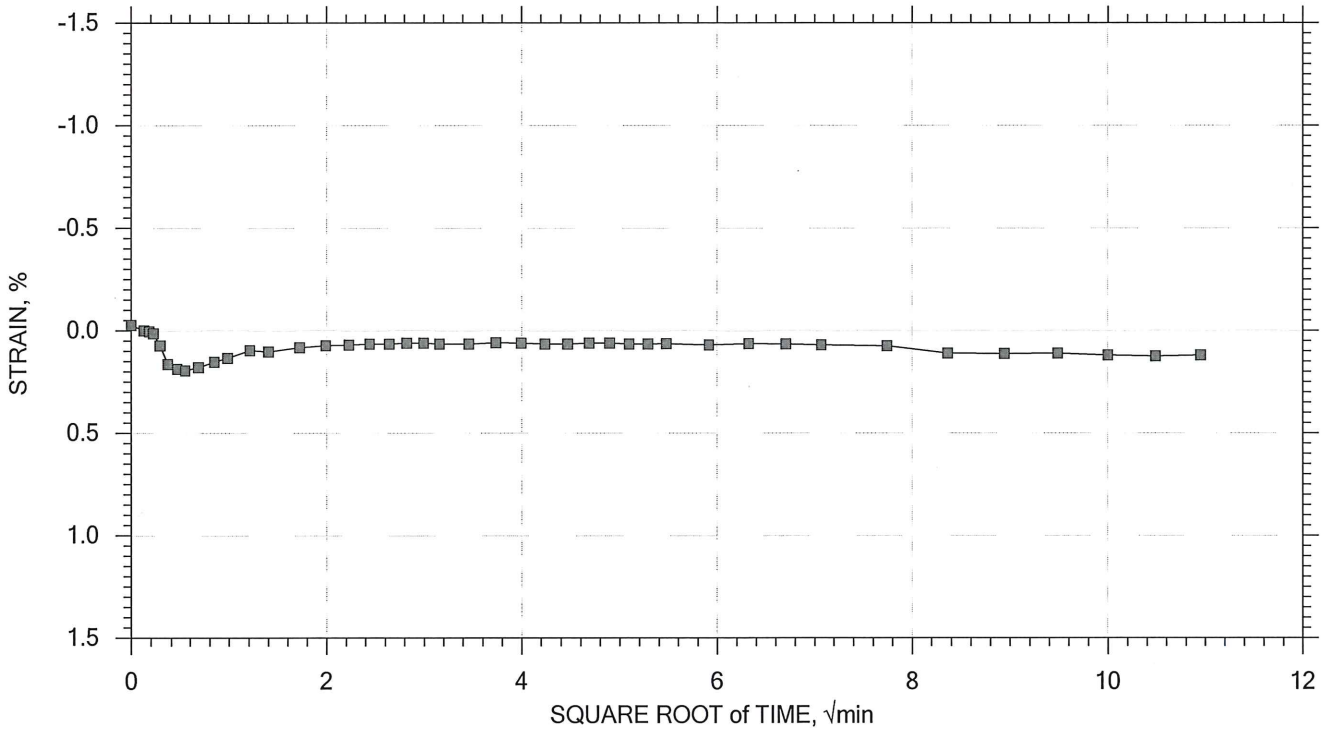
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 1 of 16

Stress: 0.125 tsf





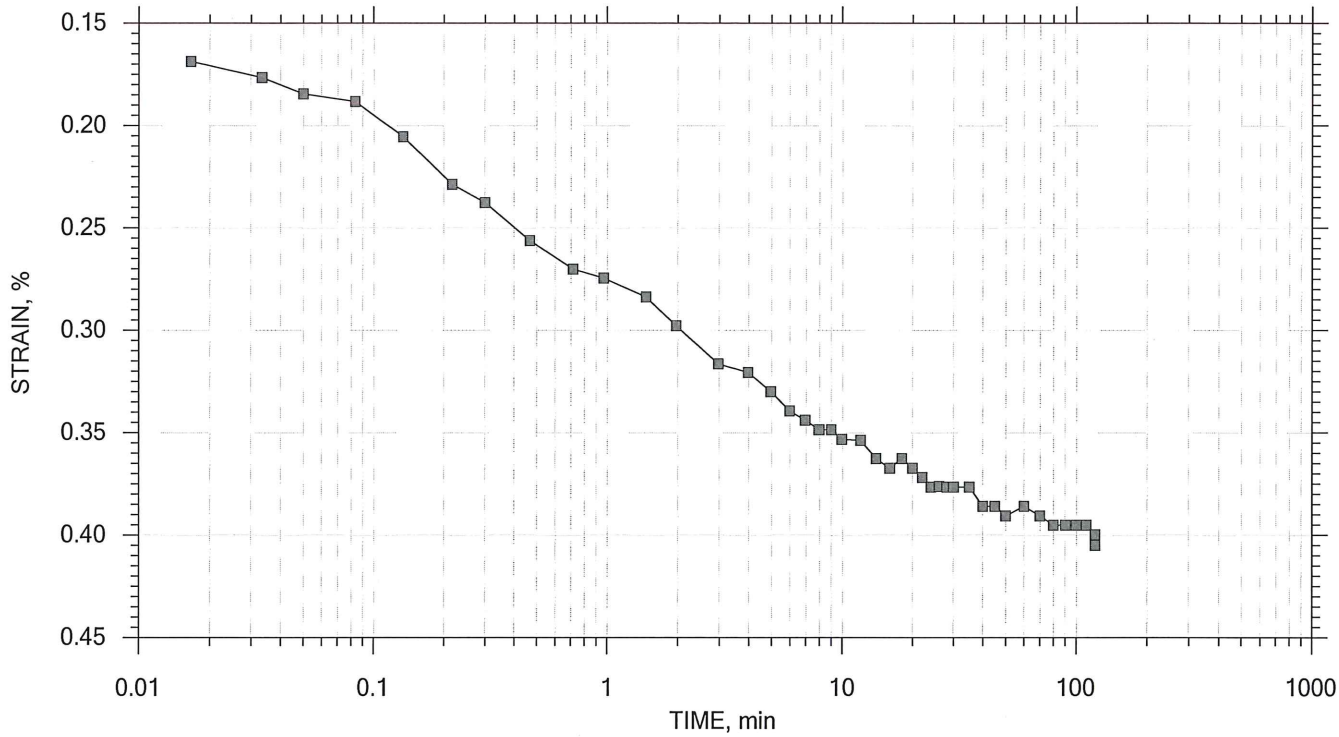
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

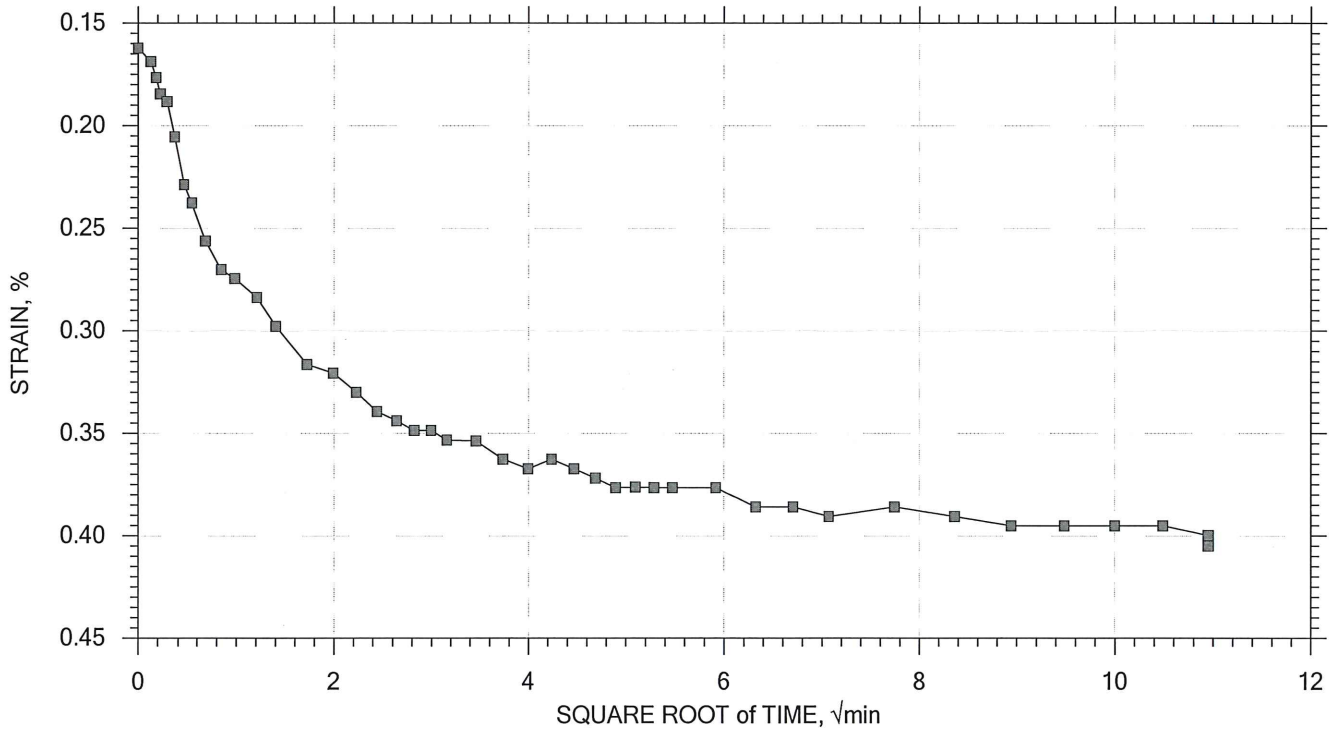
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 2 of 16

Stress: 0.25 tsf





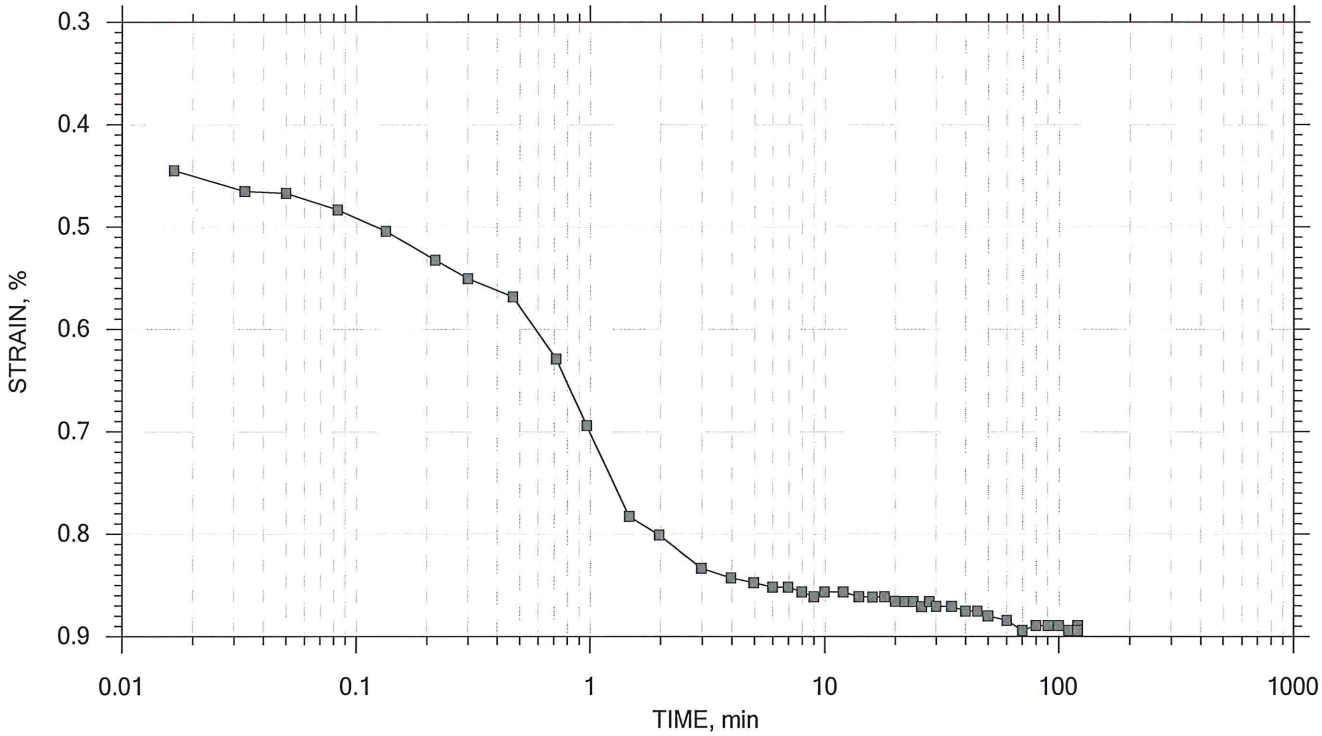
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

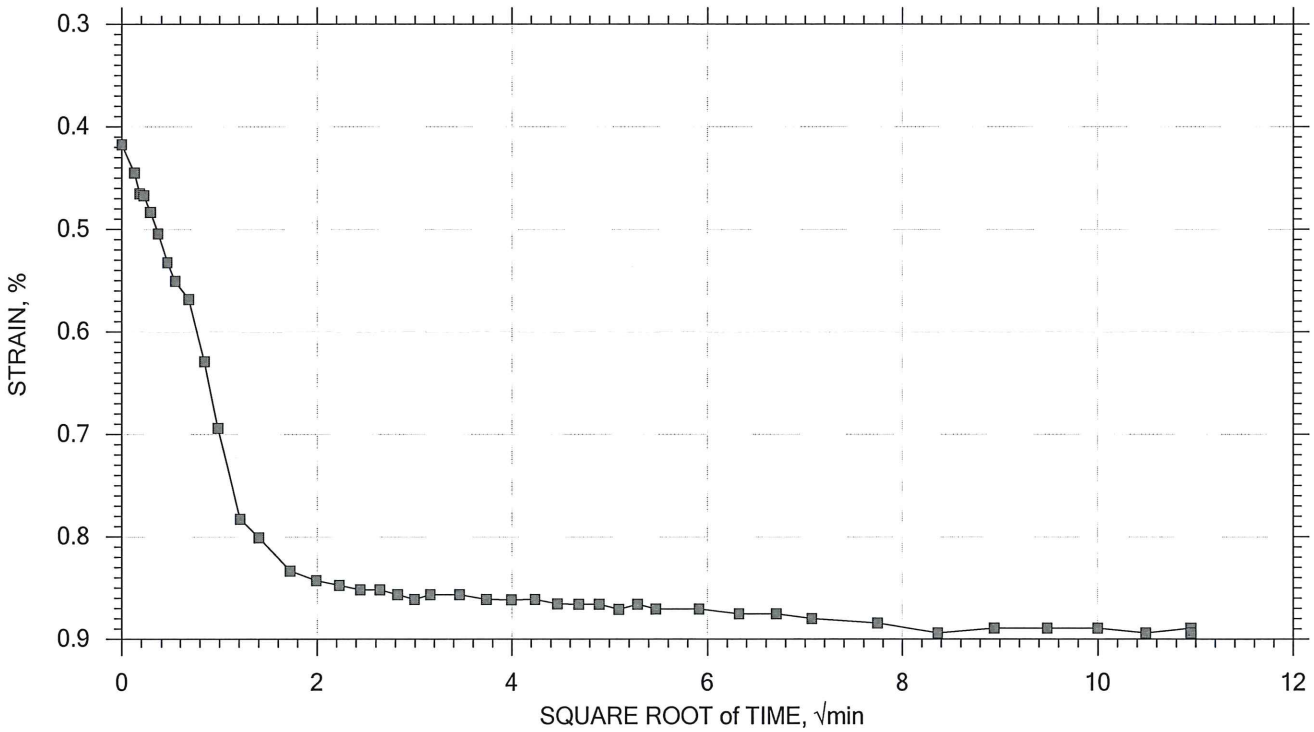
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 3 of 16

Stress: 0.5 tsf





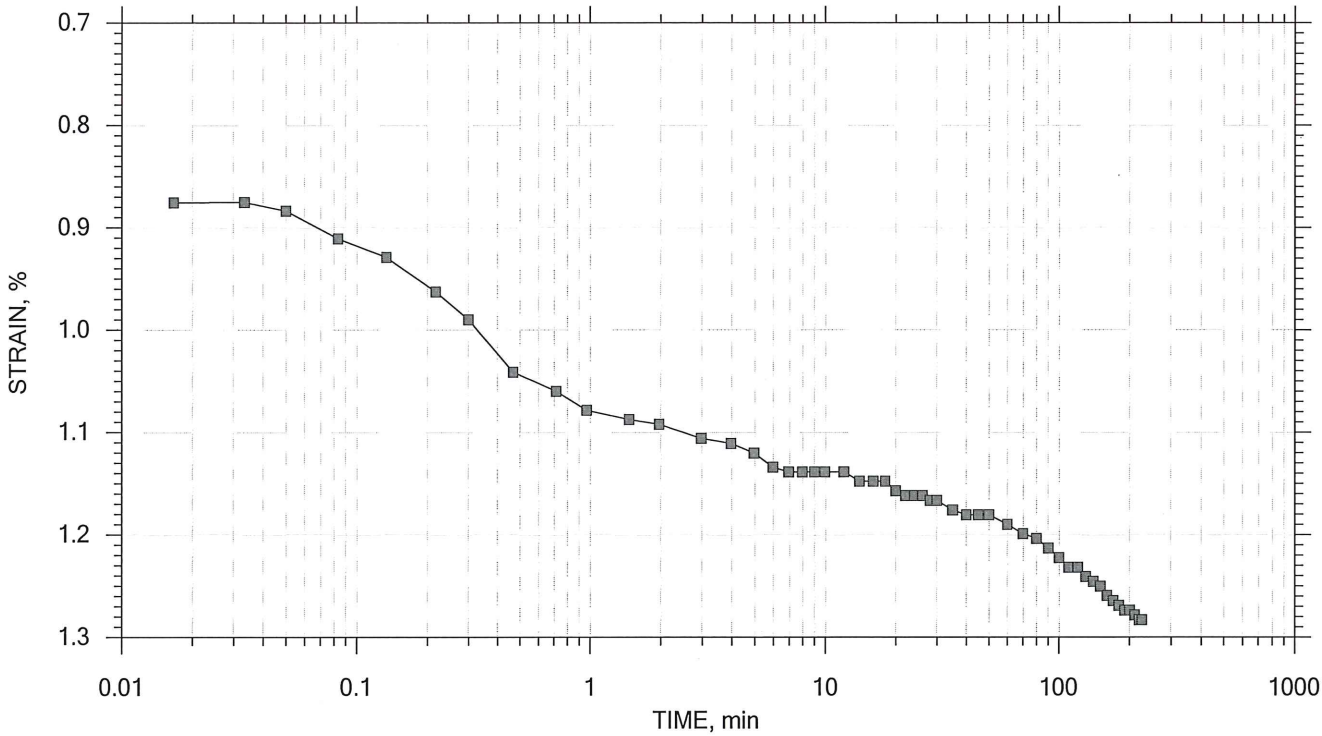
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

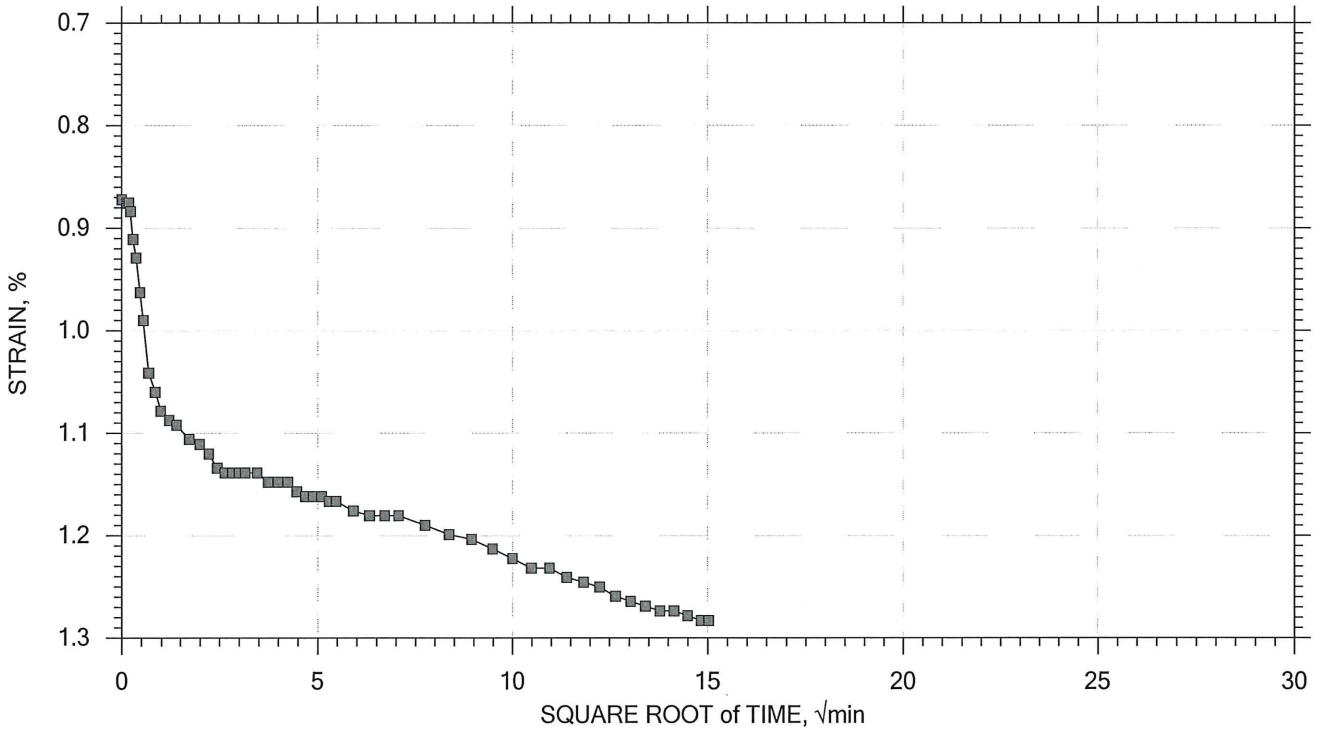
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 4 of 16

Stress: 0.75 tsf





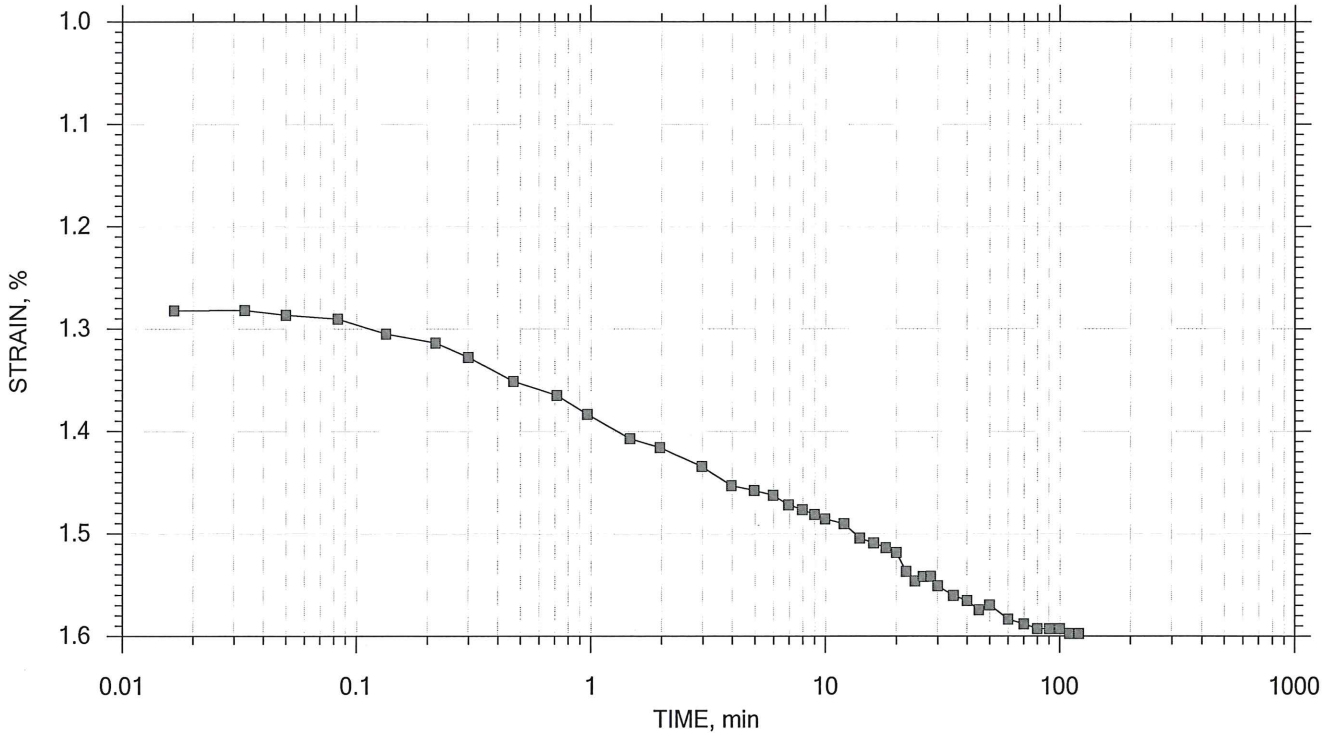
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

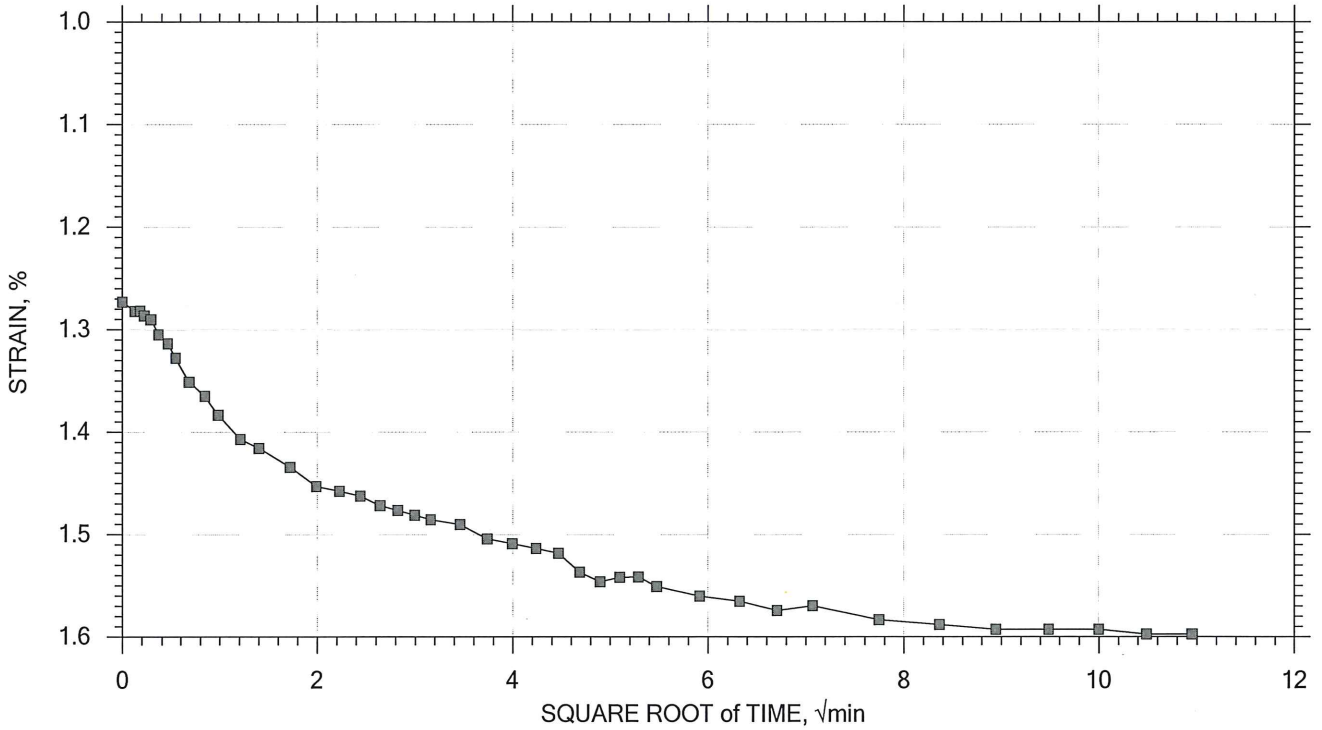
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 5 of 16

Stress: 1 tsf





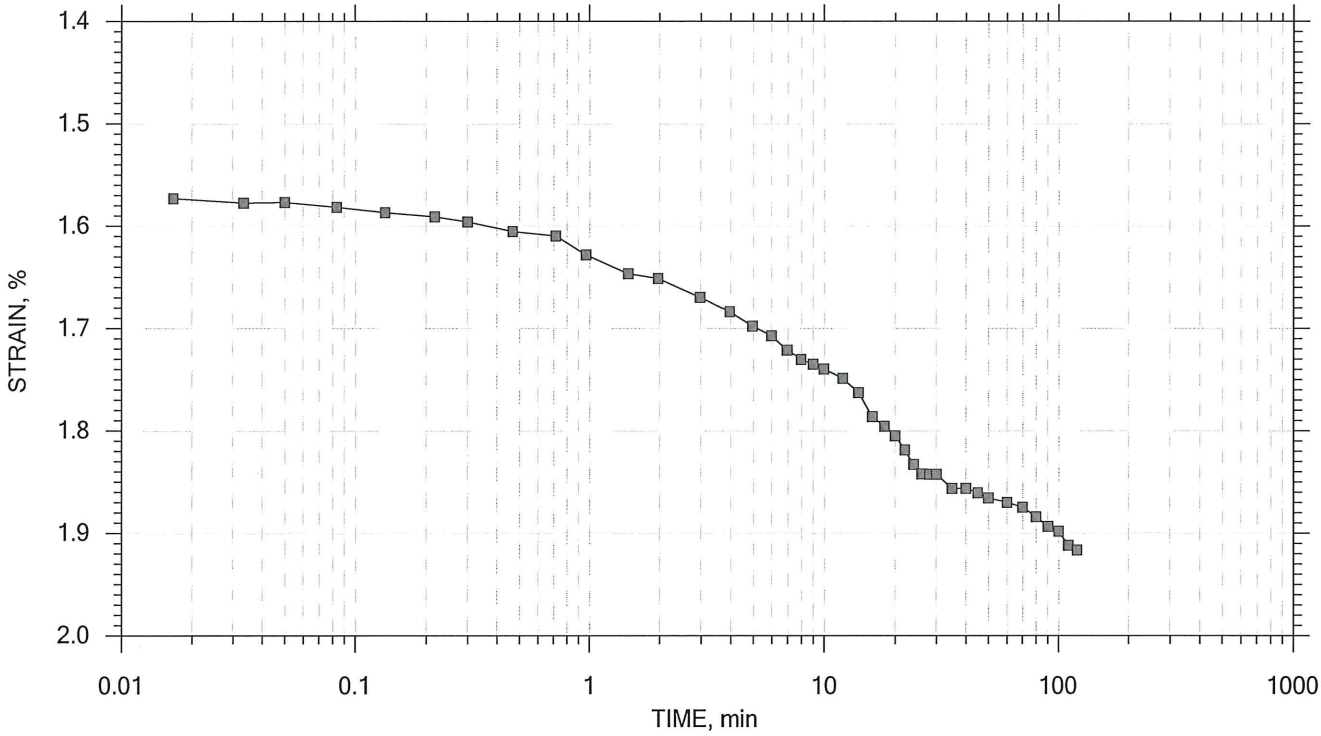
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

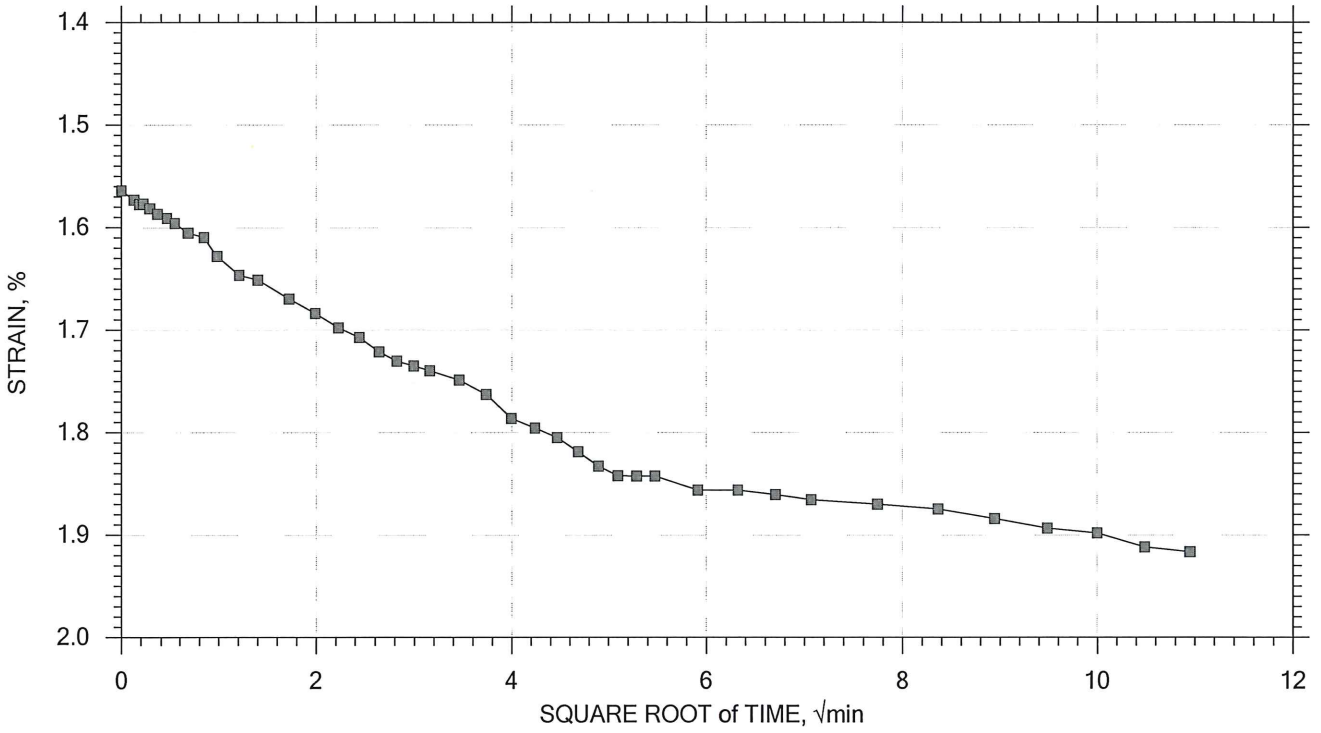
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 6 of 16

Stress: 1.25 tsf





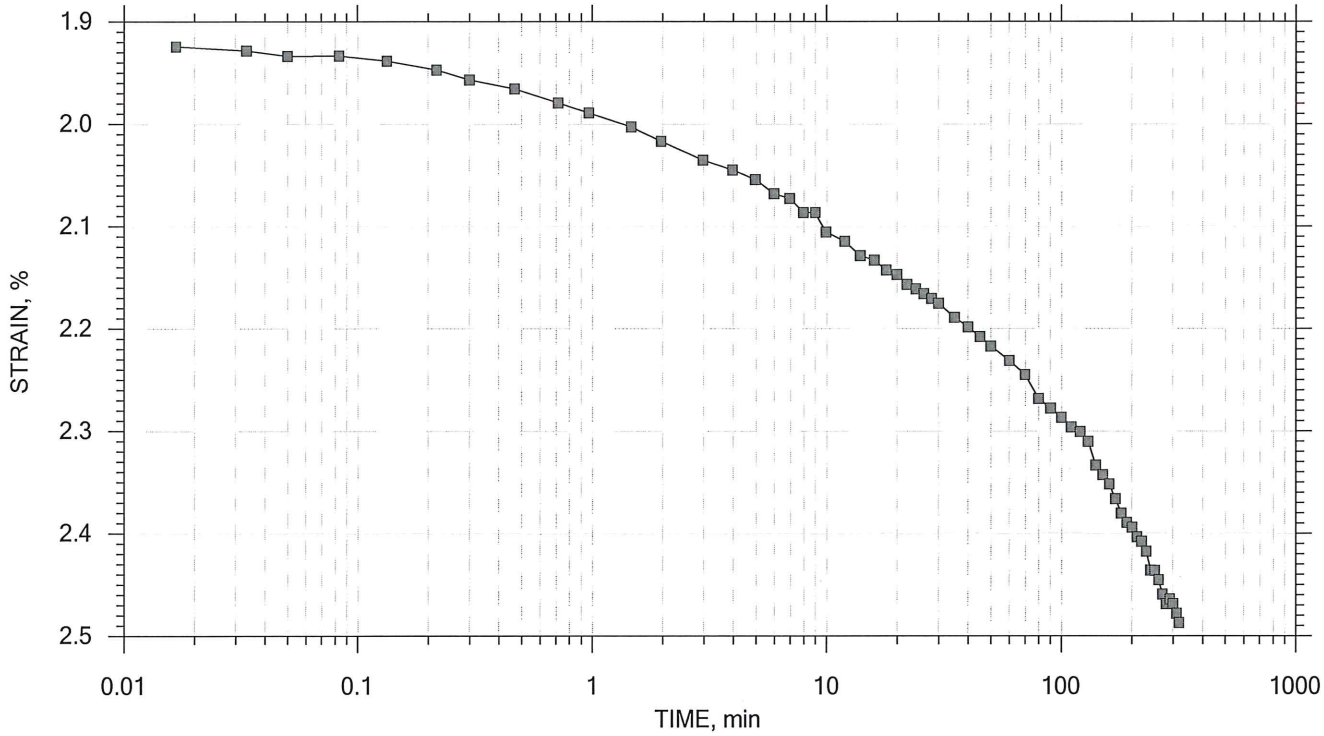
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

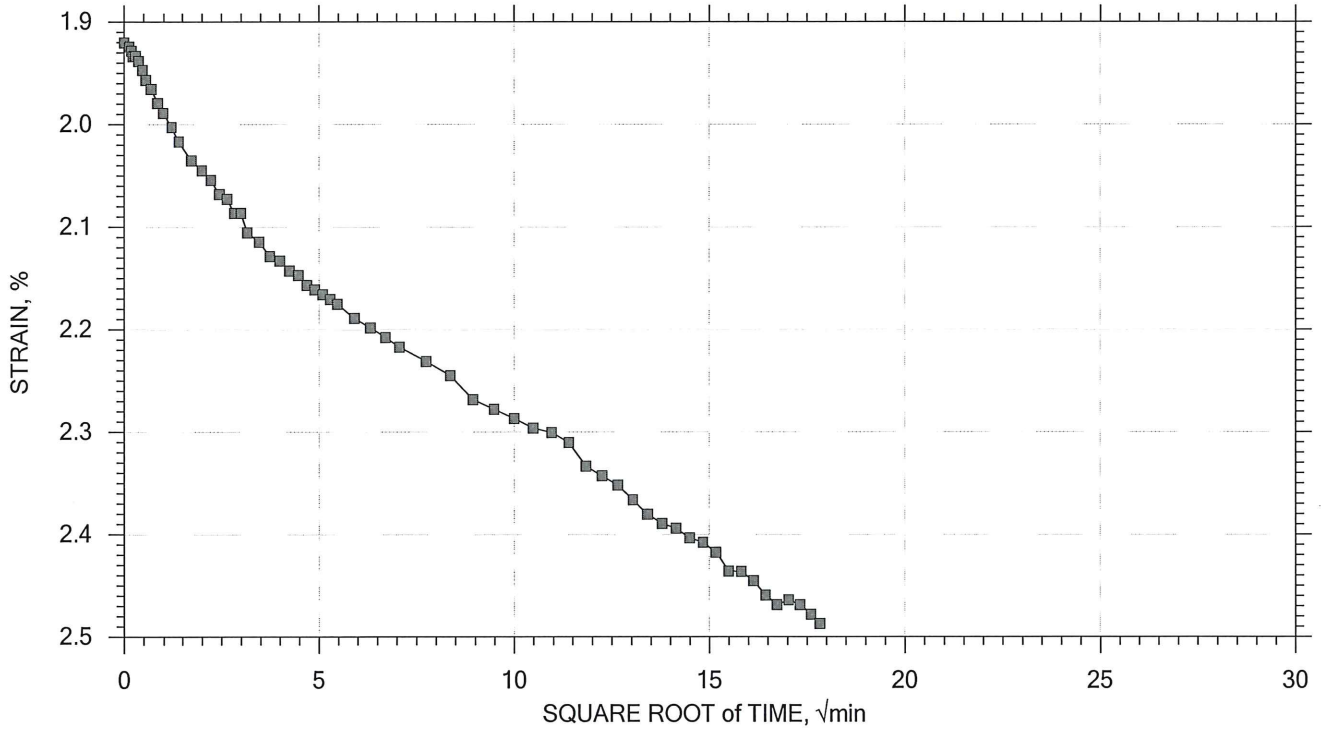
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 7 of 16

Stress: 1.5 tsf





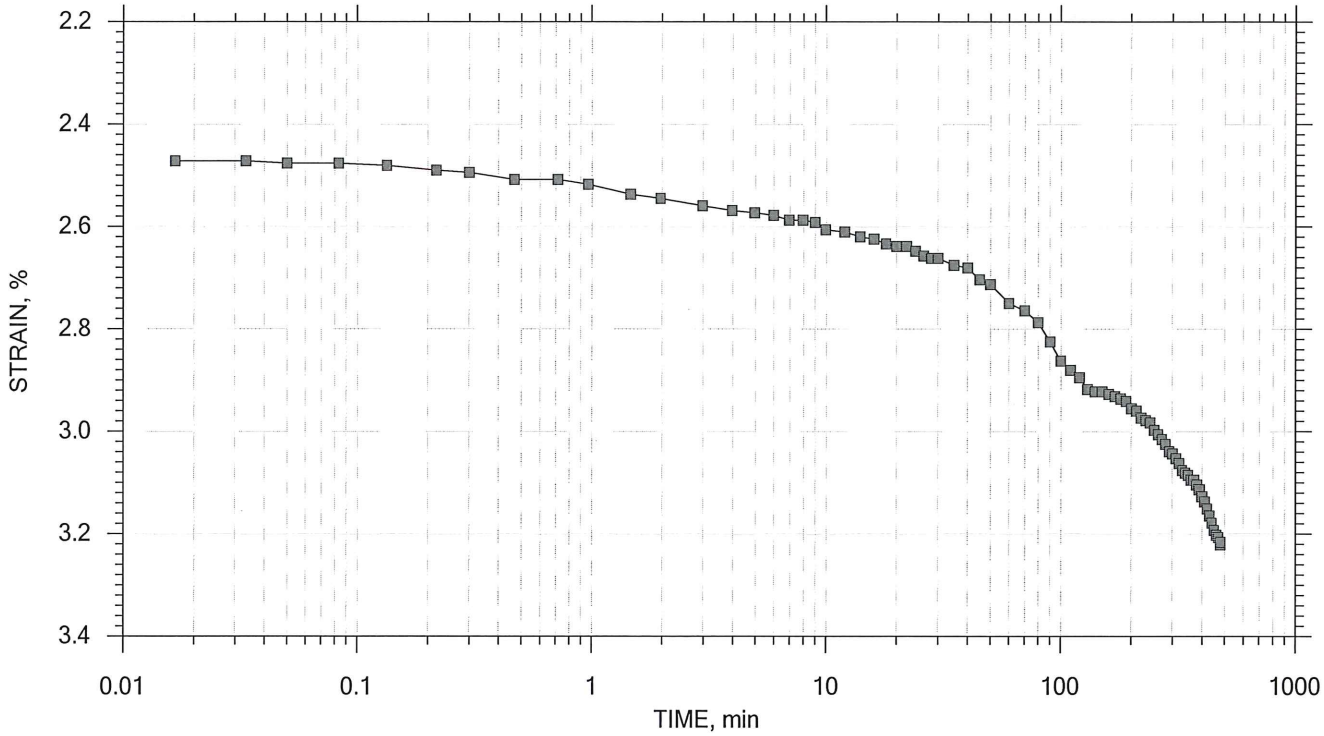
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

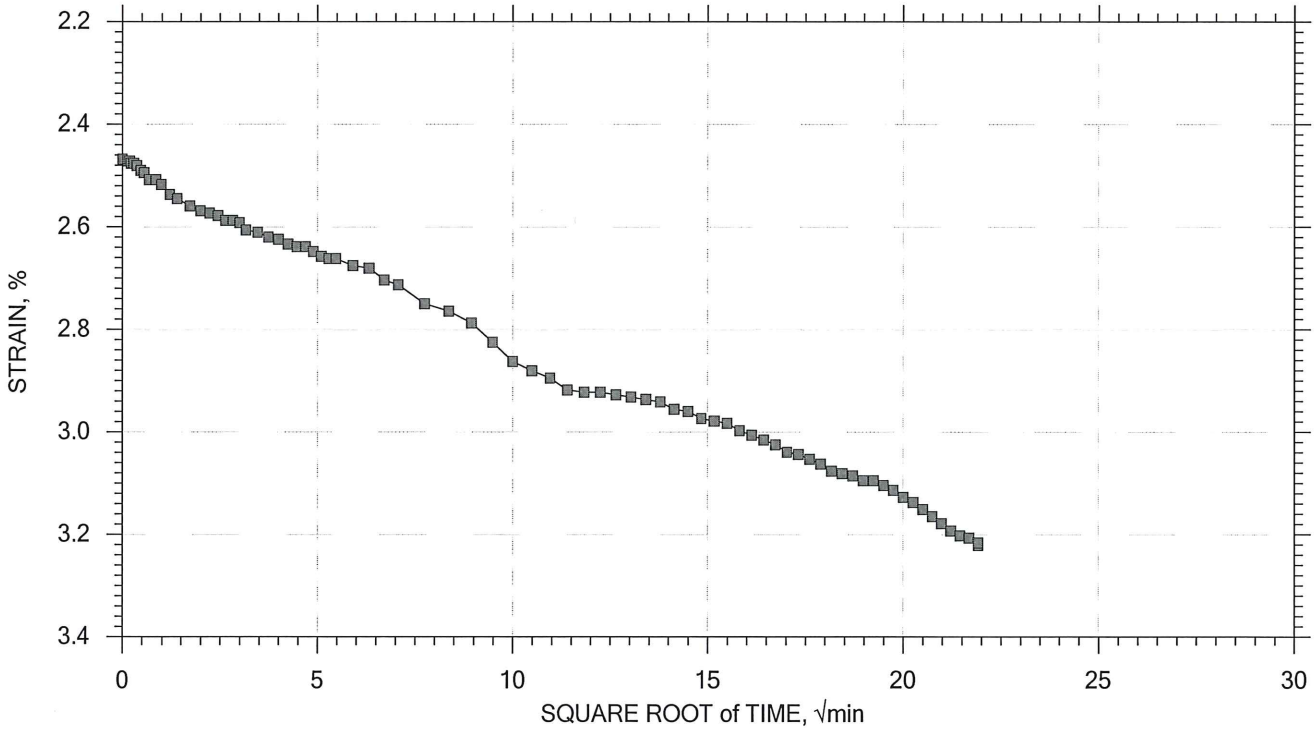
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 8 of 16

Stress: 1.75 tsf





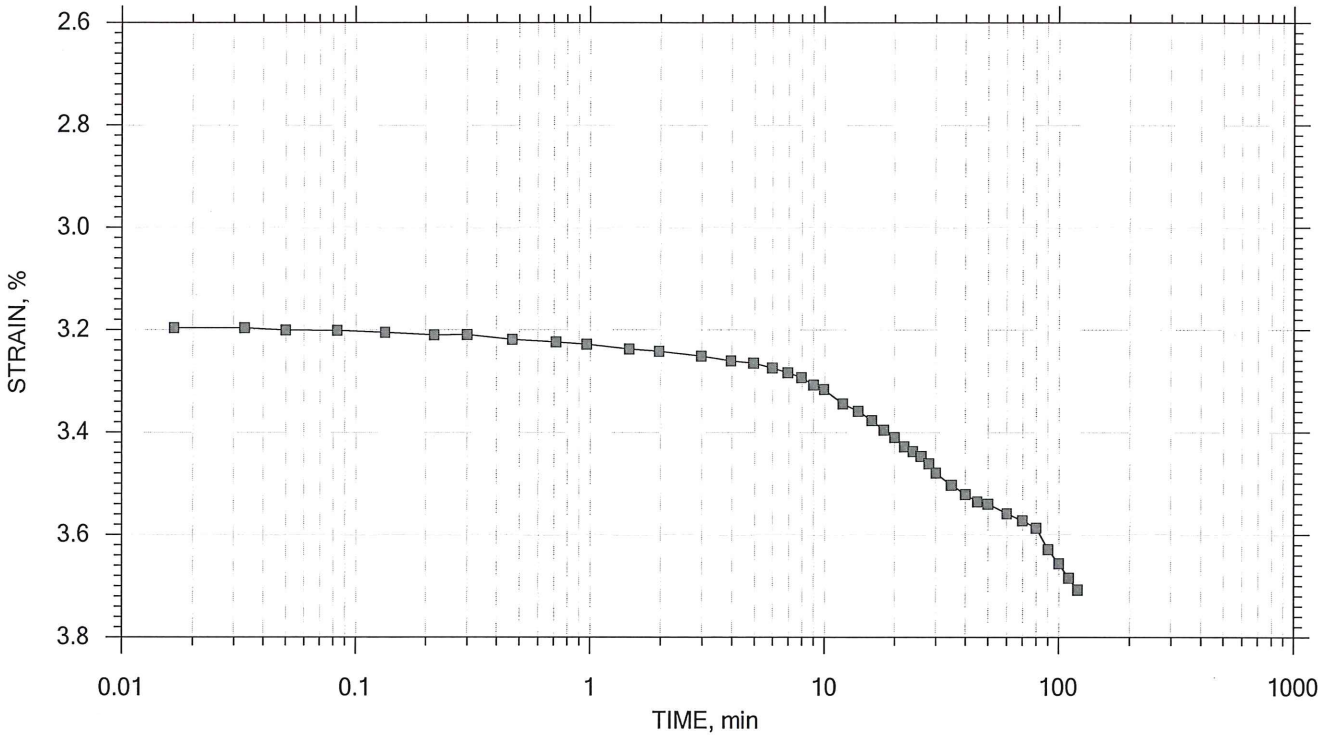
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

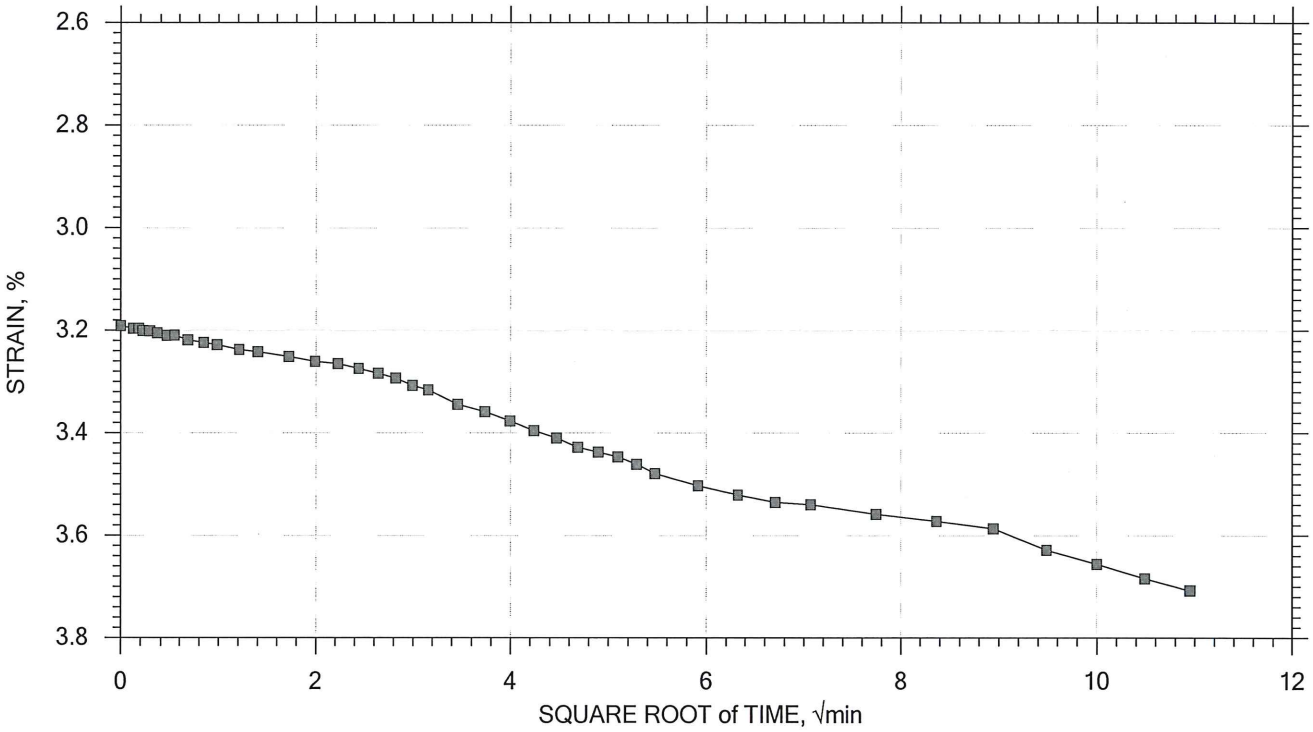
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 9 of 16

Stress: 2 tsf





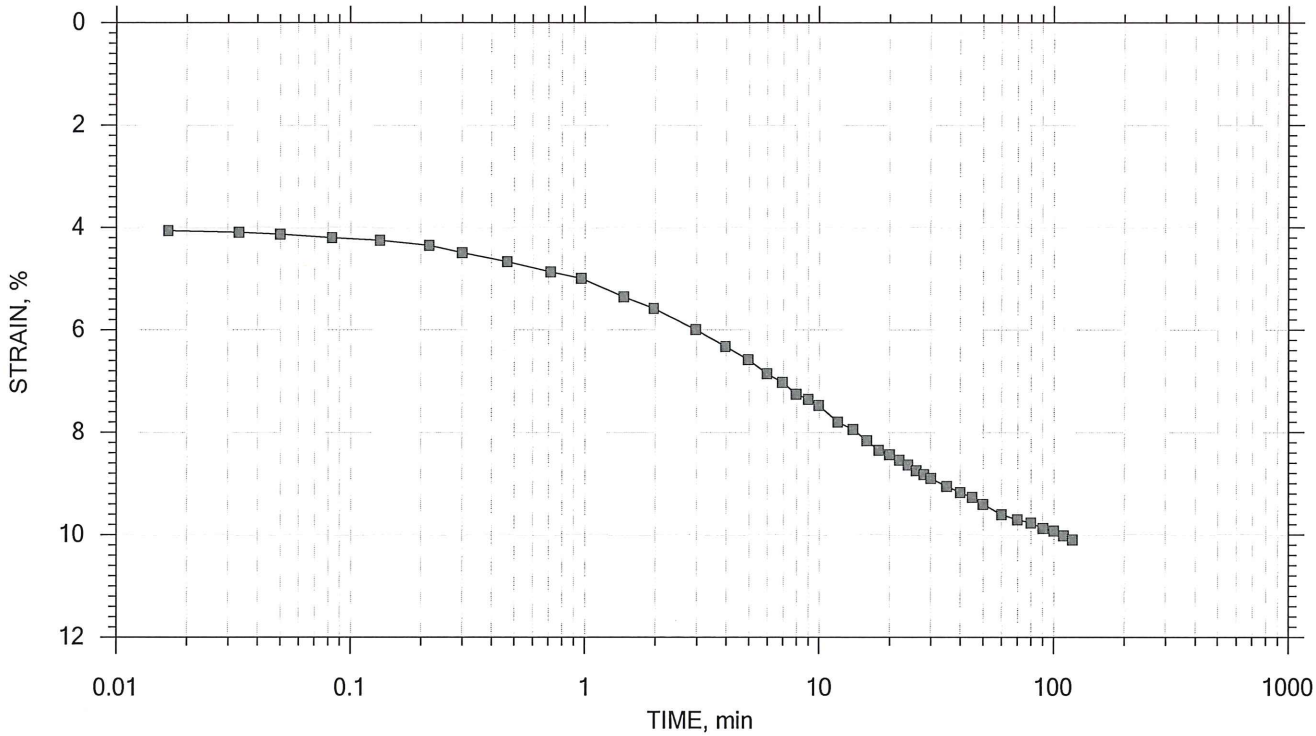
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

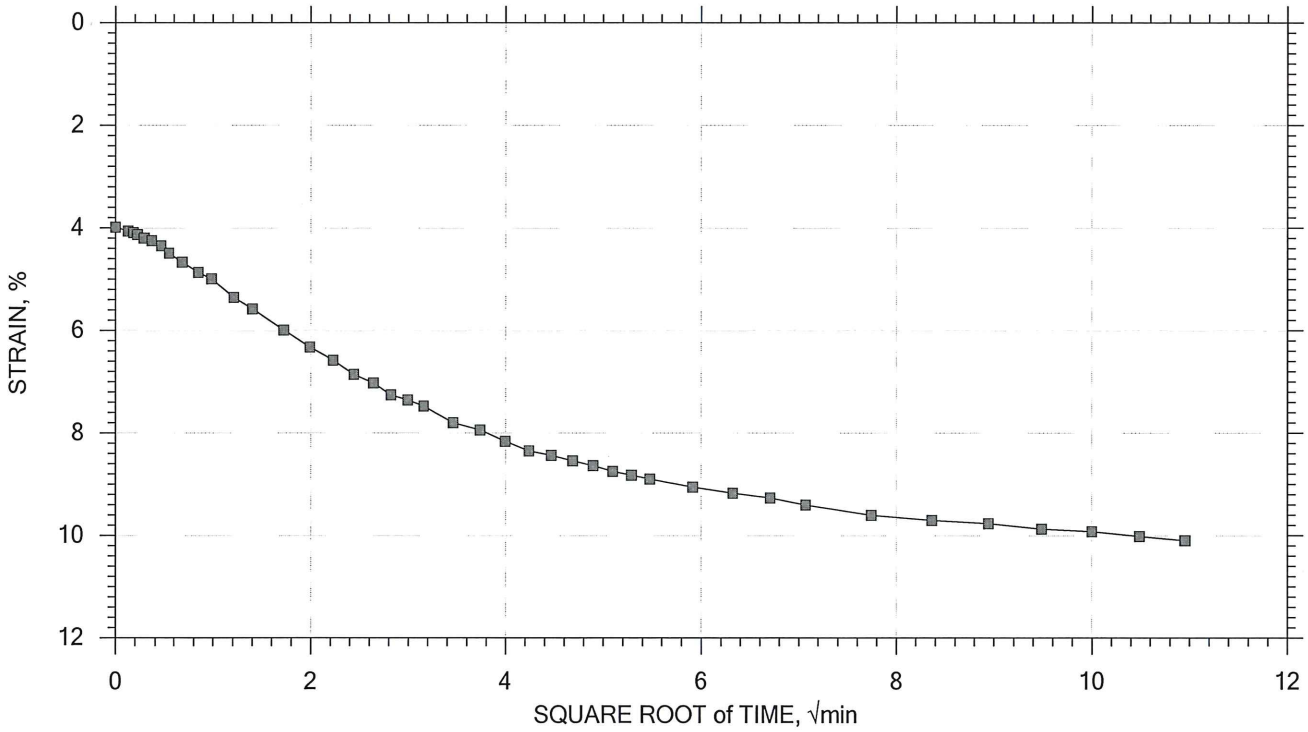
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 10 of 16

Stress: 4 tsf





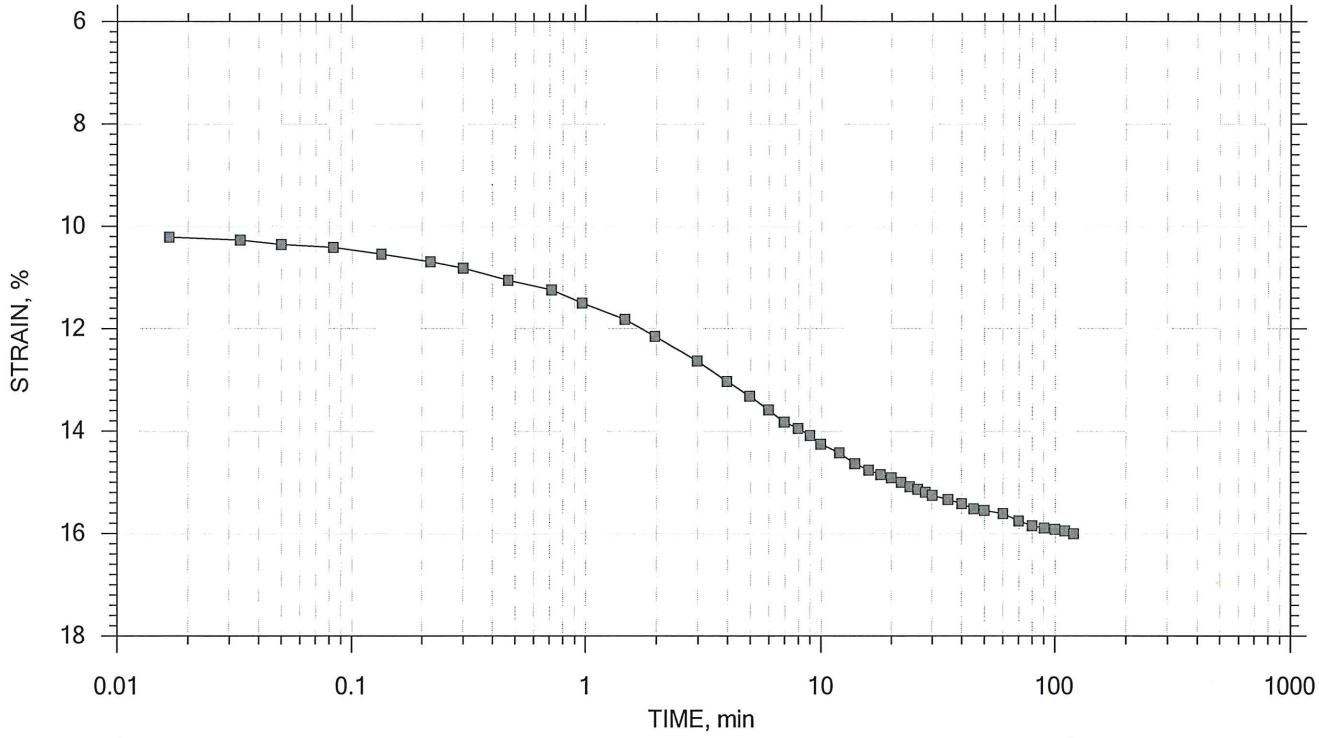
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

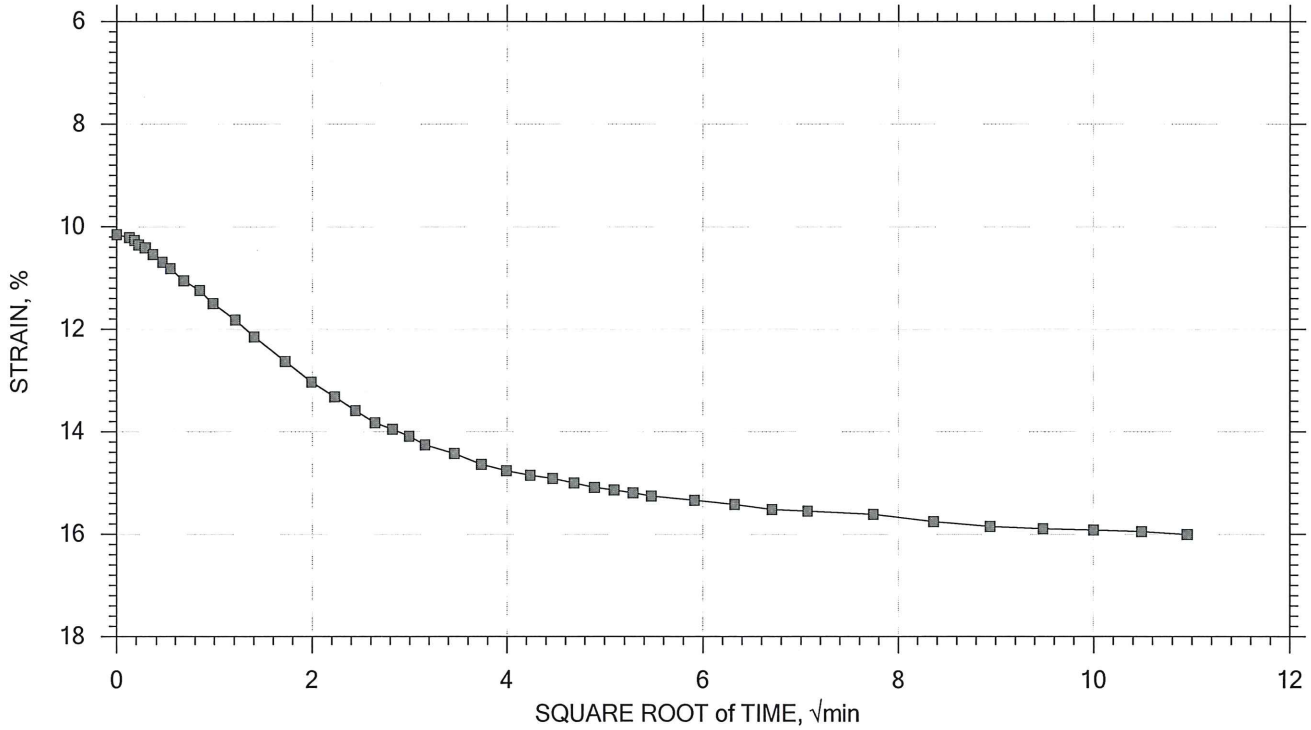
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 11 of 16

Stress: 8 tsf





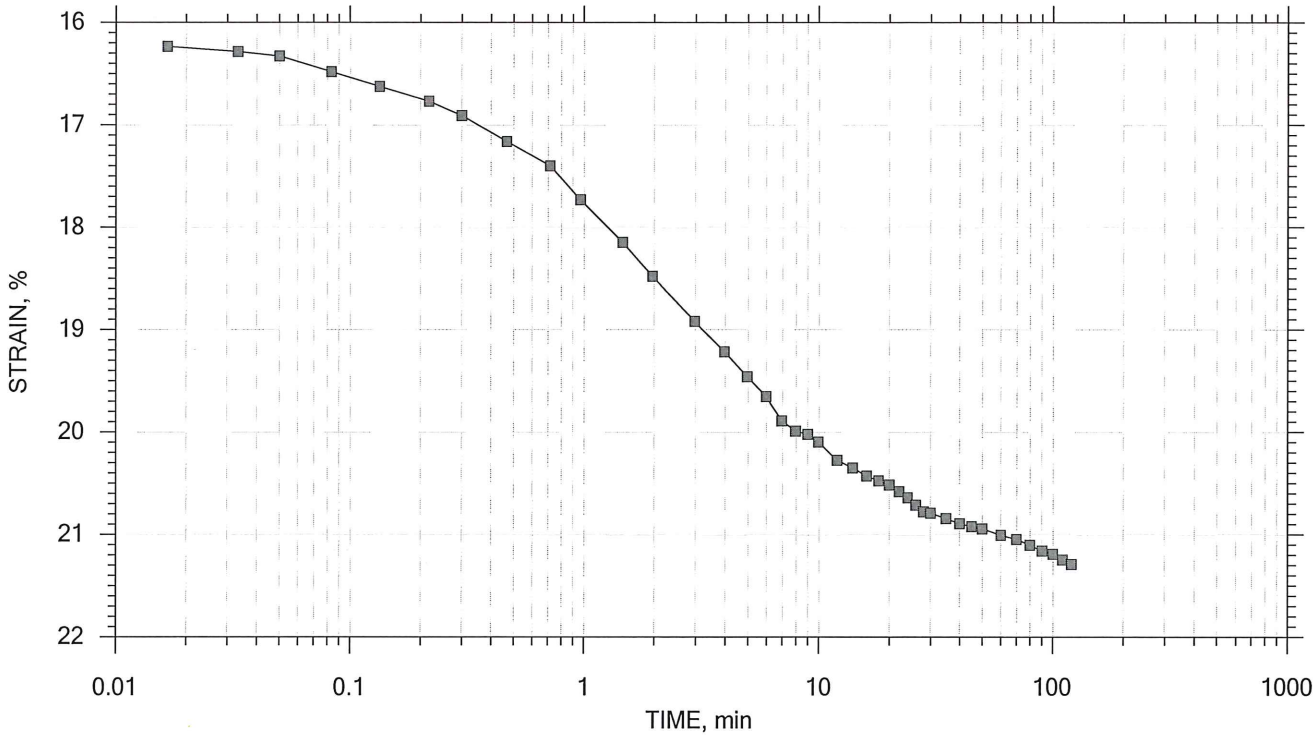
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

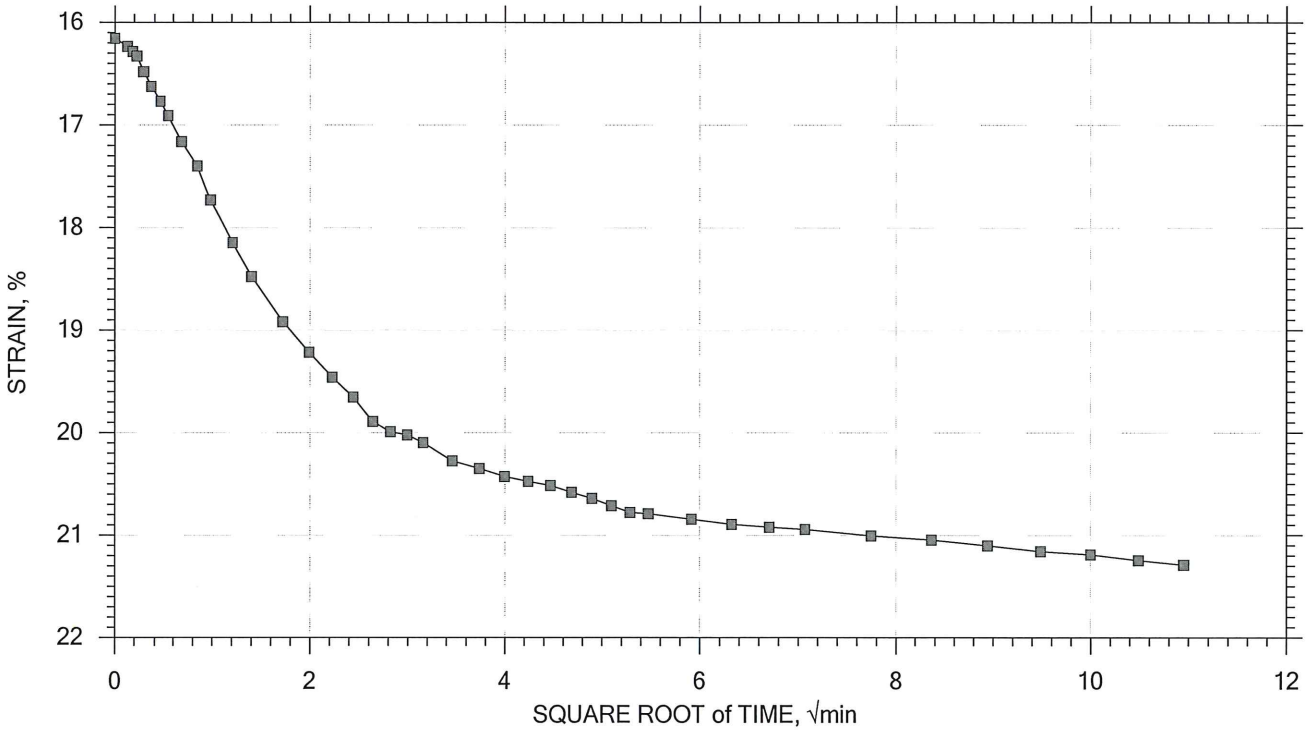
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 12 of 16

Stress: 16 tsf





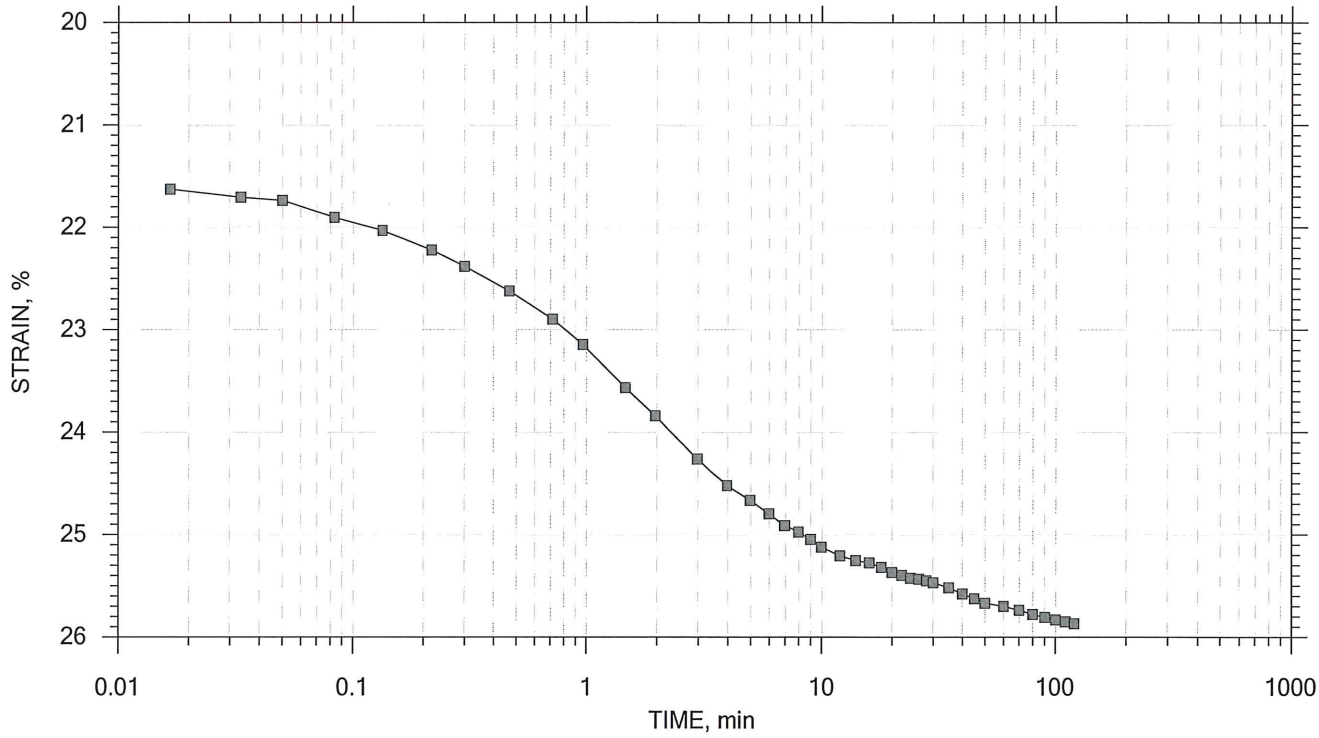
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

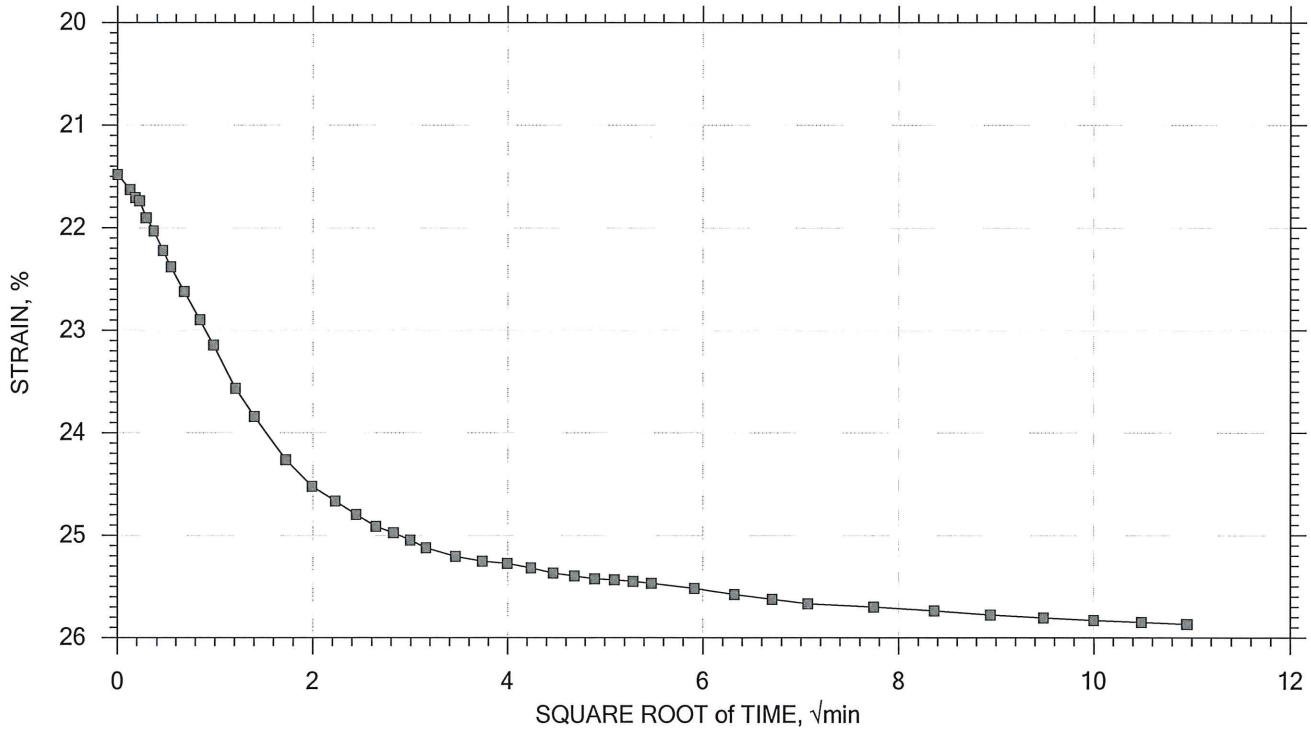
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 13 of 16

Stress: 32 tsf





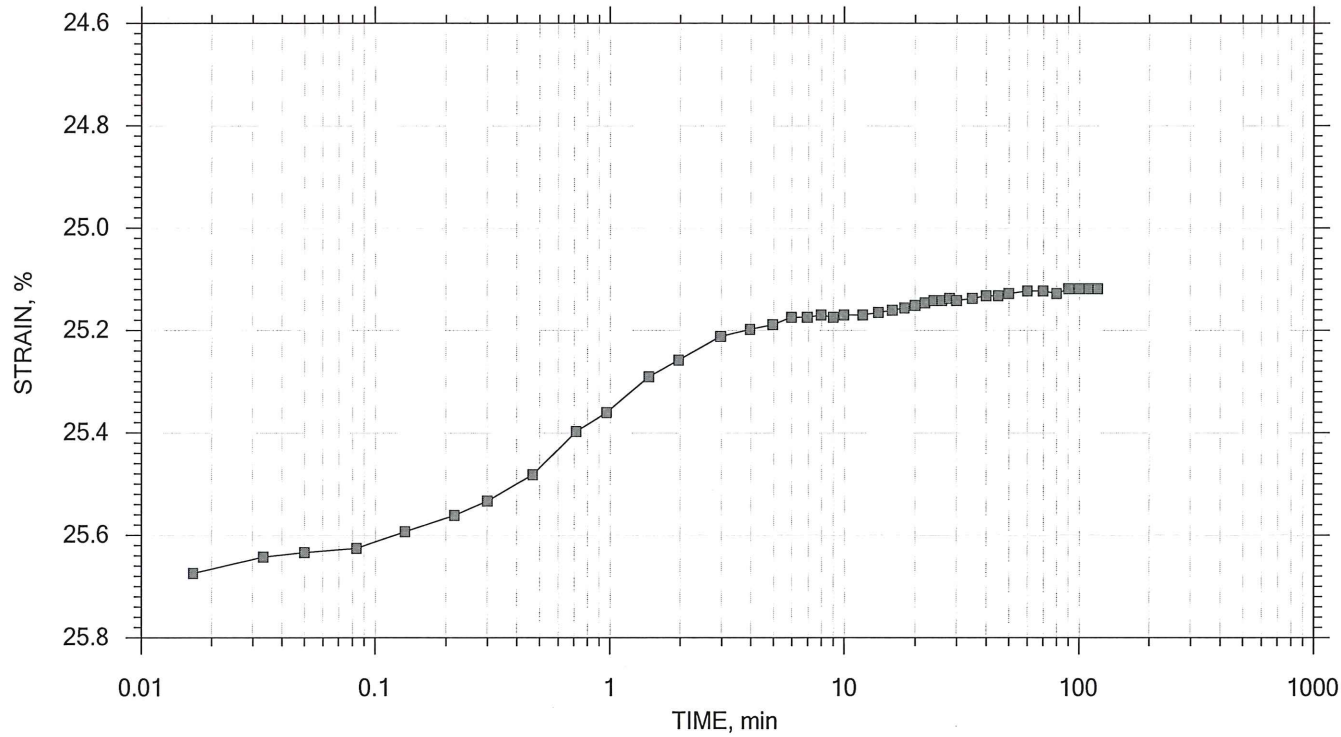
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

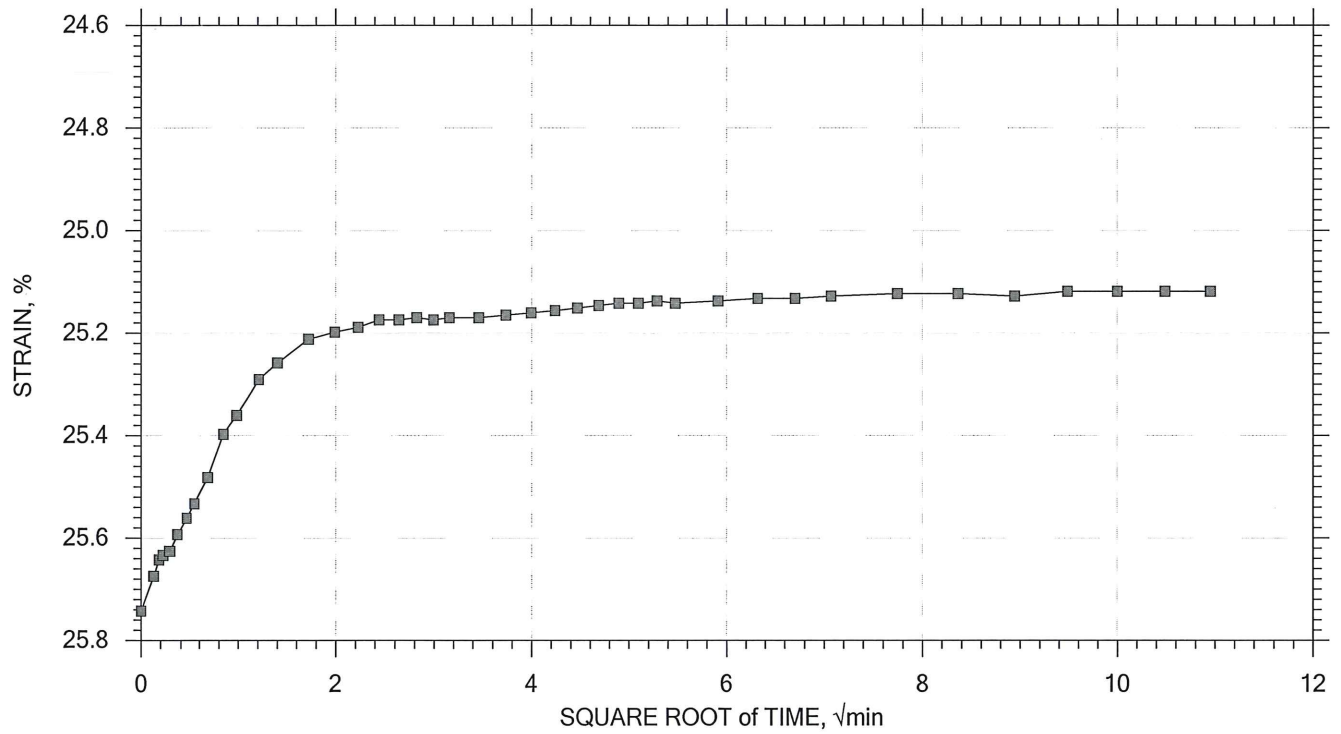
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 14 of 16

Stress: 8 tsf





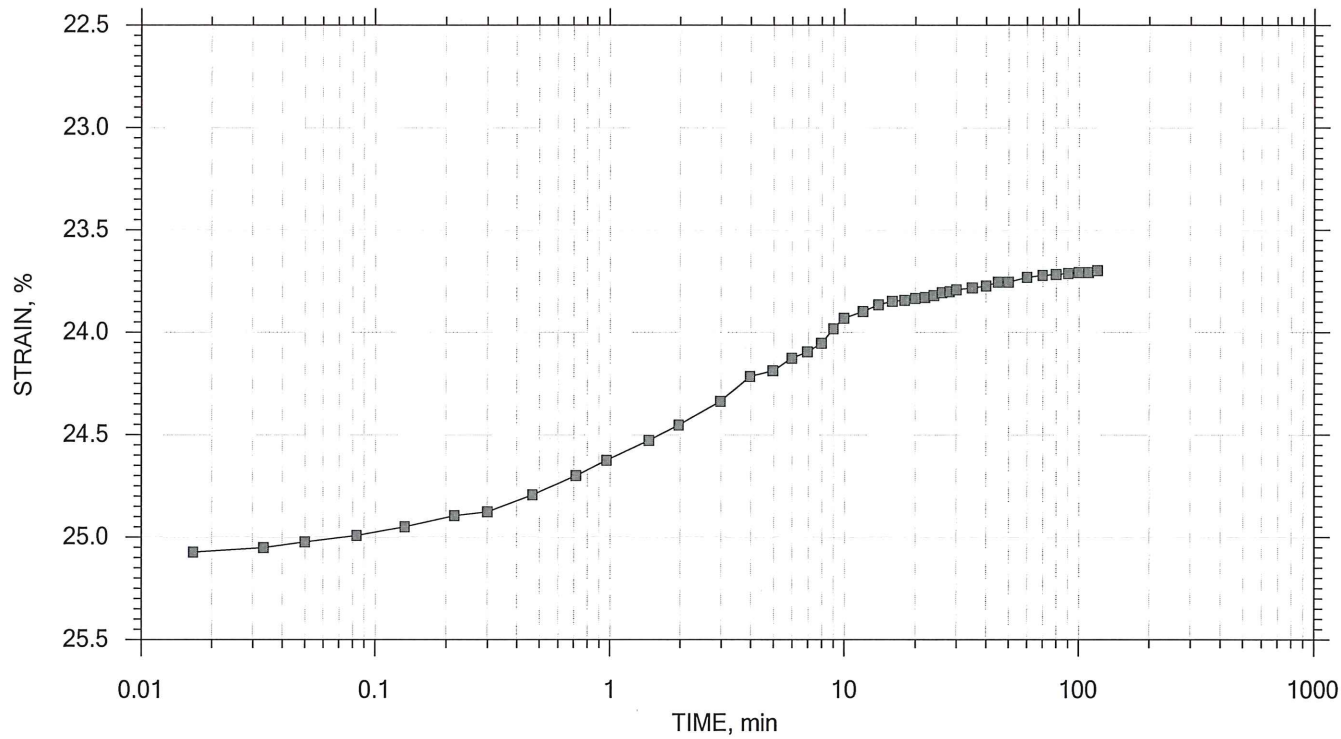
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

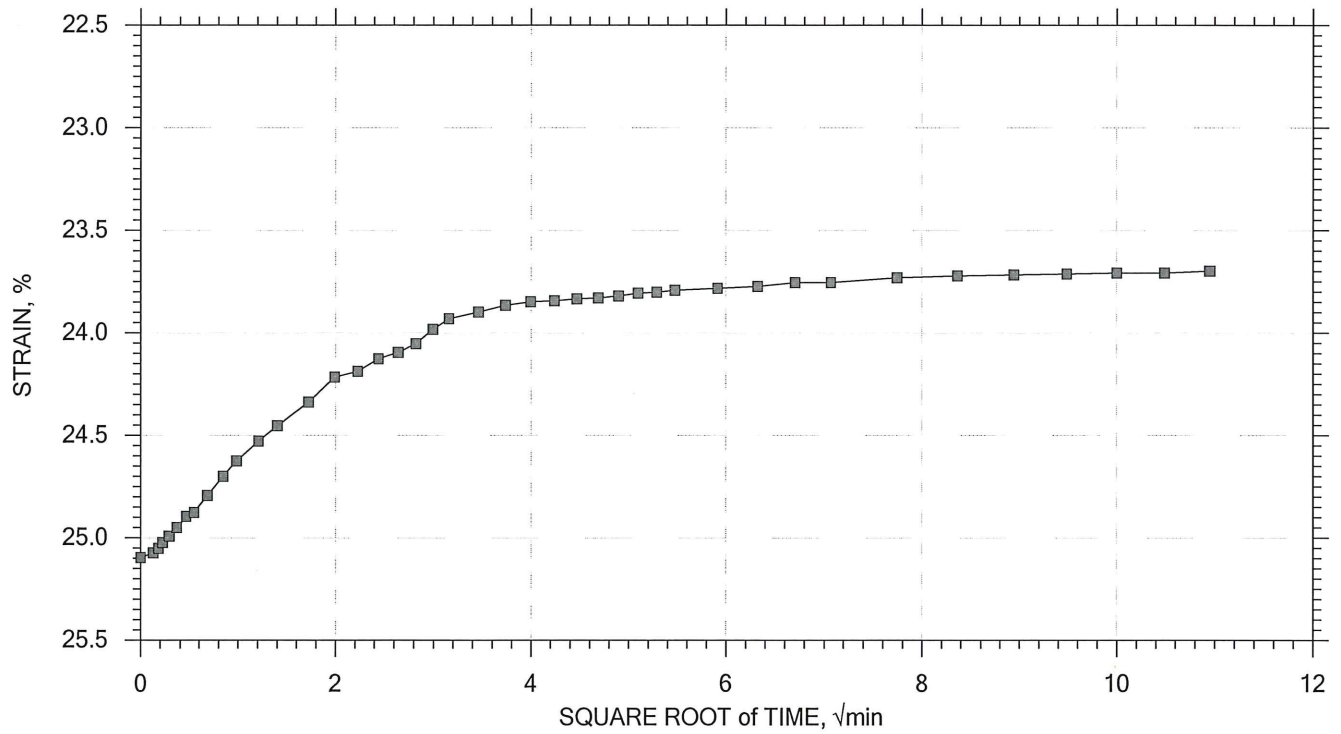
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 15 of 16

Stress: 2 tsf





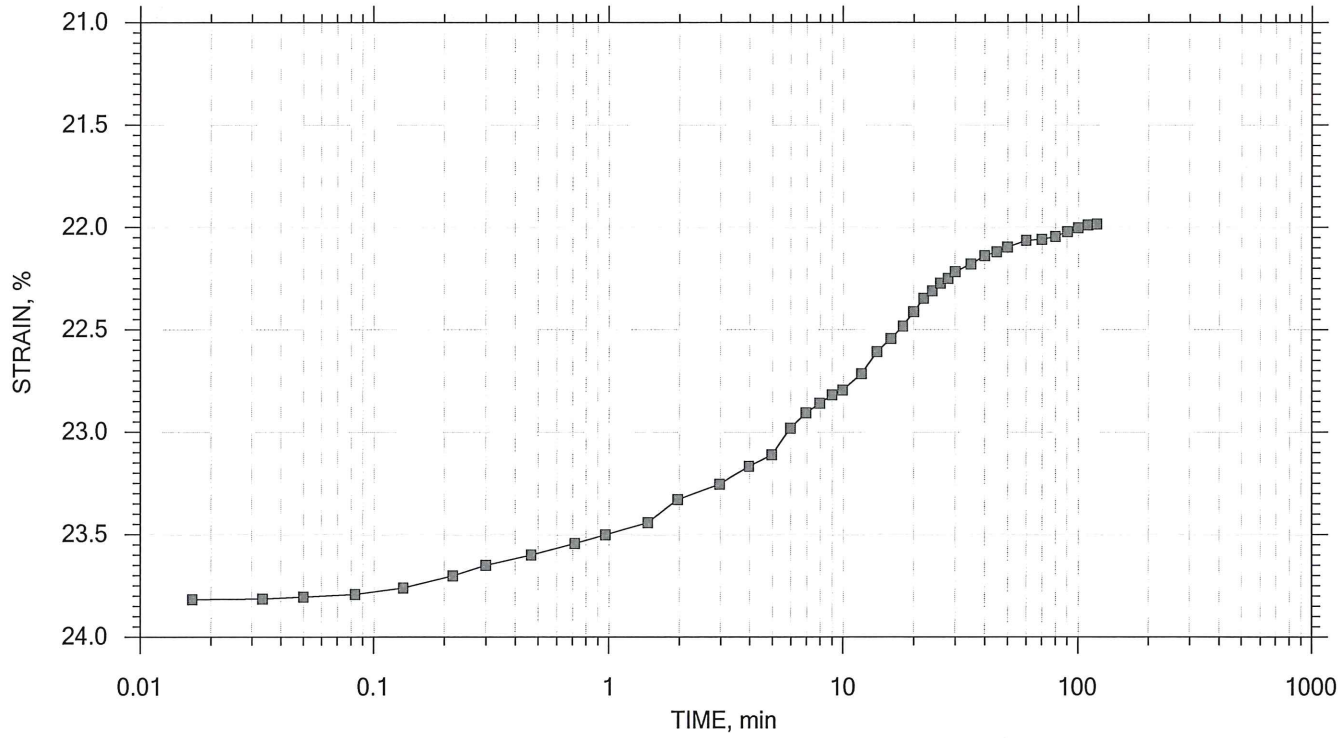
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

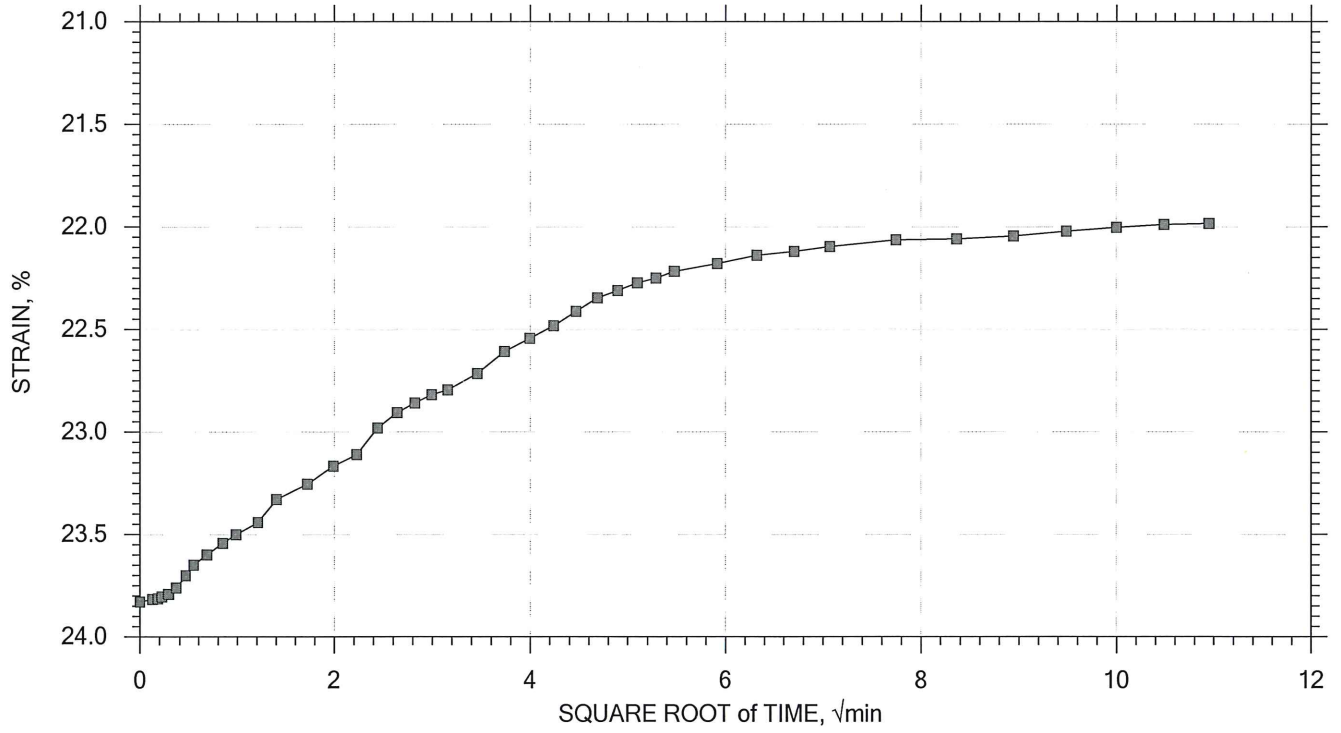
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 16 of 16

Stress: 0.5 tsf

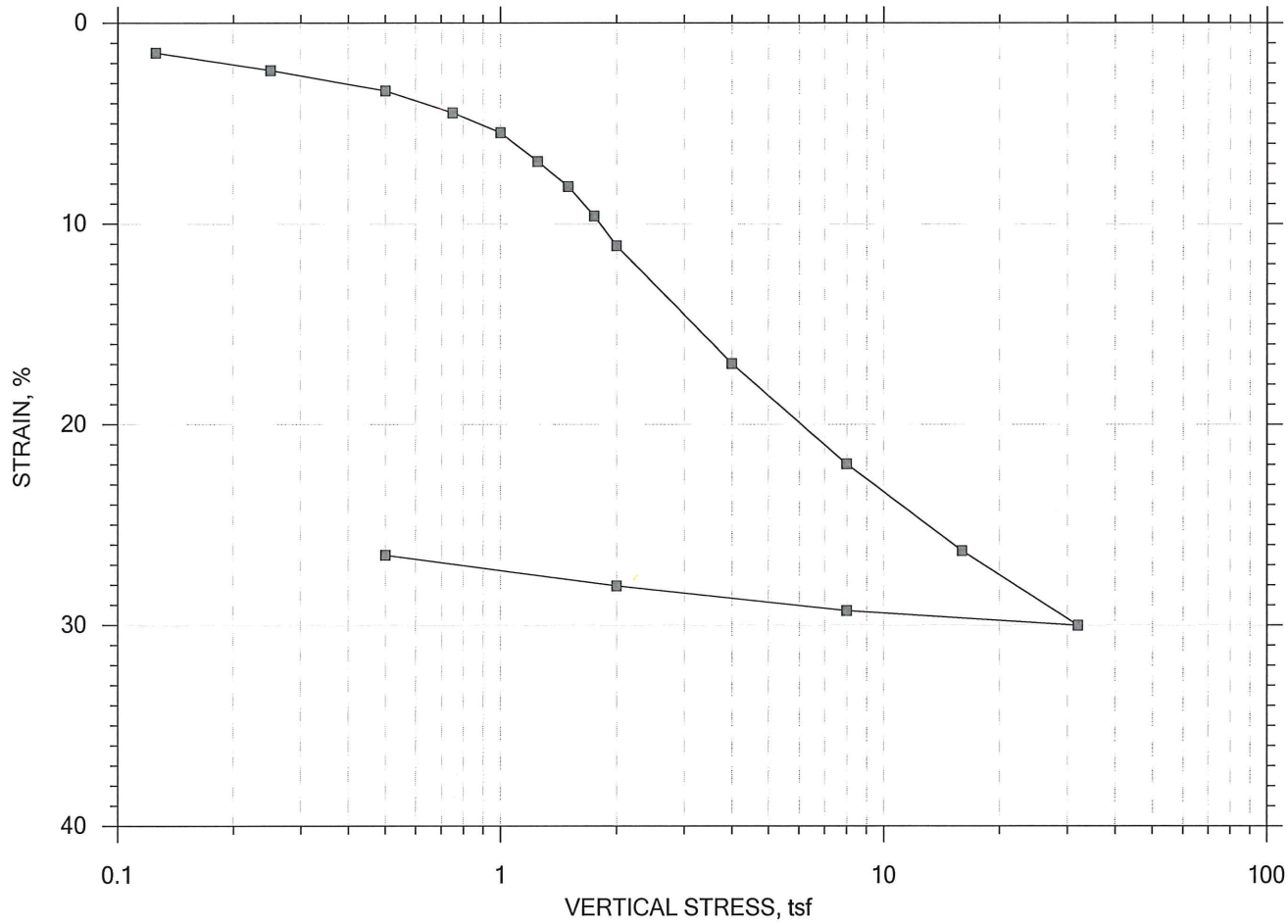


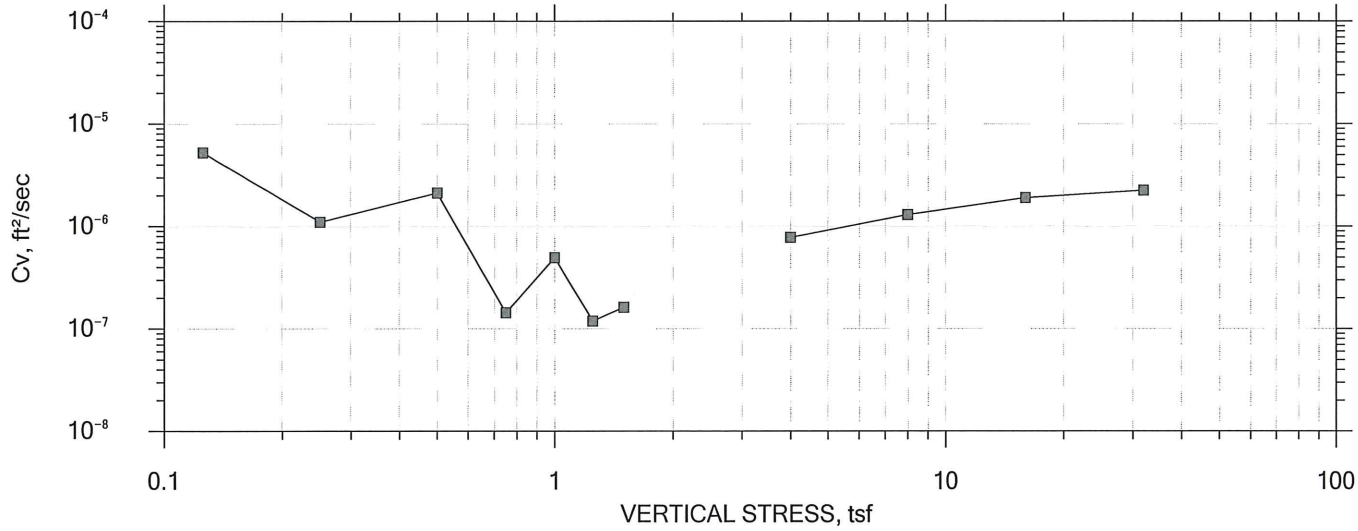


GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U1	Test Date: 04/02/13	Test No.: IP-1
	Depth: 21.0-23.0 ft	Sample Type: remolded	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System X		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

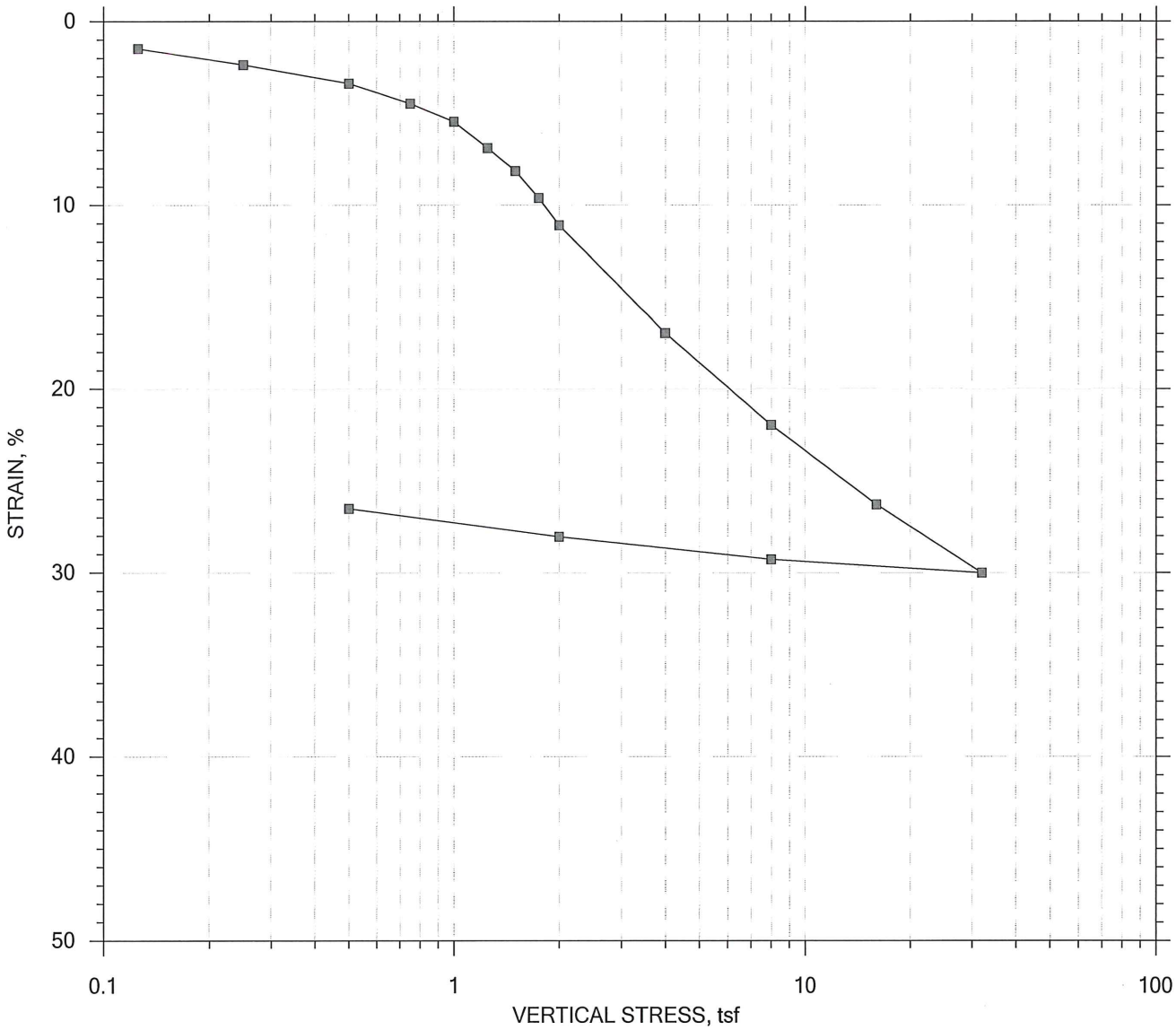




GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		
	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, %	47.46	24.82
Preconsolidation Stress: ---				Dry Unit Weight, pcf	74.249	103.12
Compression Ratio: ---				Saturation, %	98.13	100.00
Diameter: 2.5 in		Height: 1 in		Void Ratio	1.35	0.70
LL: 38	PL: 19	PI: 19	GS: 2.80			

GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: U.S. Rt.1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U2
Test No.: IP-3

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: intact

Project No.: GTX-300397
Checked By: jdt
Depth: 30.0-32.0 ft
Elevation: ---

Soil Description: Wet, gray clay
Remarks: System Y

Estimated Specific Gravity: 2.80
Initial Void Ratio: 1.35
Final Void Ratio: 0.695

Liquid Limit: 38
Plastic Limit: 19
Plasticity Index: 19

Specimen Diameter: 2.50 in
Initial Height: 1.00 in
Final Height: 0.72 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	9431	RING	W	11497
Wt. Container + Wet Soil, gm	171.05	249.06	227.40	127.83
Wt. Container + Dry Soil, gm	119.38	203.65	203.65	103.91
Wt. Container, gm	8.2900	107.98	107.98	7.5500
Wt. Dry Soil, gm	111.09	95.671	95.671	96.360
Water Content, %	46.51	47.46	24.82	24.82
Void Ratio	---	1.35	0.695	---
Degree of Saturation, %	---	98.13	100.00	---
Dry Unit Weight, pcf	---	74.249	103.12	---

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: U.S. Rt.1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U2
Test No.: IP-3

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: intact

Project No.: GTX-300397
Checked By: jdt
Depth: 30.0-32.0 ft
Elevation: ---

Soil Description: Wet, gray clay
Remarks: System Y

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	0.125	0.01482	1.32	1.48	5.825	4.15e-006	1.19e-001	1.33e-003
2	0.250	0.02349	1.30	2.35	24.021	9.83e-007	6.94e-002	1.84e-004
3	0.500	0.03362	1.28	3.36	9.945	2.33e-006	4.05e-002	2.54e-004
4	0.750	0.04459	1.25	4.46	158.670	1.43e-007	4.39e-002	1.69e-005
5	1.00	0.05449	1.23	5.45	41.887	5.29e-007	3.96e-002	5.66e-005
6	1.25	0.06888	1.19	6.89	184.631	1.17e-007	5.75e-002	1.82e-005
7	1.50	0.08141	1.16	8.14	91.385	2.30e-007	5.01e-002	3.10e-005
8	1.75	0.09597	1.13	9.60	0.000	0.00e+000	5.83e-002	0.00e+000
9	2.00	0.1108	1.09	11.1	0.000	0.00e+000	5.95e-002	0.00e+000
10	4.00	0.1696	0.955	17.0	27.227	6.66e-007	2.94e-002	5.28e-005
11	8.00	0.2196	0.837	22.0	14.358	1.11e-006	1.25e-002	3.74e-005
12	16.0	0.2629	0.735	26.3	7.336	1.93e-006	5.41e-003	2.81e-005
13	32.0	0.3001	0.648	30.0	5.838	2.17e-006	2.33e-003	1.36e-005
14	8.00	0.2927	0.665	29.3	2.070	5.87e-006	3.08e-004	4.87e-006
15	2.00	0.2803	0.694	28.0	8.079	1.55e-006	2.06e-003	8.60e-006
16	0.500	0.2651	0.730	26.5	44.939	2.89e-007	1.01e-002	7.90e-006

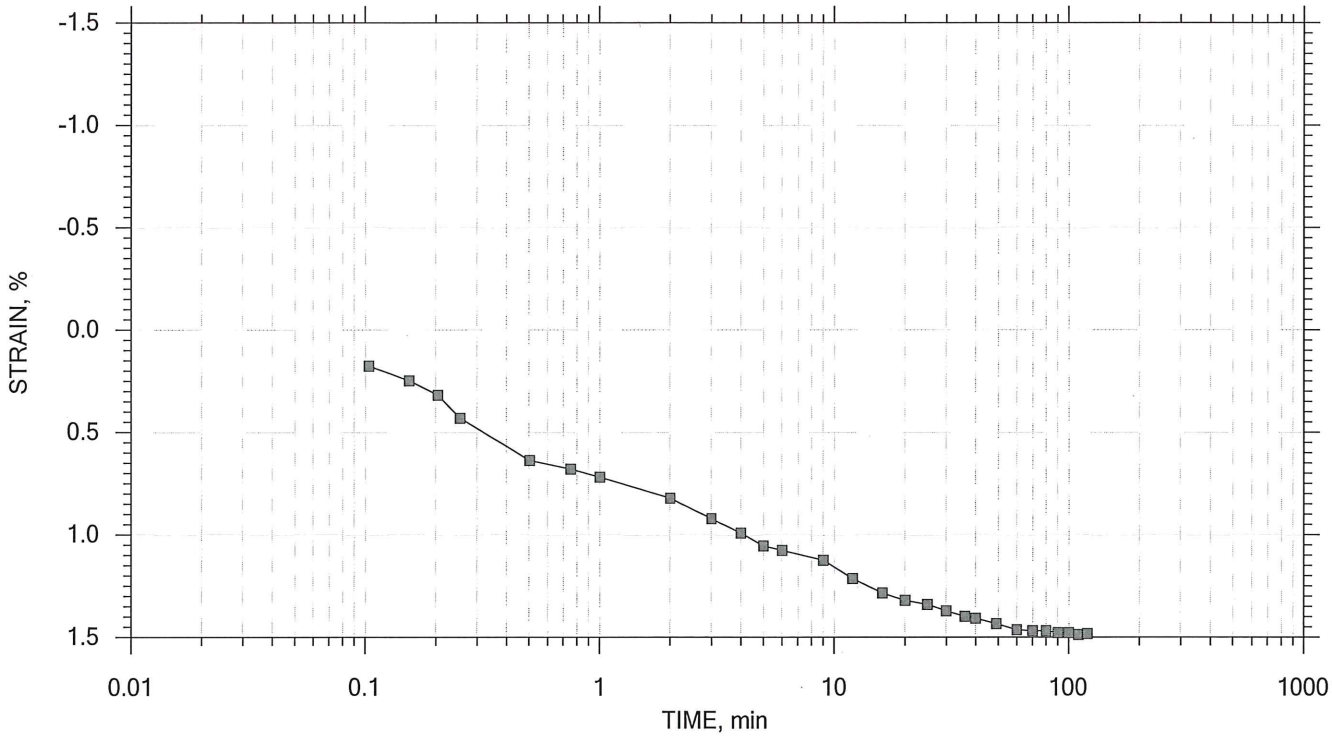
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.125	0.01482	1.32	1.48	0.911	6.16e-006	1.19e-001	1.97e-003	0.00e+000
2	0.250	0.02349	1.30	2.35	0.000	0.00e+000	6.94e-002	0.00e+000	0.00e+000
3	0.500	0.03362	1.28	3.36	3.160	1.70e-006	4.05e-002	1.86e-004	0.00e+000
4	0.750	0.04459	1.25	4.46	0.000	0.00e+000	4.39e-002	0.00e+000	0.00e+000
5	1.00	0.05449	1.23	5.45	0.000	0.00e+000	3.96e-002	0.00e+000	0.00e+000
6	1.25	0.06888	1.19	6.89	0.000	0.00e+000	5.75e-002	0.00e+000	0.00e+000
7	1.50	0.08141	1.16	8.14	0.000	0.00e+000	5.01e-002	0.00e+000	0.00e+000
8	1.75	0.09597	1.13	9.60	0.000	0.00e+000	5.83e-002	0.00e+000	0.00e+000
9	2.00	0.1108	1.09	11.1	0.000	0.00e+000	5.95e-002	0.00e+000	0.00e+000
10	4.00	0.1696	0.955	17.0	4.634	9.09e-007	2.94e-002	7.20e-005	0.00e+000
11	8.00	0.2196	0.837	22.0	2.449	1.51e-006	1.25e-002	5.09e-005	0.00e+000
12	16.0	0.2629	0.735	26.3	1.816	1.81e-006	5.41e-003	2.64e-005	0.00e+000
13	32.0	0.3001	0.648	30.0	1.350	2.18e-006	2.33e-003	1.37e-005	0.00e+000
14	8.00	0.2927	0.665	29.3	0.433	6.51e-006	3.08e-004	5.41e-006	0.00e+000
15	2.00	0.2803	0.694	28.0	1.462	1.99e-006	2.06e-003	1.10e-005	0.00e+000
16	0.500	0.2651	0.730	26.5	10.622	2.84e-007	1.01e-002	7.77e-006	0.00e+000

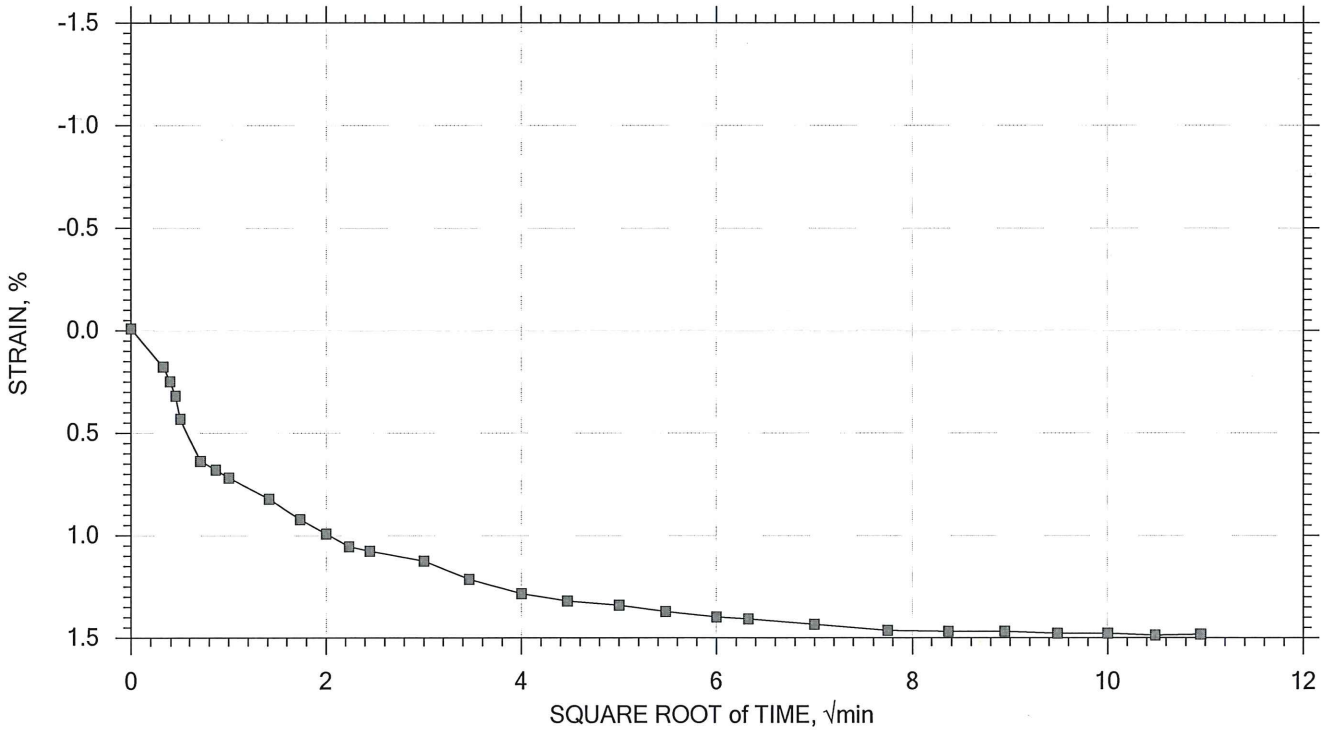
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 1 of 16

Stress: 0.125 tsf





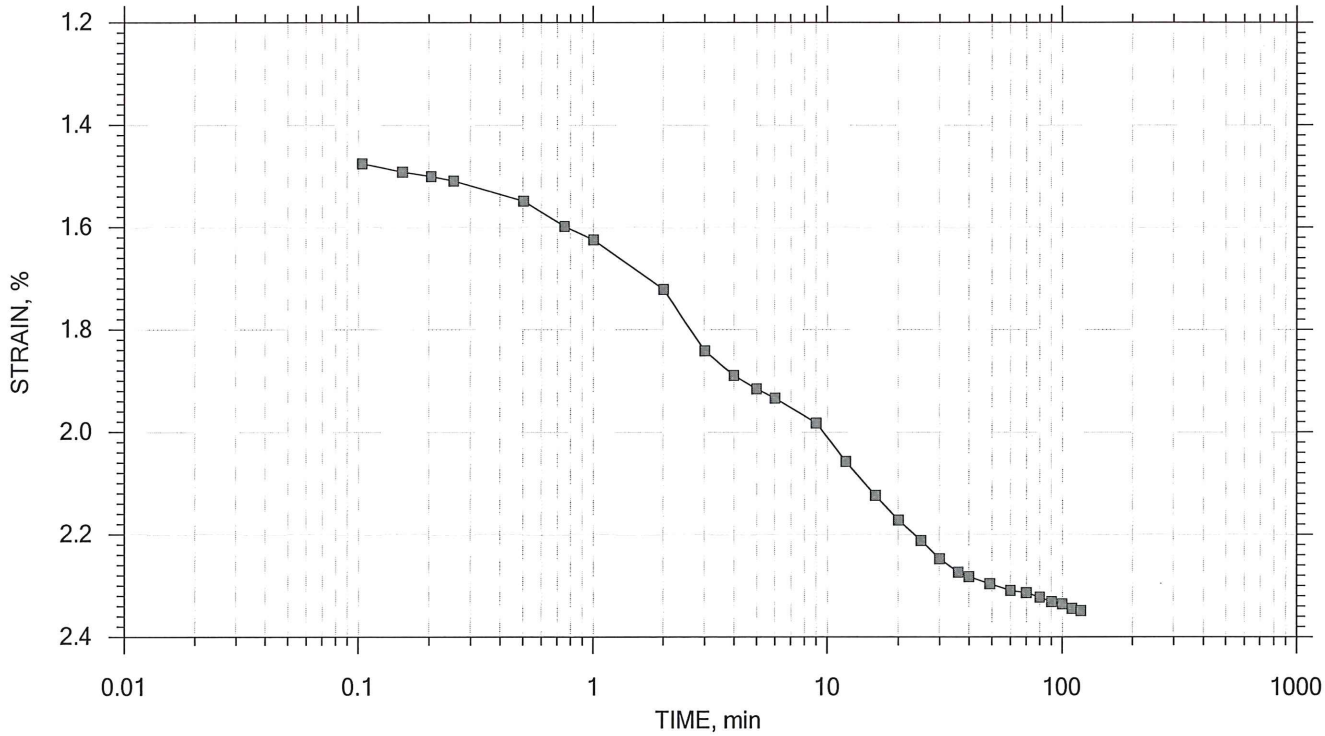
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

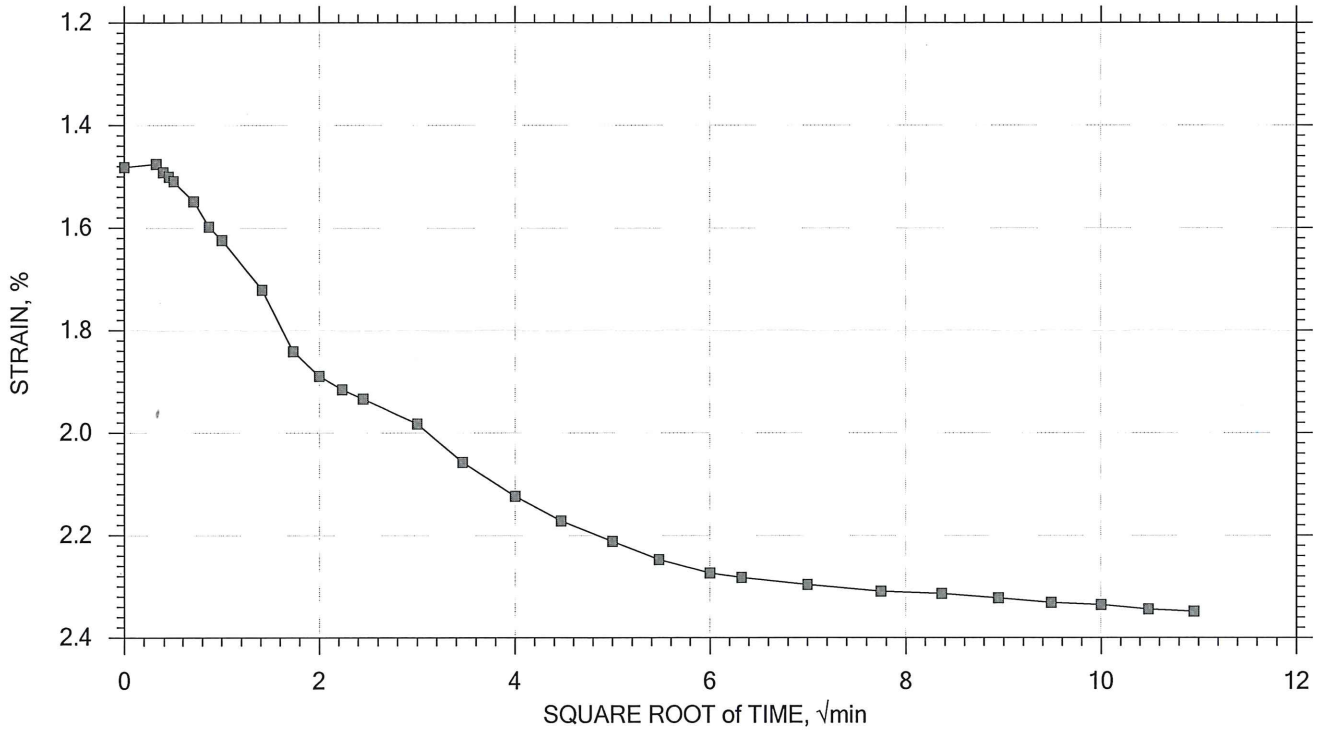
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 2 of 16

Stress: 0.25 tsf





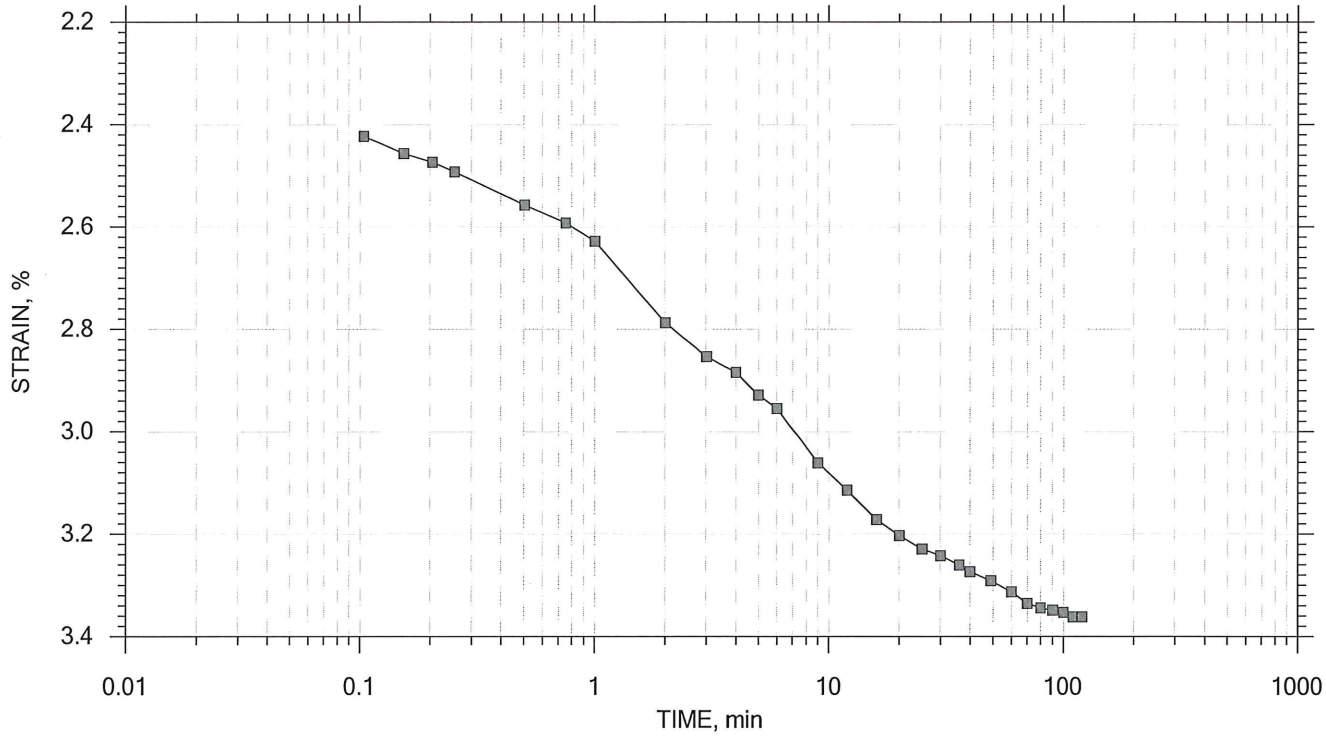
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

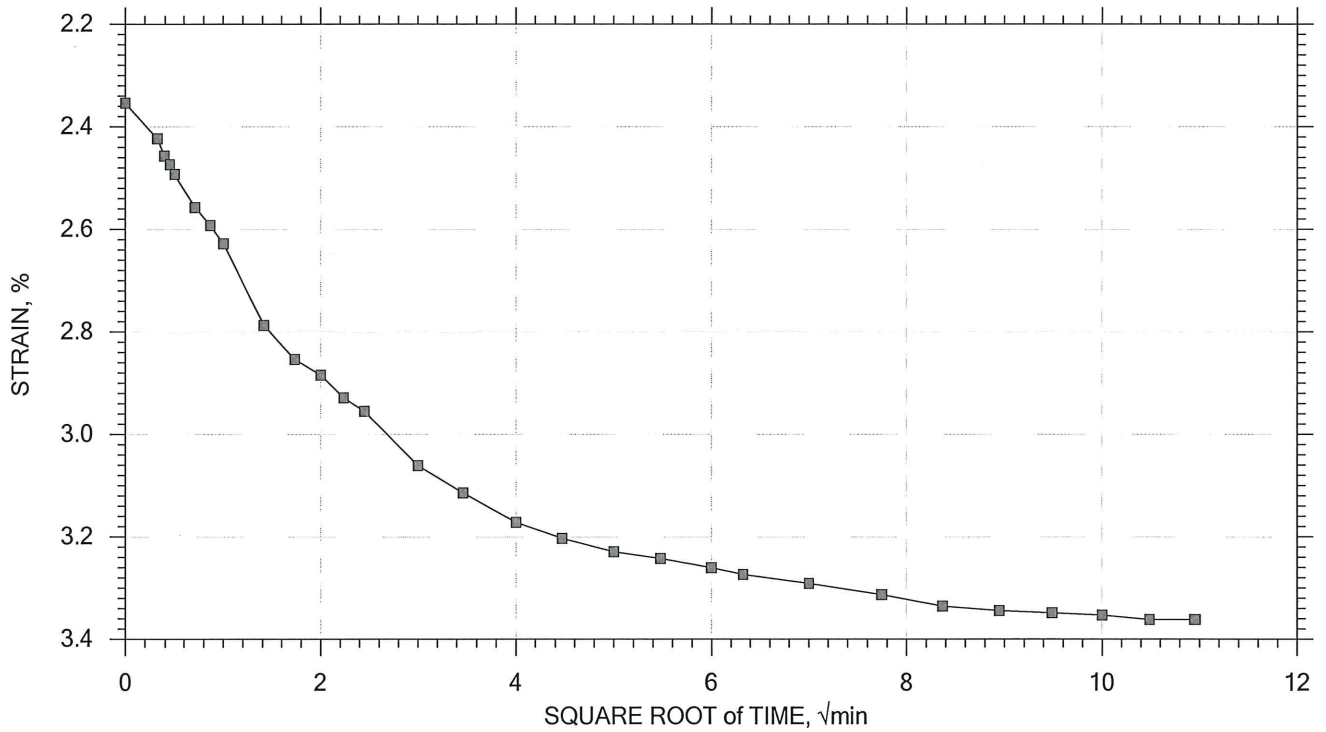
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 3 of 16

Stress: 0.5 tsf





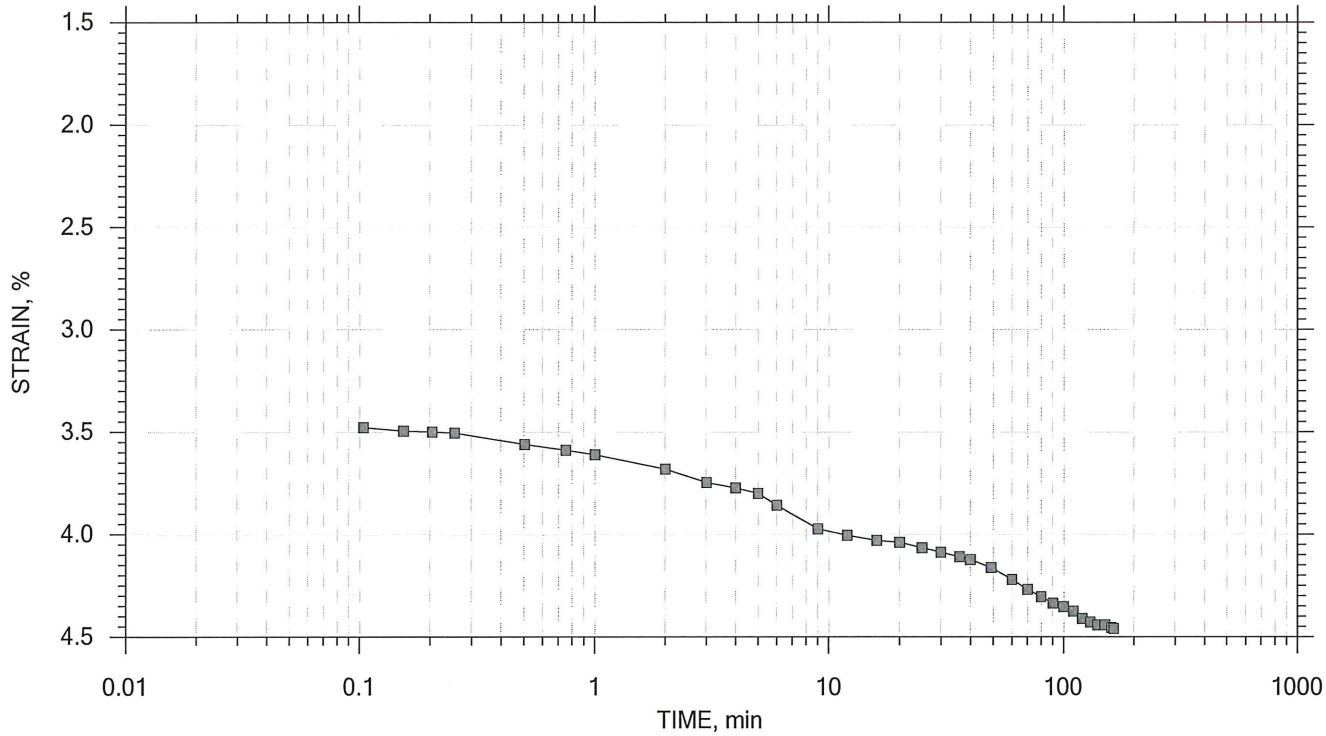
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

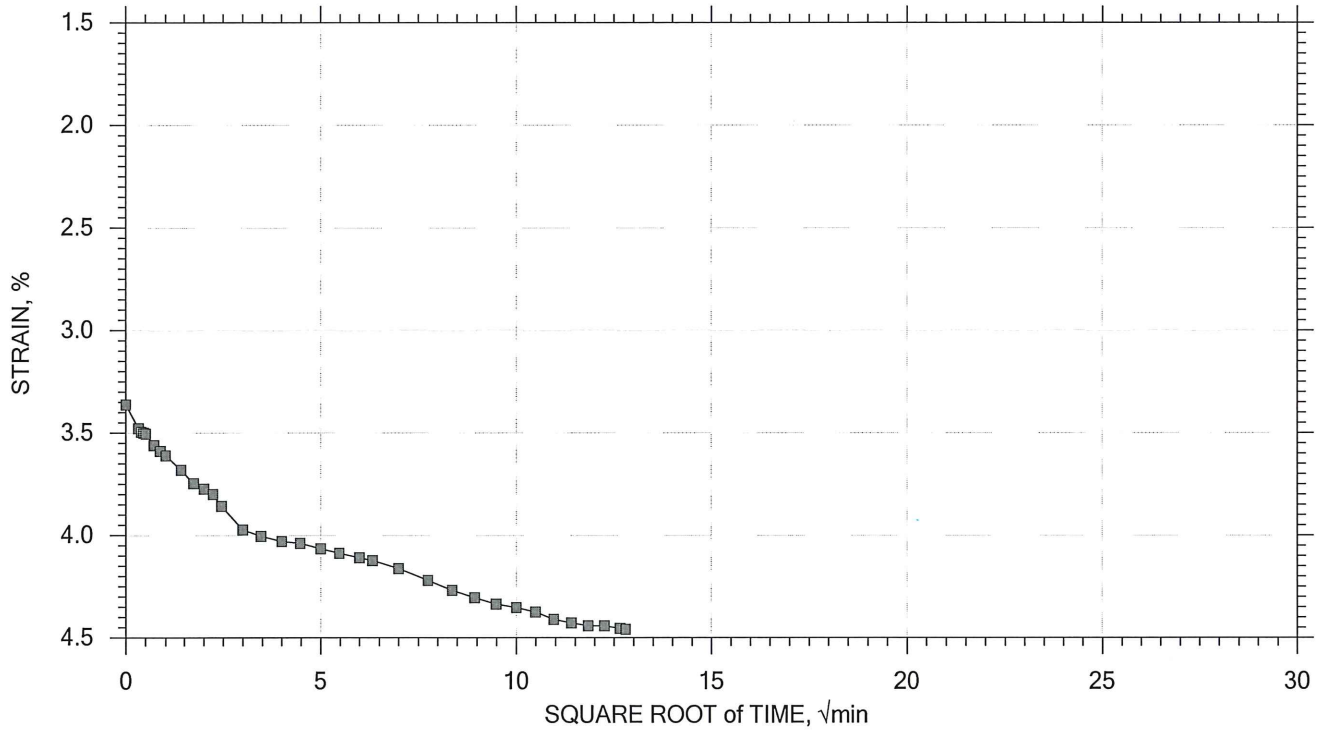
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 4 of 16

Stress: 0.75 tsf





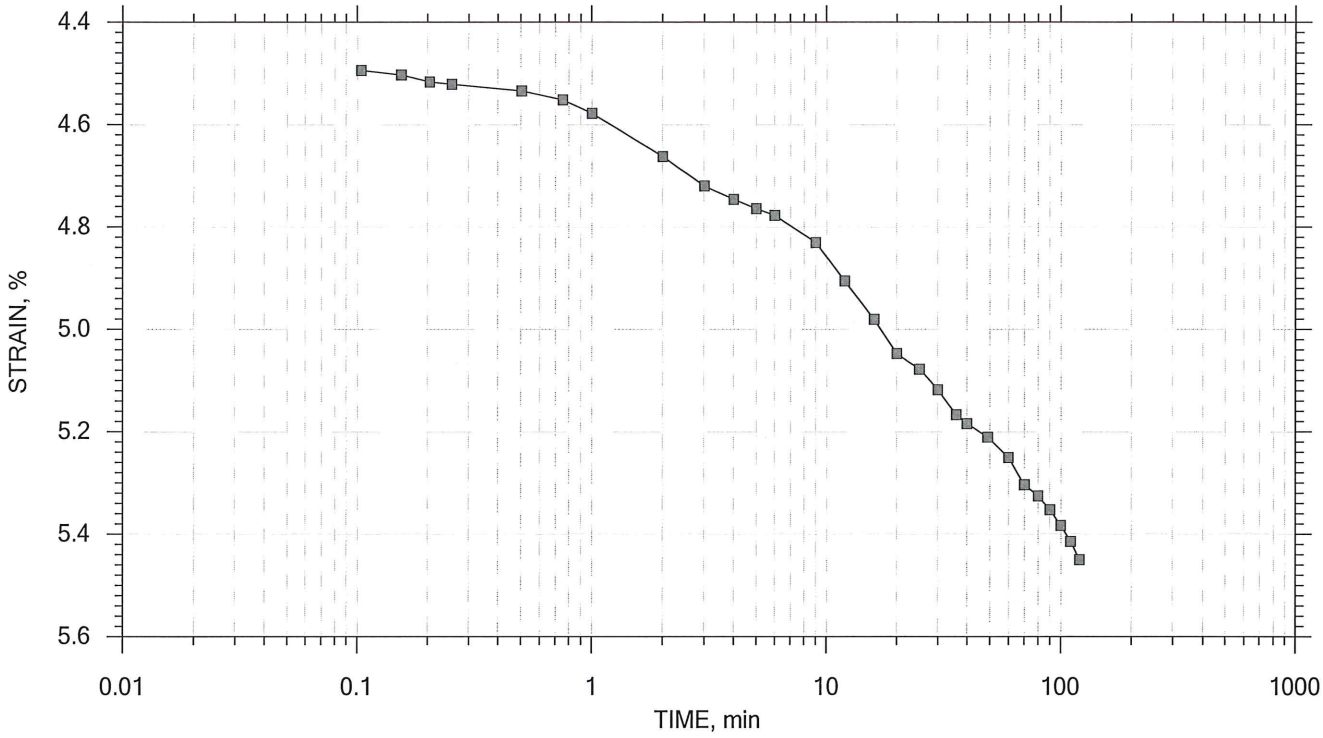
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

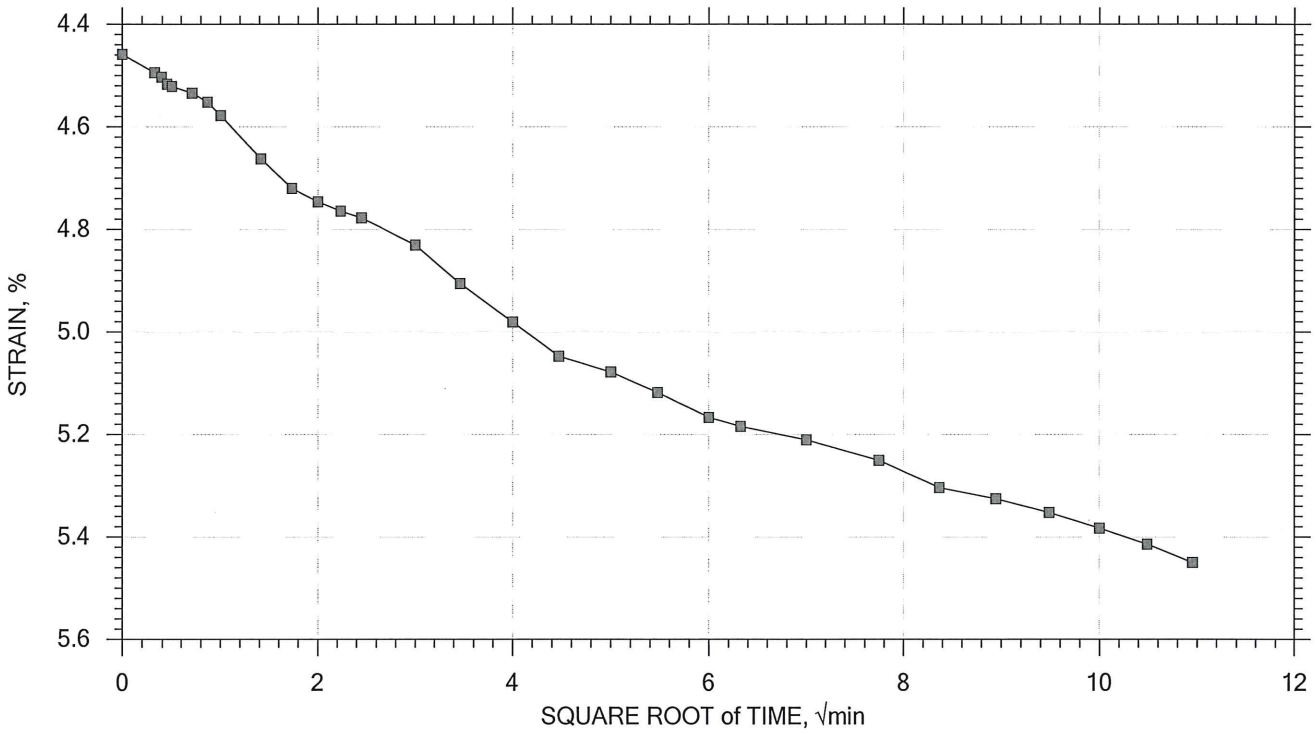
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 5 of 16

Stress: 1 tsf





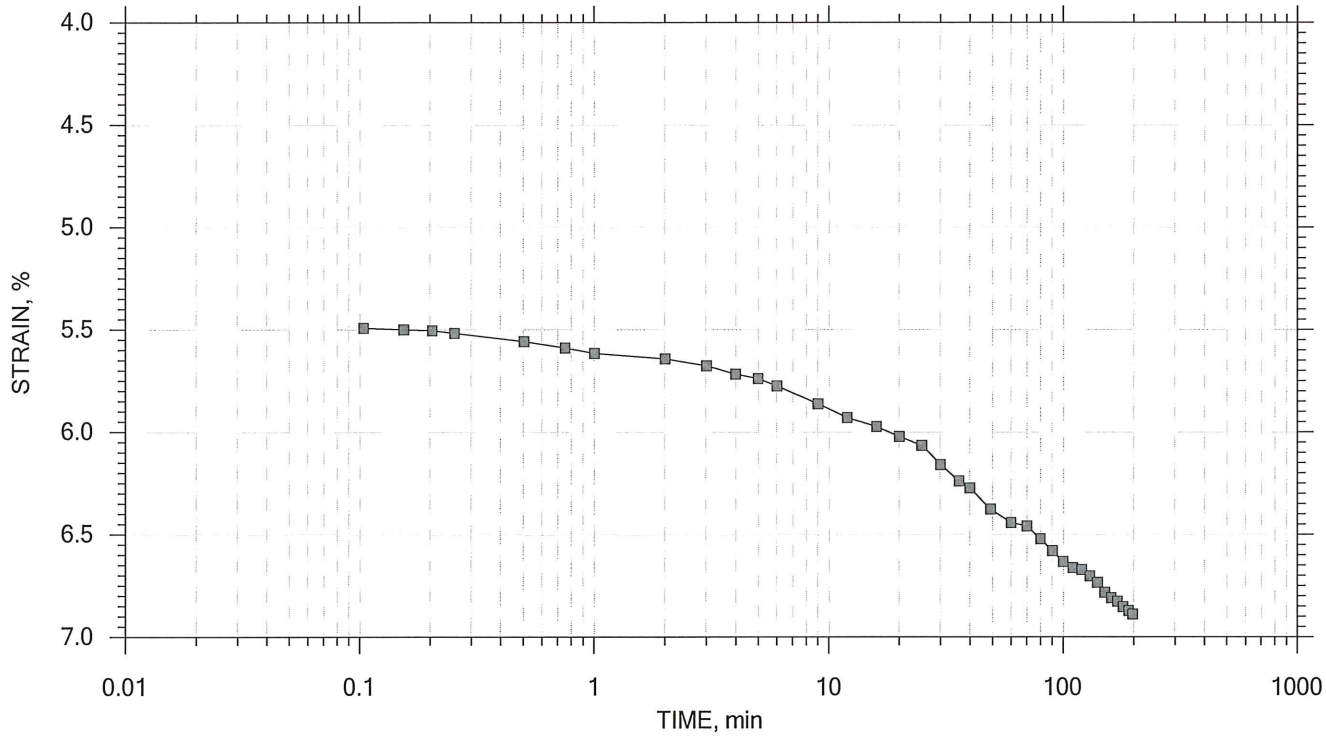
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

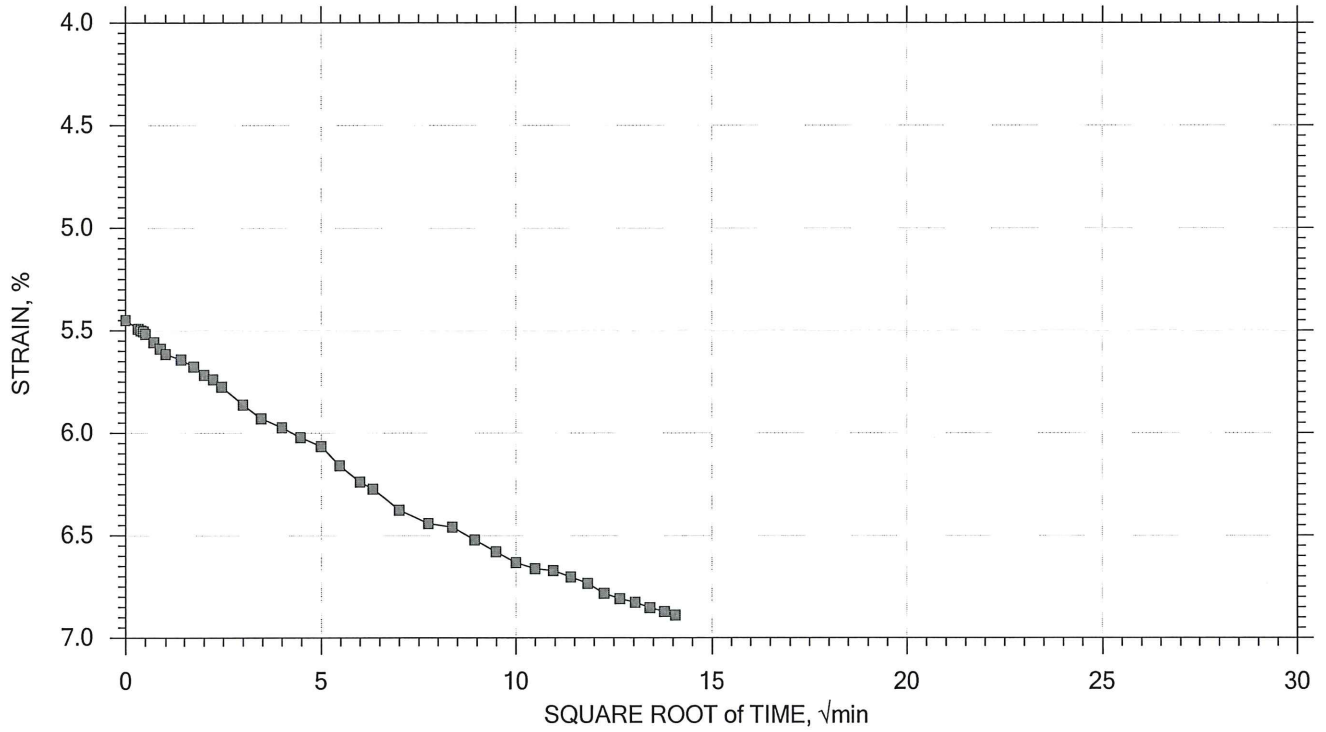
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 6 of 16

Stress: 1.25 tsf





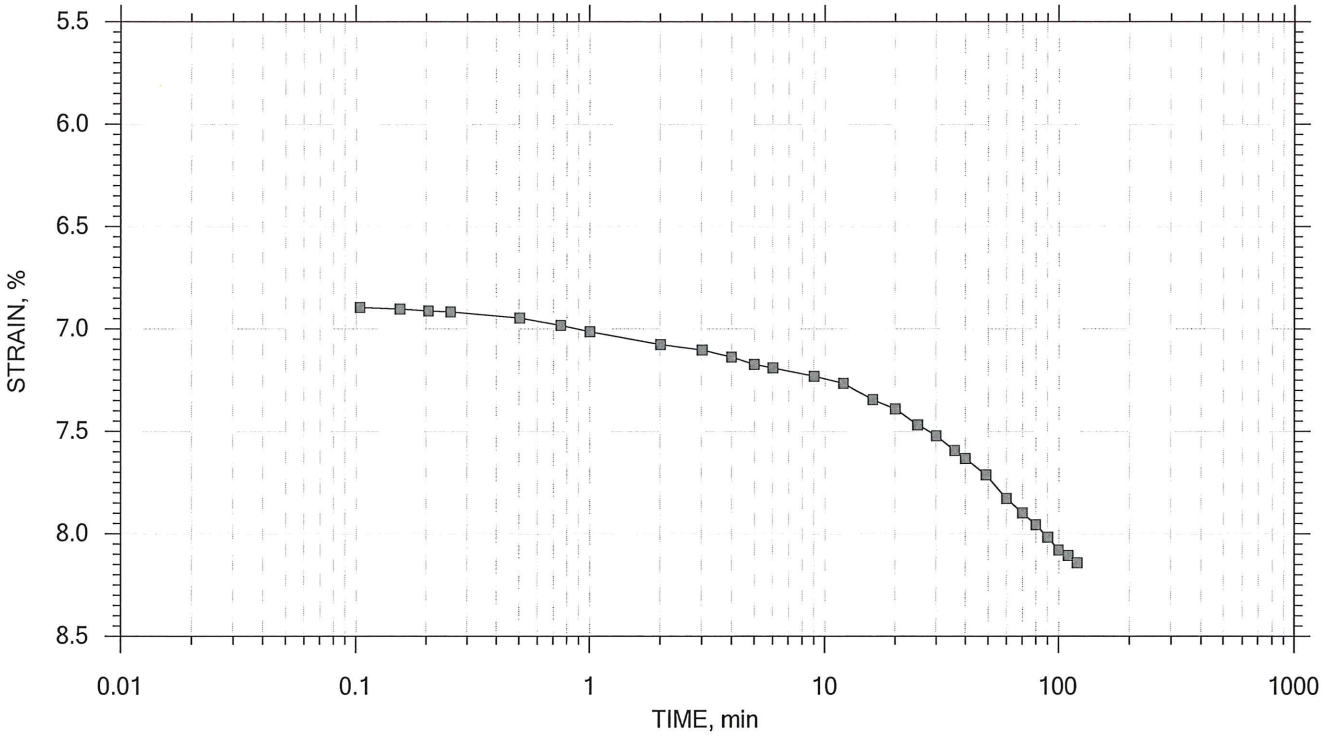
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

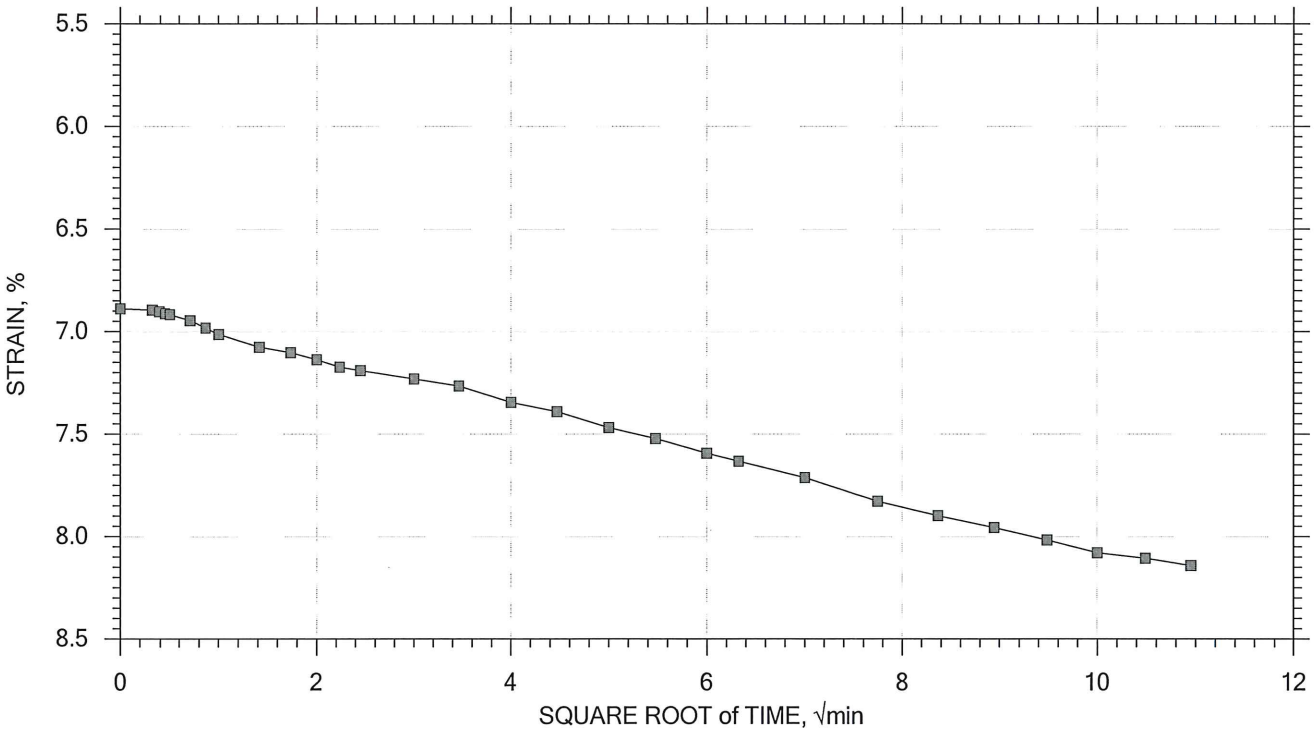
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 7 of 16

Stress: 1.5 tsf





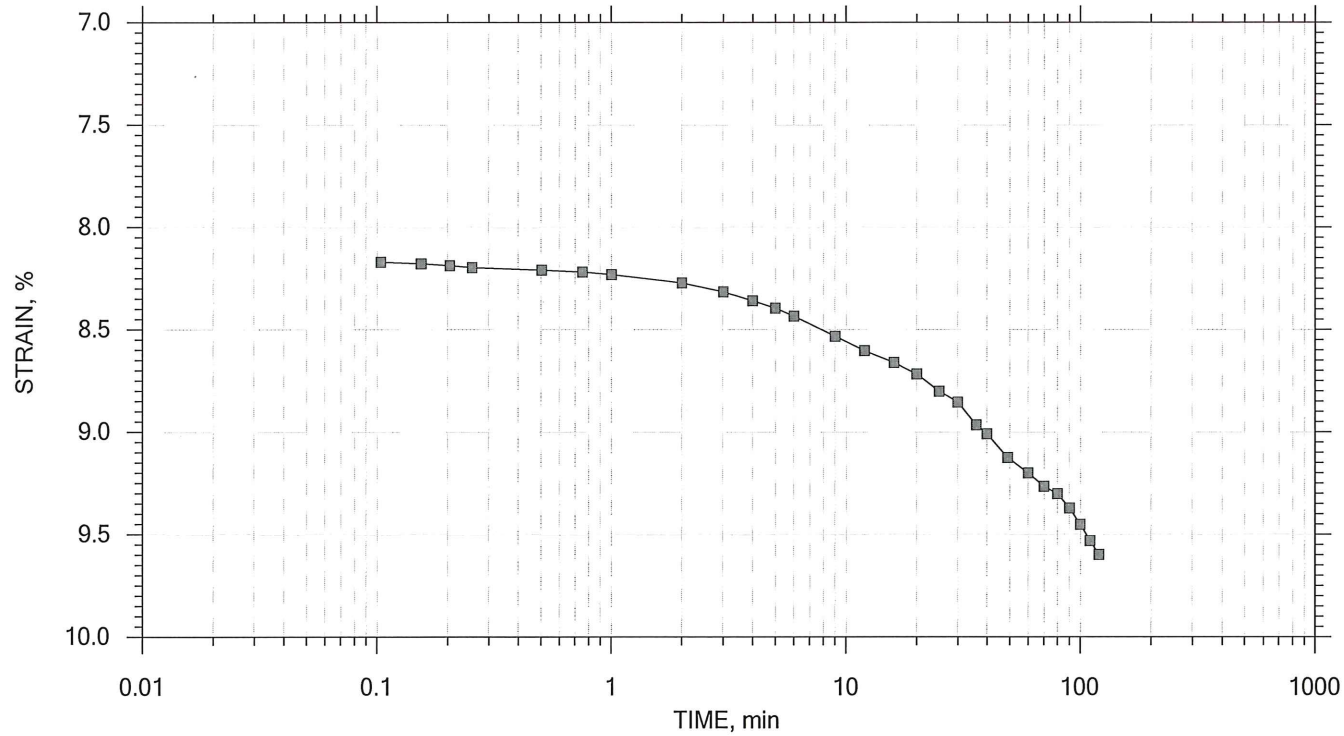
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

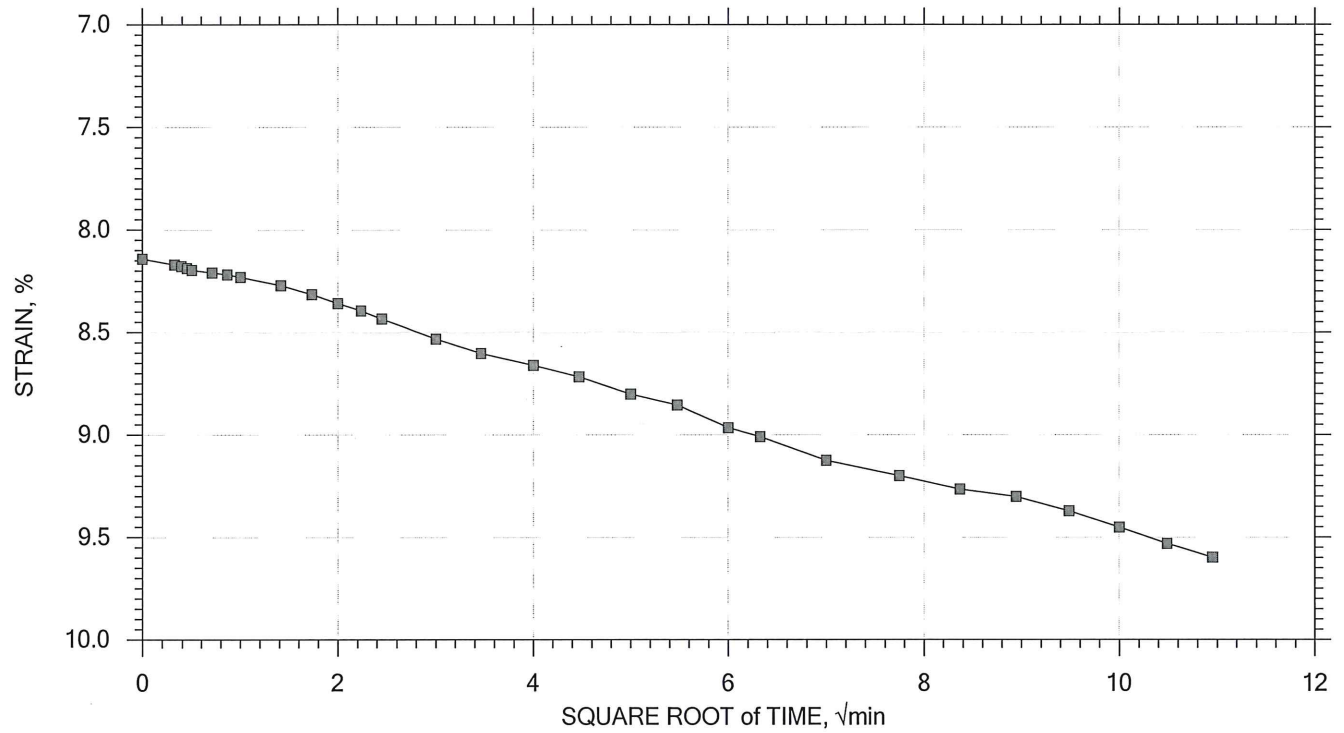
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 8 of 16

Stress: 1.75 tsf





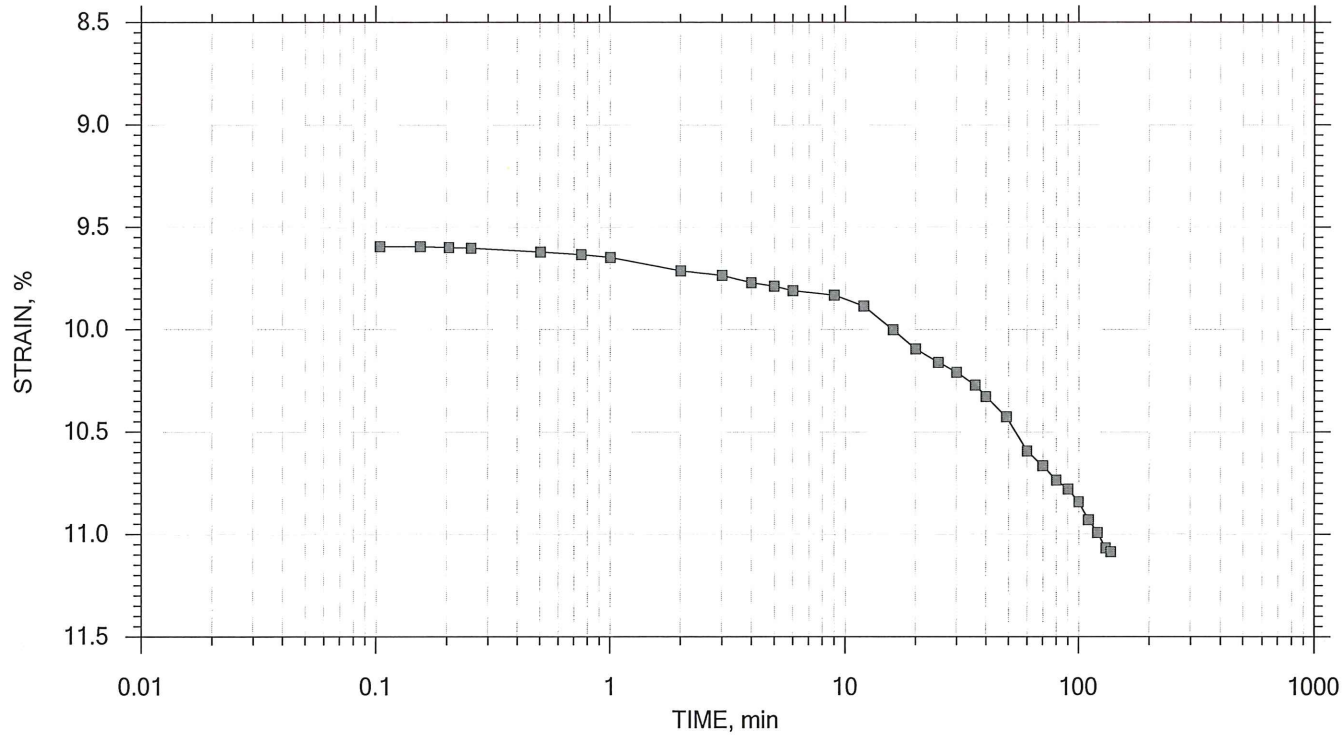
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

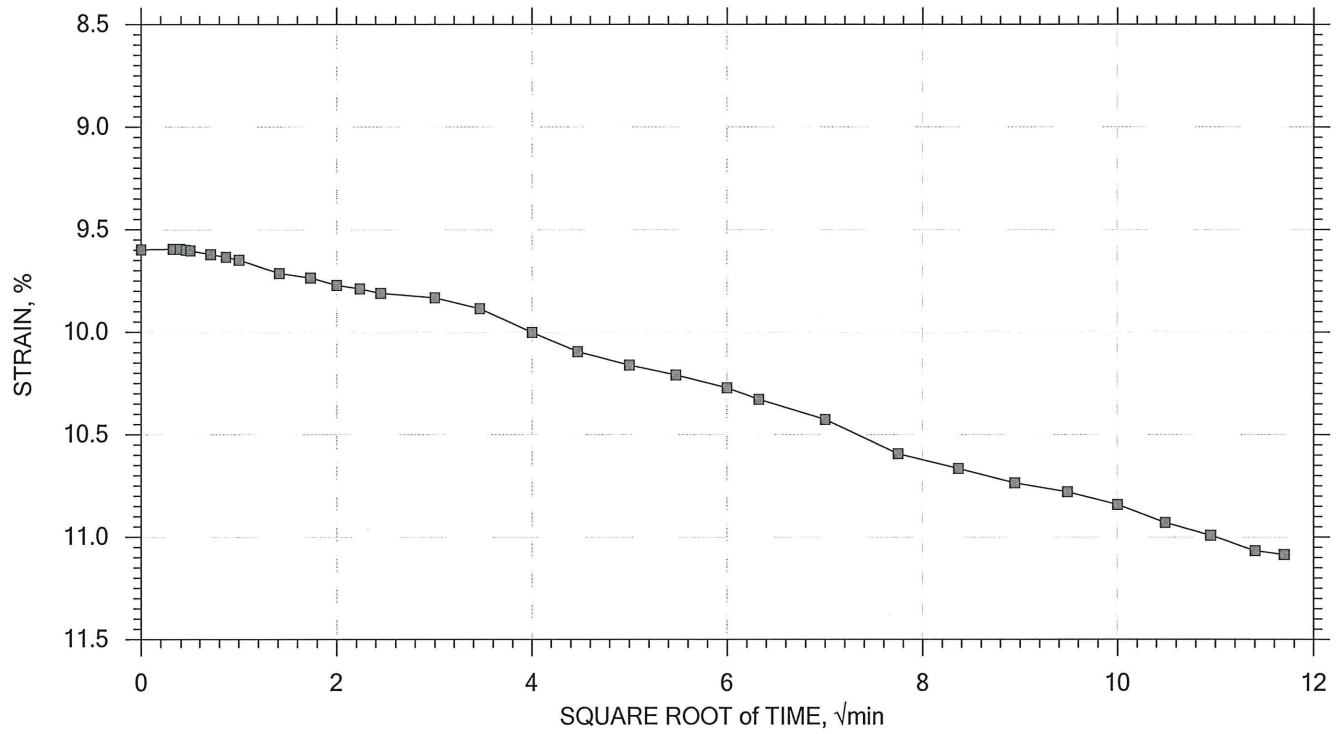
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 9 of 16

Stress: 2 tsf





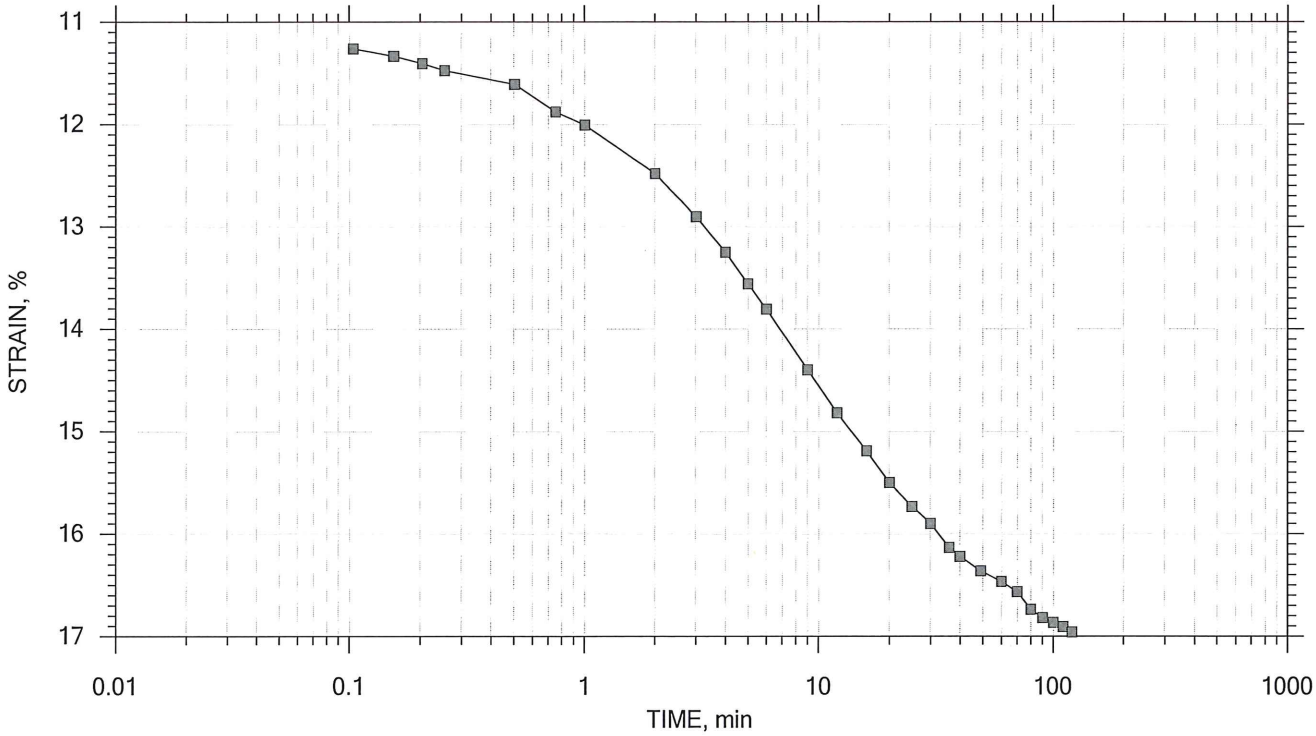
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

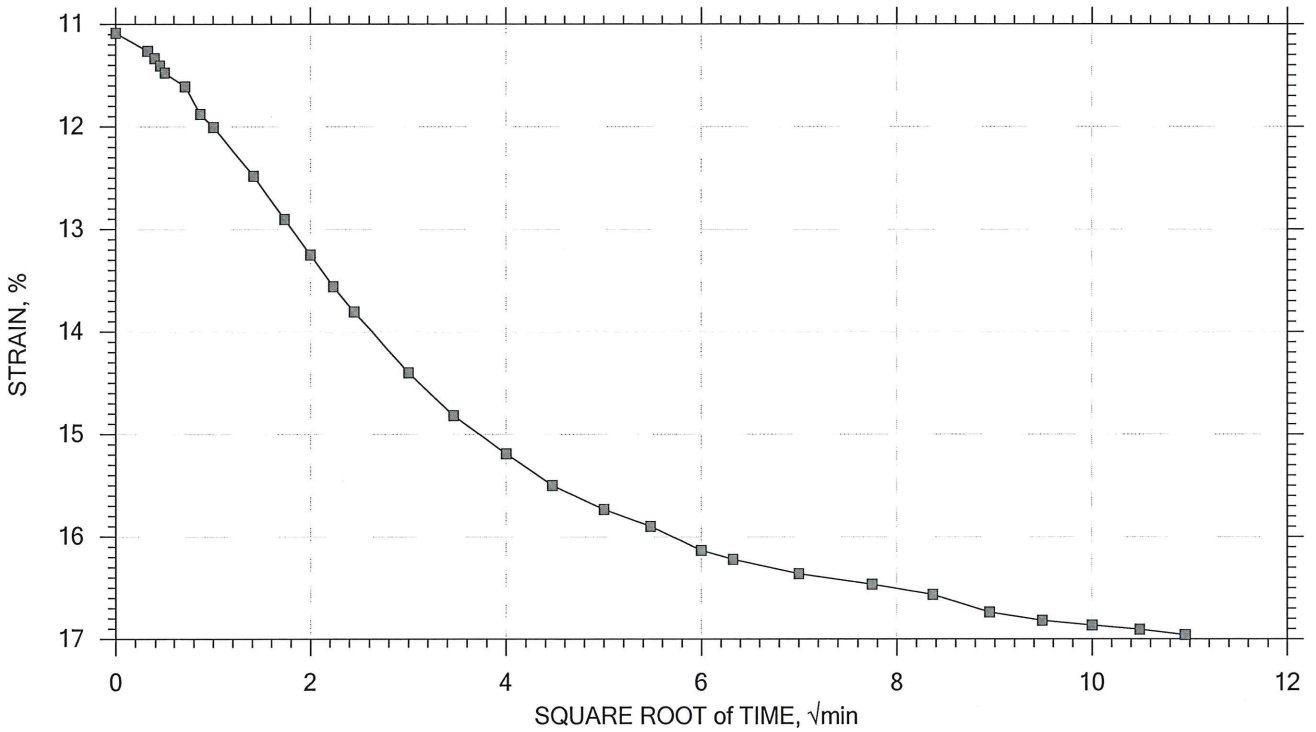
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 10 of 16

Stress: 4 tsf





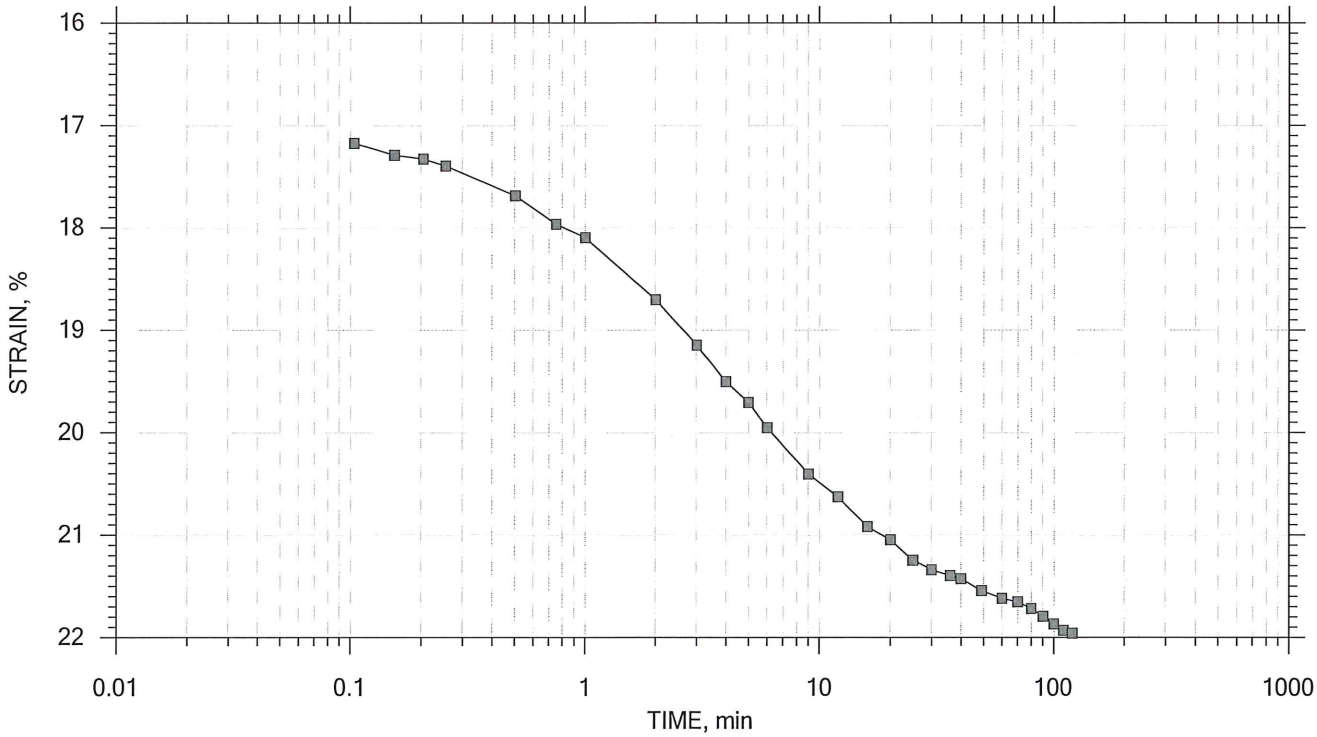
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

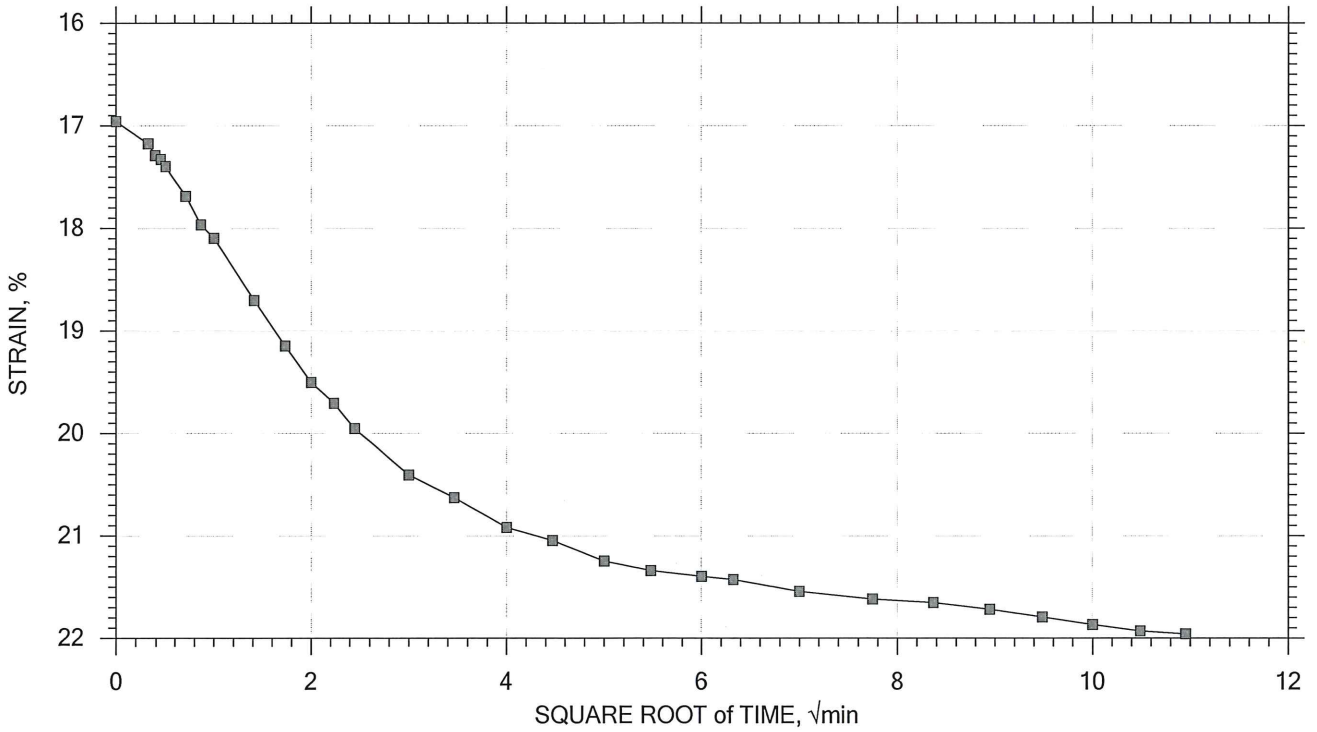
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 11 of 16

Stress: 8 tsf





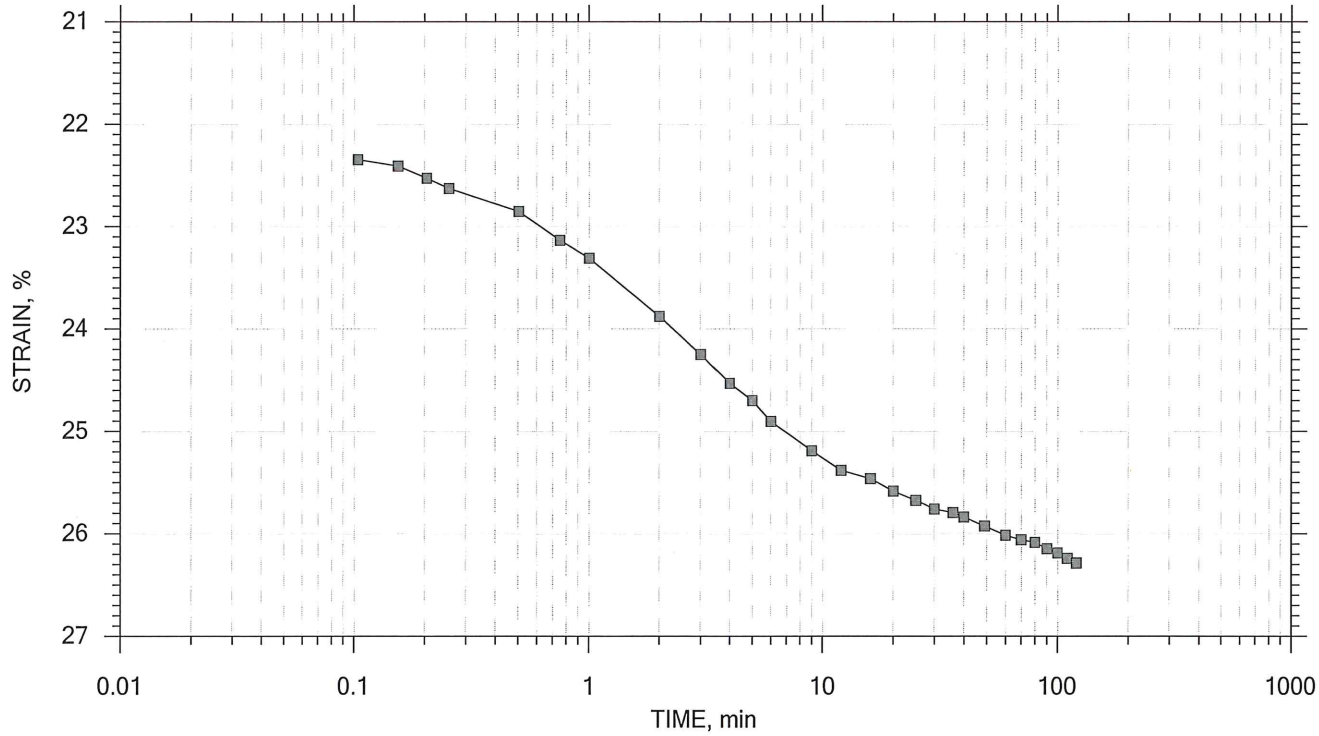
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

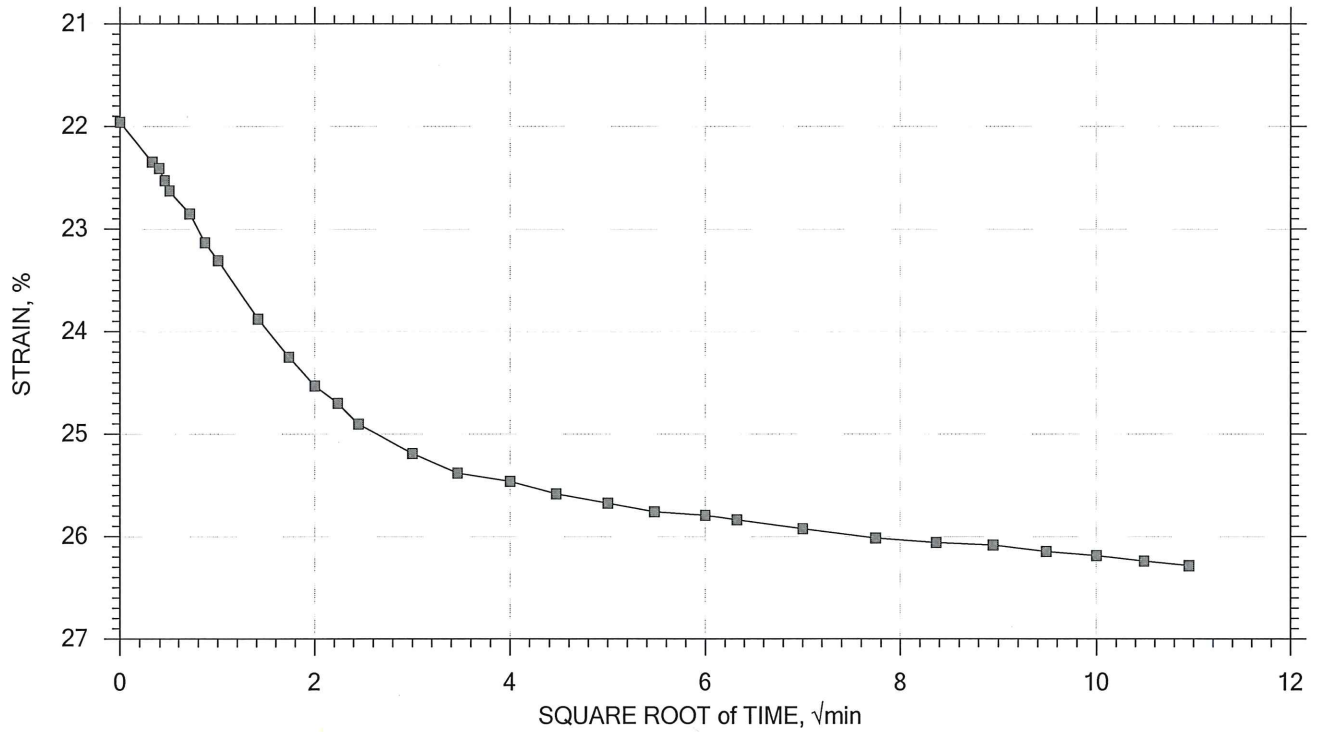
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 12 of 16

Stress: 16 tsf





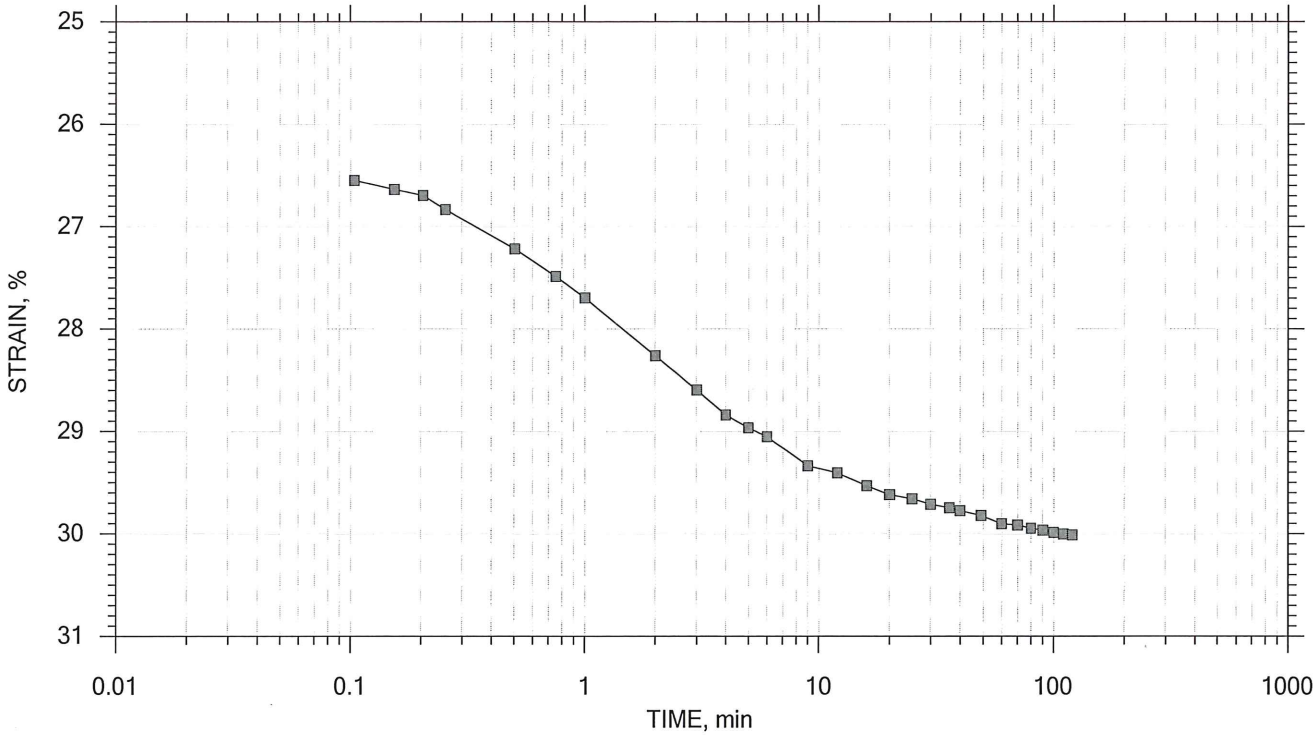
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

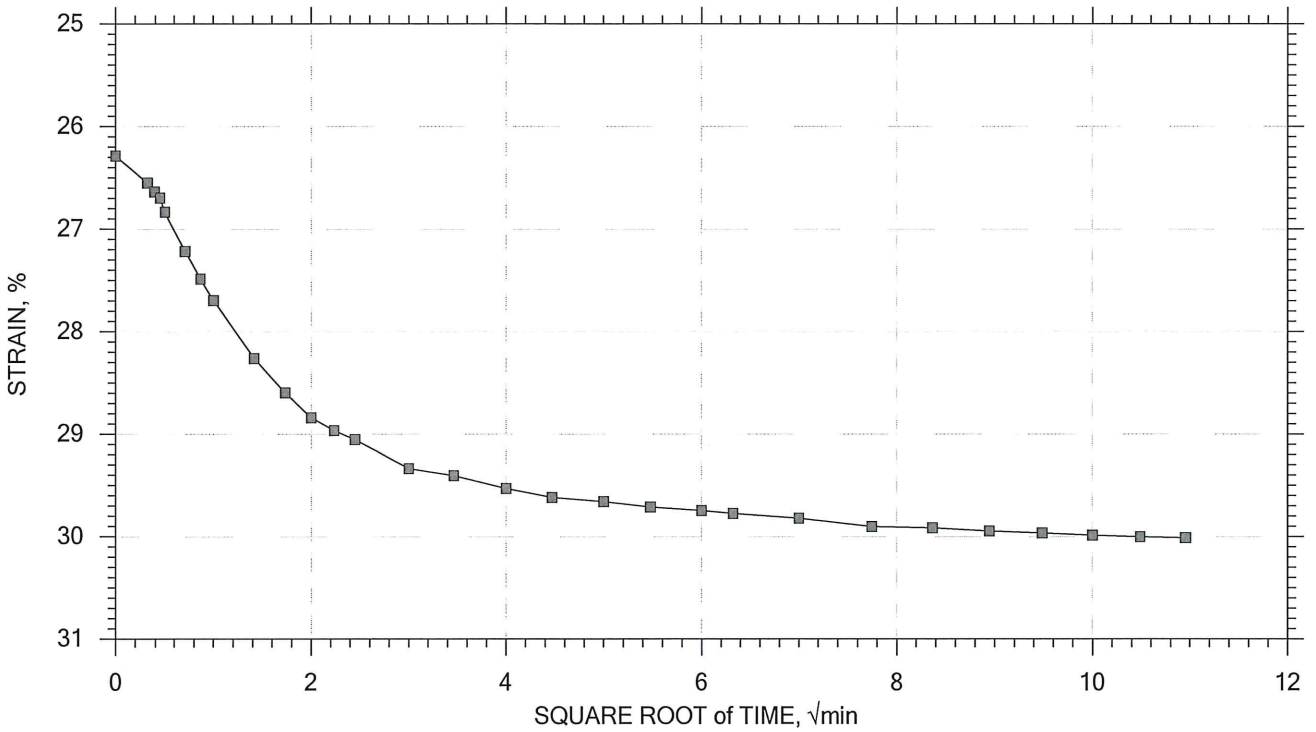
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 13 of 16

Stress: 32 tsf





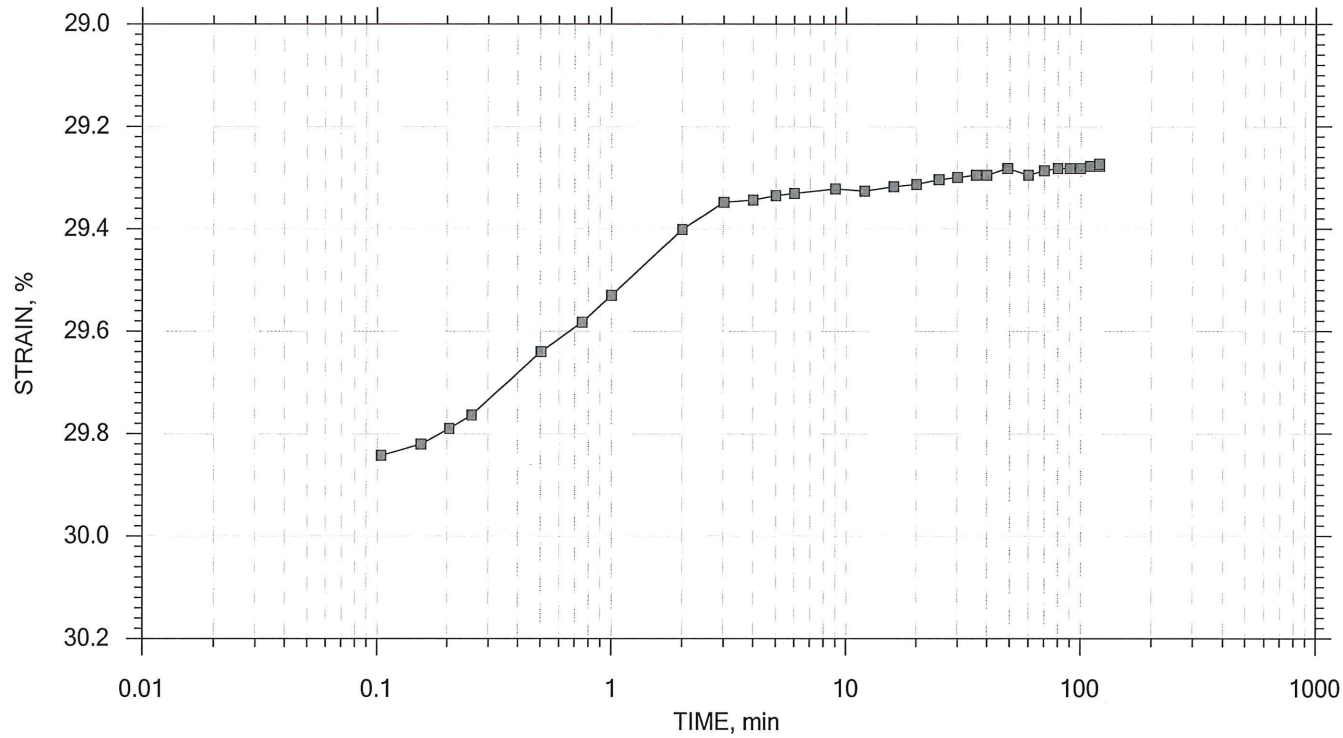
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

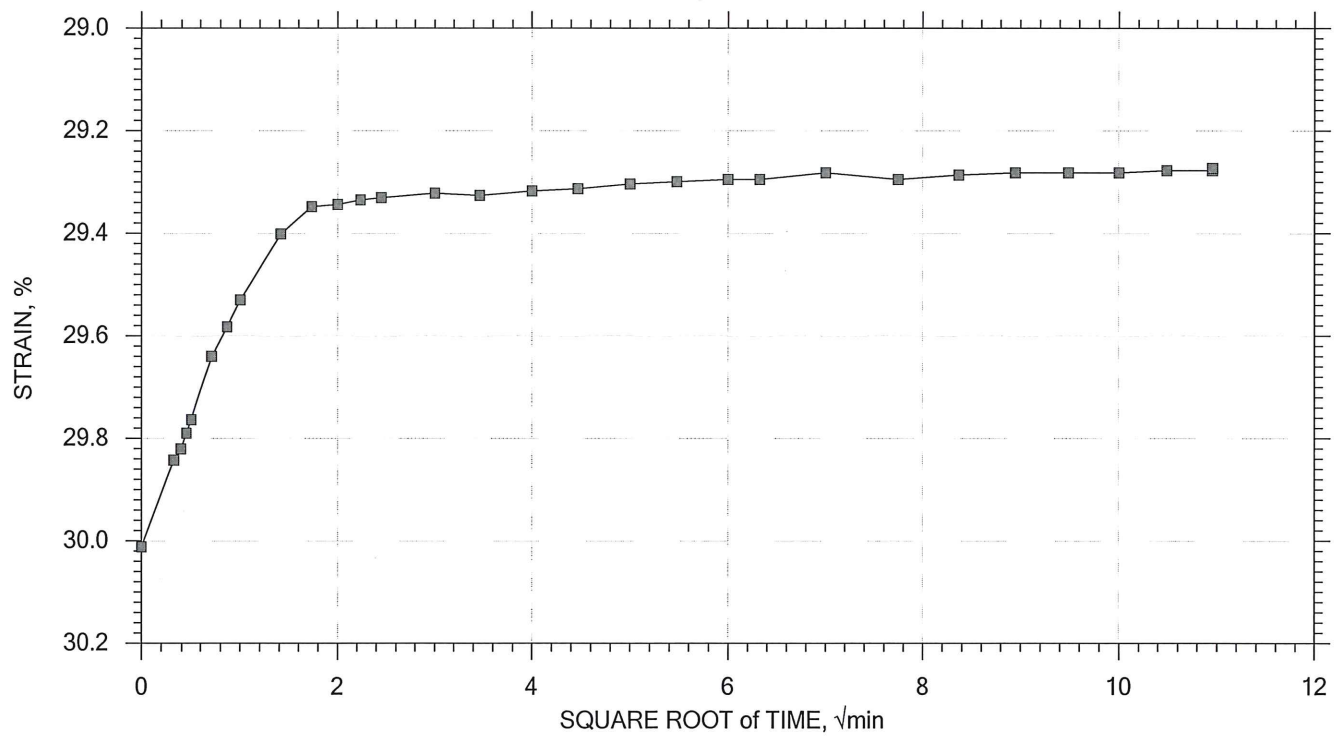
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 14 of 16

Stress: 8 tsf





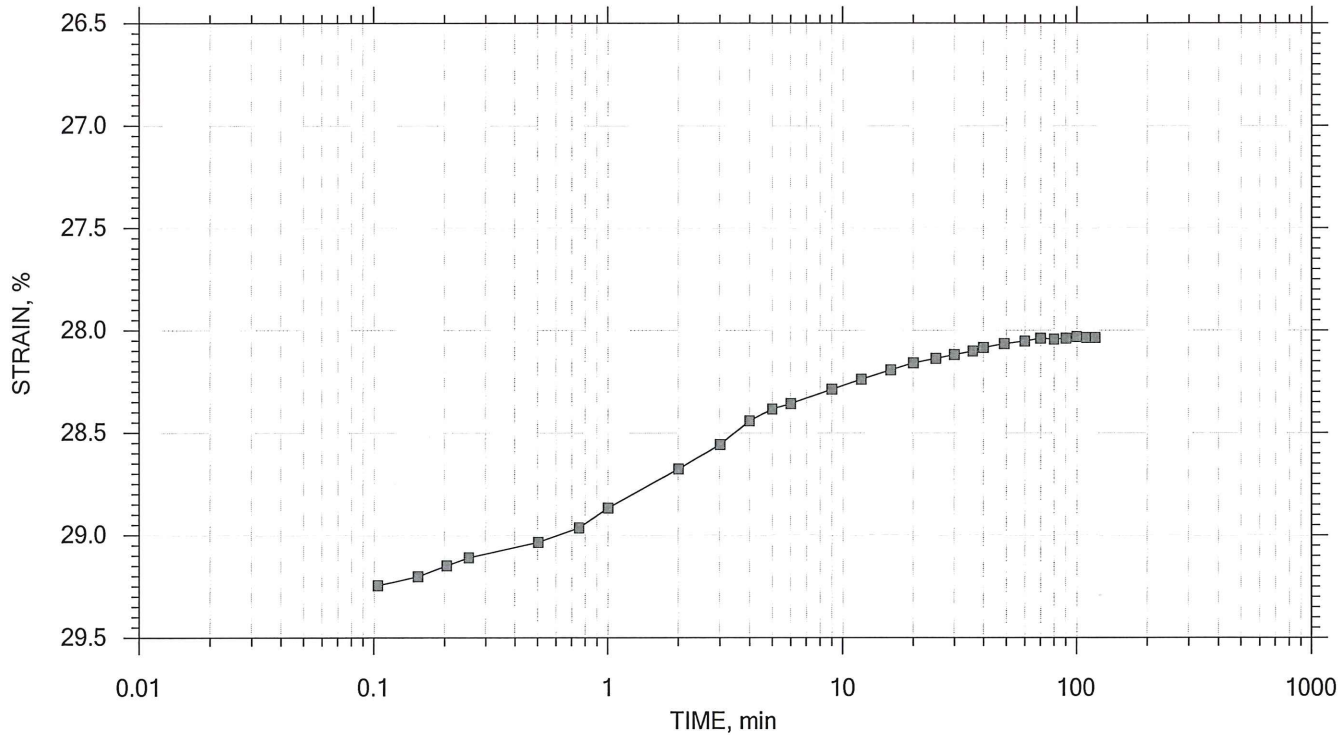
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

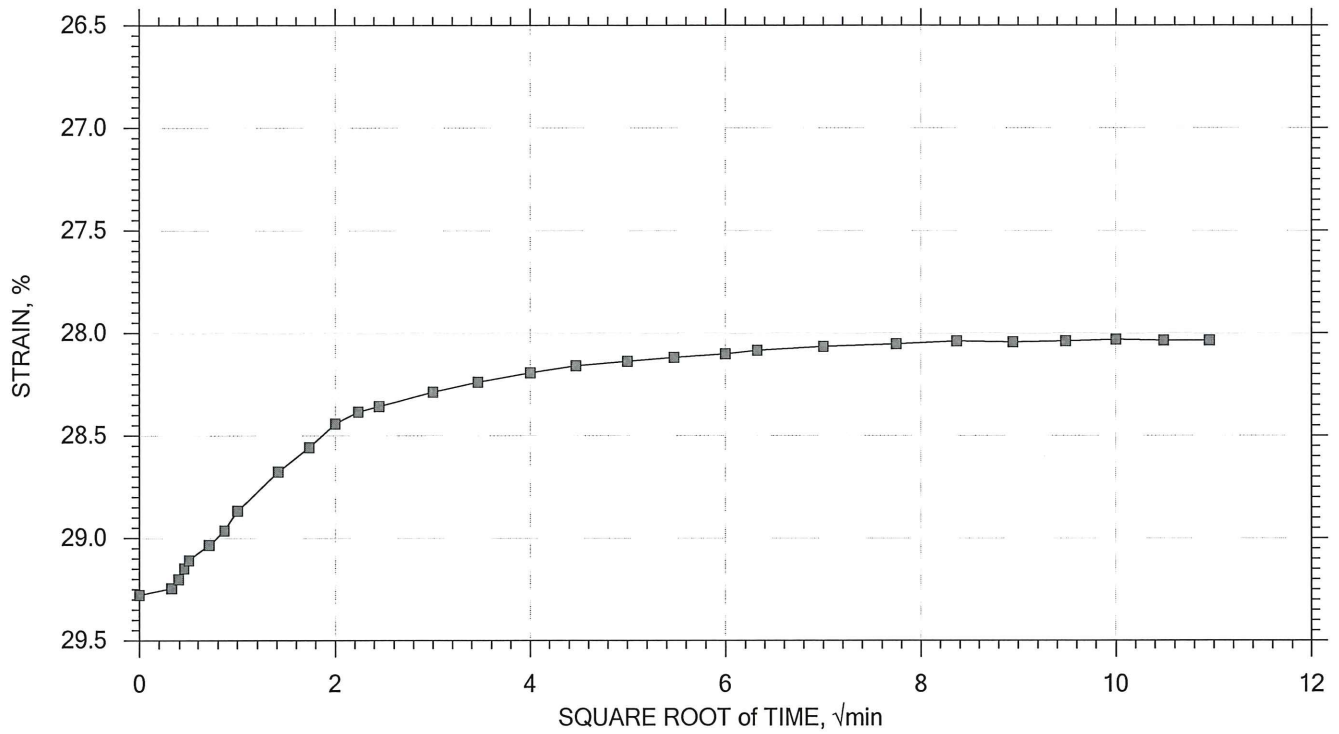
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 15 of 16

Stress: 2 tsf





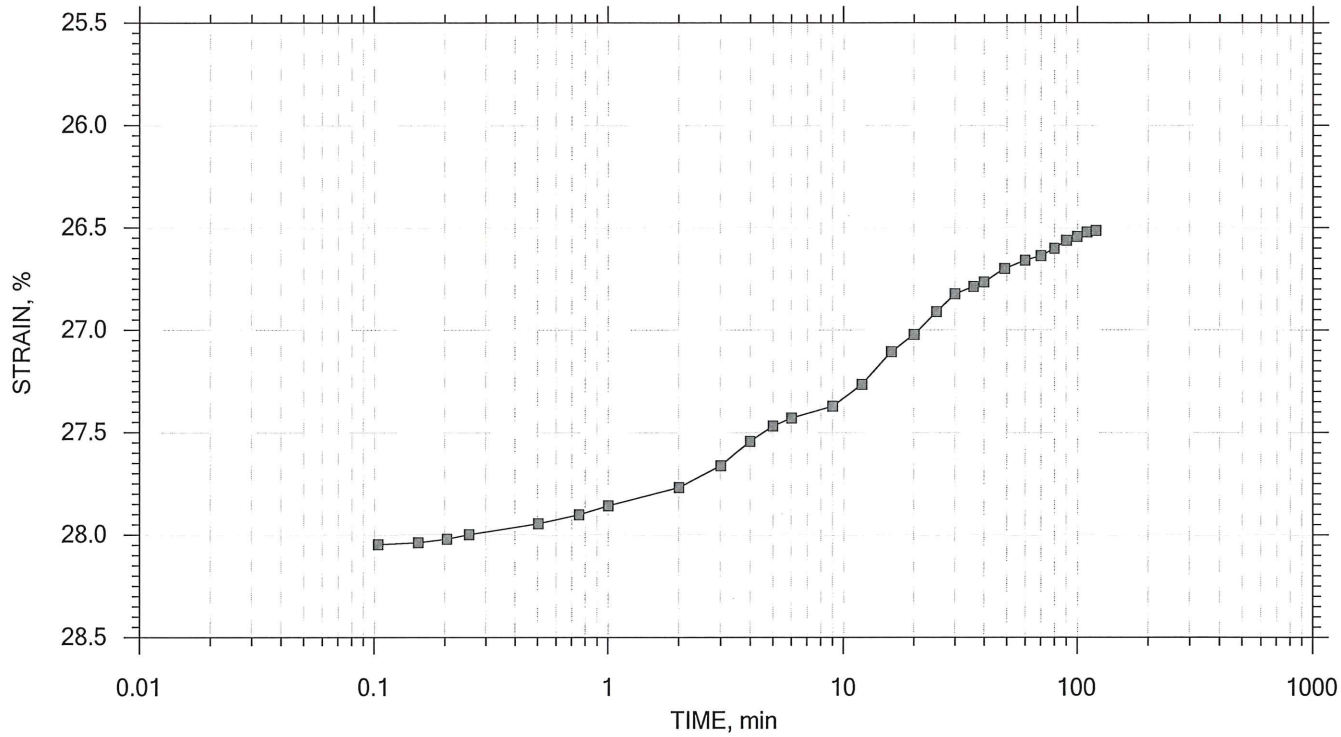
GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

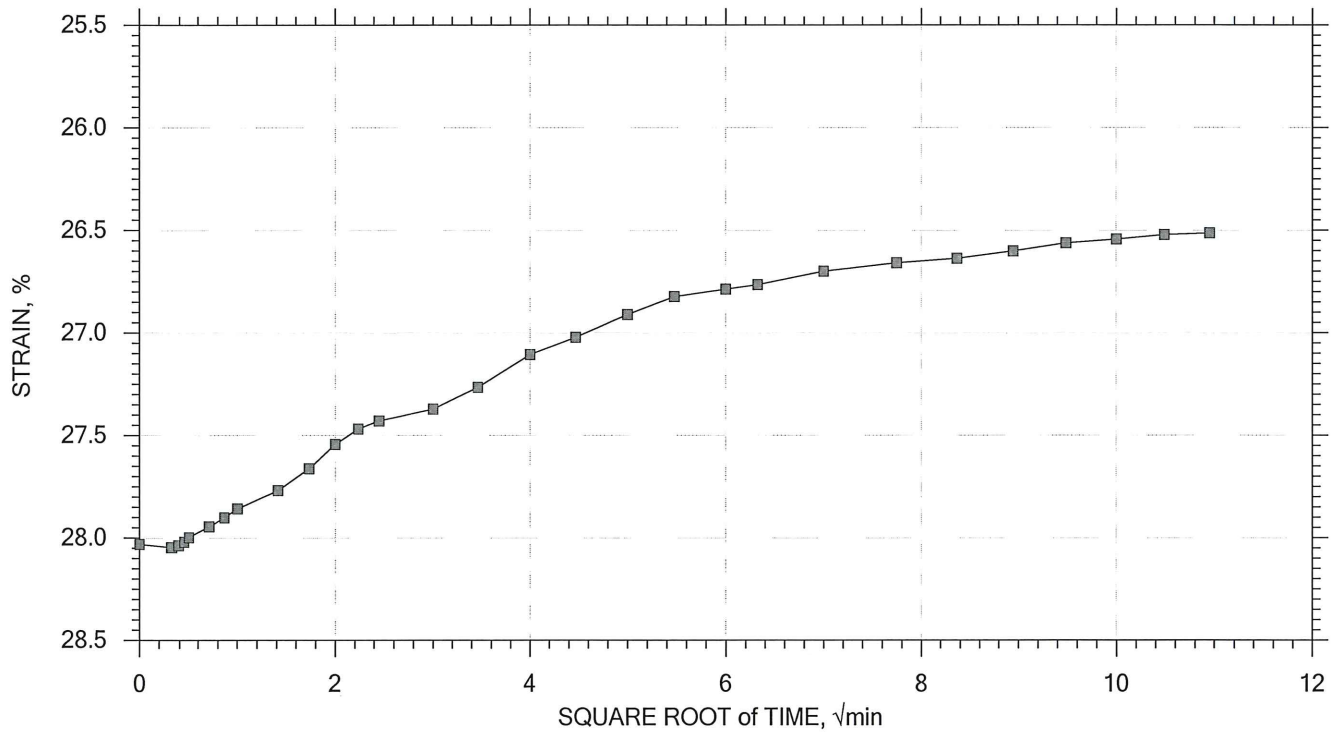
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 16 of 16

Stress: 0.5 tsf

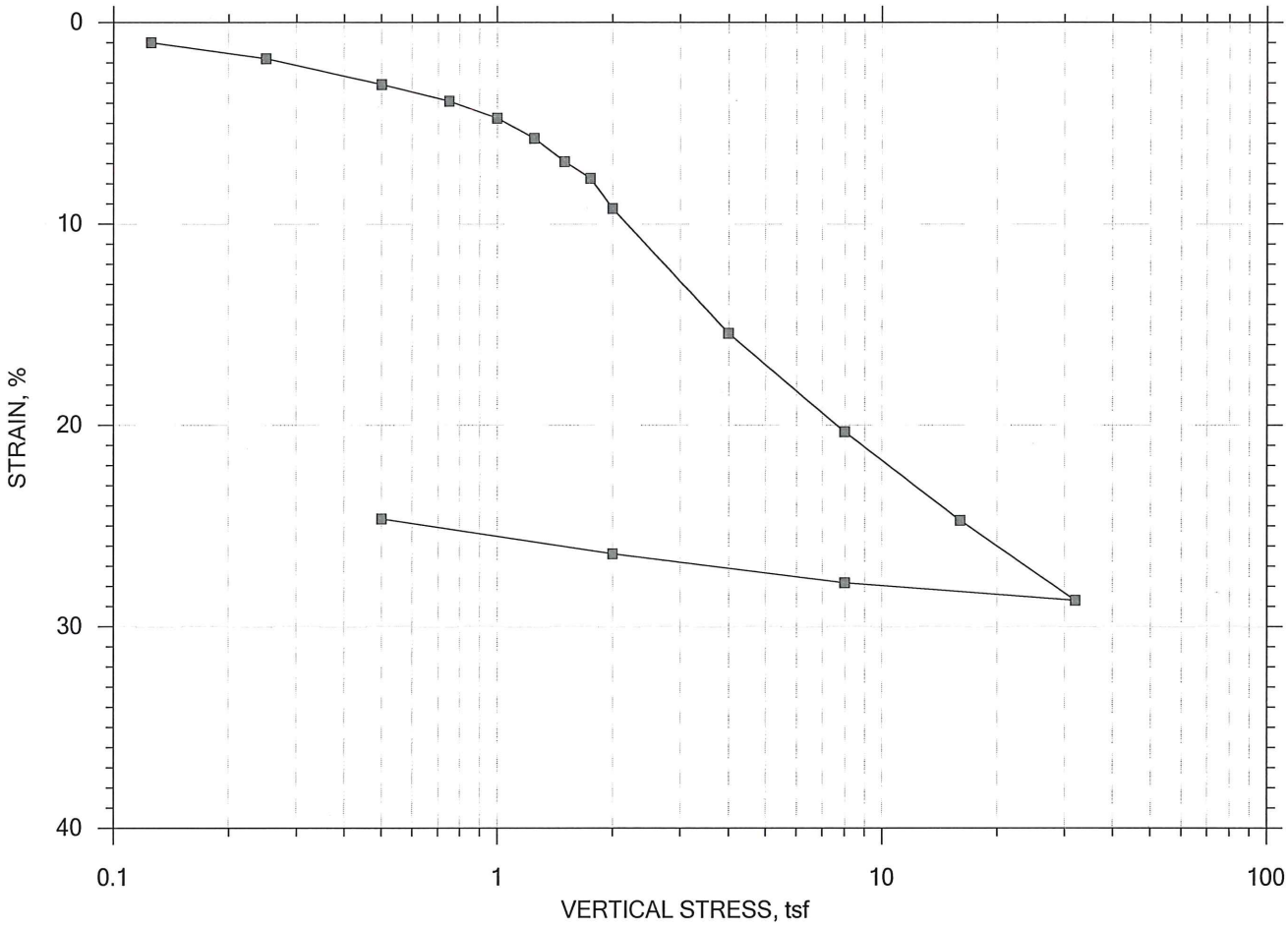


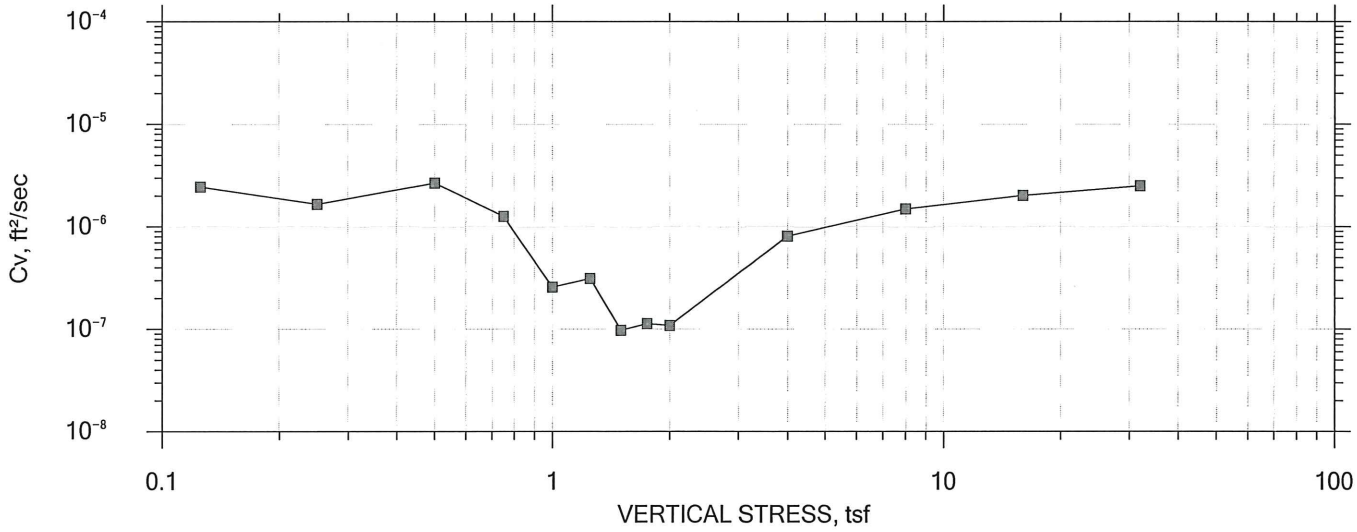


GeoTesting EXPRESS	Project: U.S. Rt.1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U2	Test Date: 04/02/13	Test No.: IP-3
	Depth: 30.0-32.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Y		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

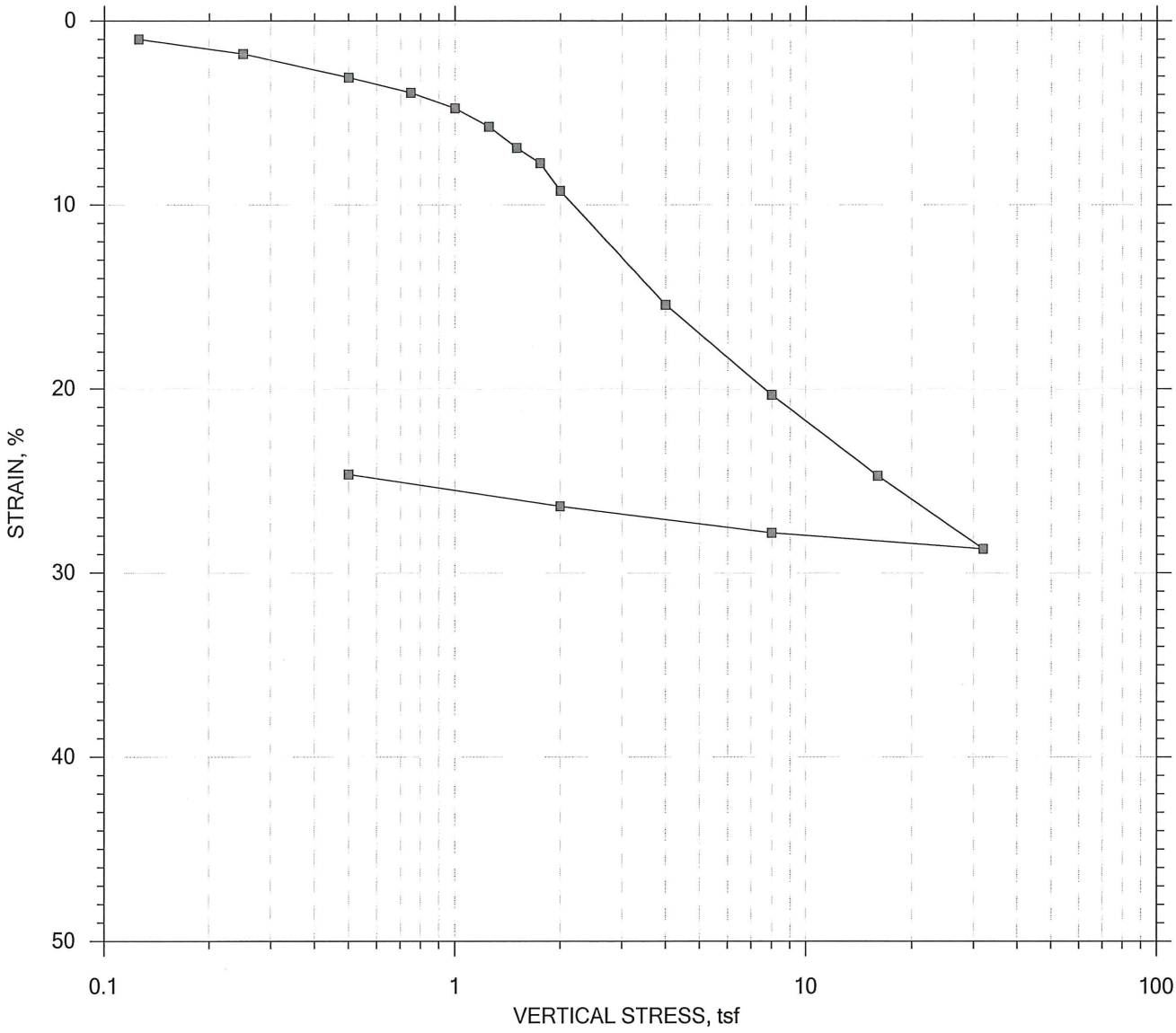




GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		
	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, %	43.40	24.22
Preconsolidation Stress: ---				Dry Unit Weight, pcf	79.094	105.46
Compression Ratio: ---				Saturation, %	98.75	100.00
Diameter: 2.5 in		Height: 1 in		Void Ratio	1.26	0.69
LL: 37	PL: 19	PI: 18	GS: 2.86			

GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: U.S. Rt. 1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U3
Test No.: IP-2

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: intact

Project No.: GTX-300397
Checked By: jdt
Depth: 41.0-43.0 ft
Elevation: ---

Soil Description: Wet, gray clay
Remarks: System Q

Estimated Specific Gravity: 2.86
Initial Void Ratio: 1.26
Final Void Ratio: 0.693

Liquid Limit: 37
Plastic Limit: 19
Plasticity Index: 18

Specimen Diameter: 2.50 in
Initial Height: 1.00 in
Final Height: 0.75 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	10394	RING	R	11462
Wt. Container + Wet Soil, gm	180.98	257.04	237.49	134.78
Wt. Container + Dry Soil, gm	131.85	212.80	212.80	109.97
Wt. Container, gm	7.8600	110.89	110.89	7.5400
Wt. Dry Soil, gm	123.99	101.91	101.91	102.43
Water Content, %	39.62	43.40	24.22	24.22
Void Ratio	---	1.26	0.693	---
Degree of Saturation, %	---	98.75	100.00	---
Dry Unit Weight, pcf	---	79.094	105.46	---

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: U.S. Rt. 1 Viaduct Bridge Repl.
Boring No.: BB-BROV-103
Sample No.: U3
Test No.: IP-2

Location: Bath, ME
Tested By: md
Test Date: 04/02/13
Sample Type: intact

Project No.: GTX-300397
Checked By: jdt
Depth: 41.0-43.0 ft
Elevation: ---

Soil Description: Wet, gray clay
Remarks: System Q

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	0.125	0.01000	1.23	1.00	9.773	2.49e-006	8.00e-002	5.36e-004
2	0.250	0.01797	1.22	1.80	16.138	1.48e-006	6.38e-002	2.54e-004
3	0.500	0.03065	1.19	3.07	9.347	2.50e-006	5.07e-002	3.42e-004
4	0.750	0.03896	1.17	3.90	20.172	1.13e-006	3.32e-002	1.02e-004
5	1.00	0.04742	1.15	4.74	87.568	2.57e-007	3.38e-002	2.34e-005
6	1.25	0.05737	1.13	5.74	68.062	3.24e-007	3.98e-002	3.48e-005
7	1.50	0.06897	1.10	6.90	221.424	9.73e-008	4.64e-002	1.22e-005
8	1.75	0.07722	1.08	7.72	88.470	2.38e-007	3.30e-002	2.12e-005
9	2.00	0.09225	1.05	9.23	195.276	1.05e-007	6.01e-002	1.71e-005
10	4.00	0.1543	0.908	15.4	21.616	8.72e-007	3.10e-002	7.31e-005
11	8.00	0.2032	0.798	20.3	10.486	1.58e-006	1.22e-002	5.20e-005
12	16.0	0.2473	0.699	24.7	5.665	2.60e-006	5.51e-003	3.86e-005
13	32.0	0.2870	0.609	28.7	3.774	3.49e-006	2.48e-003	2.34e-005
14	8.00	0.2781	0.629	27.8	1.054	1.20e-005	3.68e-004	1.19e-005
15	2.00	0.2639	0.661	26.4	6.376	2.05e-006	2.38e-003	1.31e-005
16	0.500	0.2465	0.700	24.7	35.525	3.83e-007	1.16e-002	1.20e-005

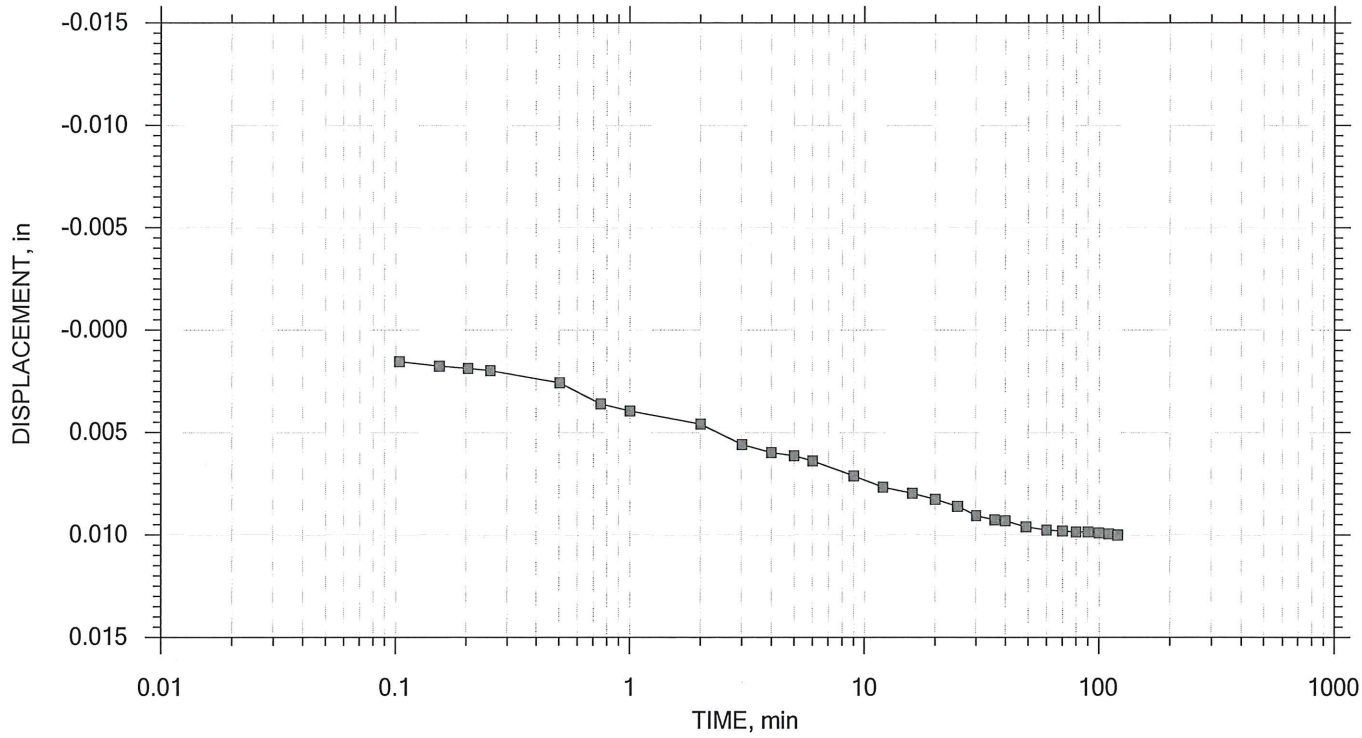
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.125	0.01000	1.23	1.00	0.000	0.00e+000	8.00e-002	0.00e+000	0.00e+000
2	0.250	0.01797	1.22	1.80	2.737	2.02e-006	6.38e-002	3.48e-004	0.00e+000
3	0.500	0.03065	1.19	3.07	1.842	2.95e-006	5.07e-002	4.03e-004	0.00e+000
4	0.750	0.03896	1.17	3.90	3.995	1.33e-006	3.32e-002	1.19e-004	0.00e+000
5	1.00	0.04742	1.15	4.74	0.000	0.00e+000	3.38e-002	0.00e+000	0.00e+000
6	1.25	0.05737	1.13	5.74	0.000	0.00e+000	3.98e-002	0.00e+000	0.00e+000
7	1.50	0.06897	1.10	6.90	0.000	0.00e+000	4.64e-002	0.00e+000	0.00e+000
8	1.75	0.07722	1.08	7.72	0.000	0.00e+000	3.30e-002	0.00e+000	0.00e+000
9	2.00	0.09225	1.05	9.23	0.000	0.00e+000	6.01e-002	0.00e+000	0.00e+000
10	4.00	0.1543	0.908	15.4	5.688	7.70e-007	3.10e-002	6.45e-005	0.00e+000
11	8.00	0.2032	0.798	20.3	2.754	1.40e-006	1.22e-002	4.60e-005	0.00e+000
12	16.0	0.2473	0.699	24.7	2.055	1.66e-006	5.51e-003	2.47e-005	0.00e+000
13	32.0	0.2870	0.609	28.7	1.560	1.96e-006	2.48e-003	1.31e-005	0.00e+000
14	8.00	0.2781	0.629	27.8	0.000	0.00e+000	3.68e-004	0.00e+000	0.00e+000
15	2.00	0.2639	0.661	26.4	0.000	0.00e+000	2.38e-003	0.00e+000	0.00e+000
16	0.500	0.2465	0.700	24.7	9.633	3.28e-007	1.16e-002	1.02e-005	0.00e+000

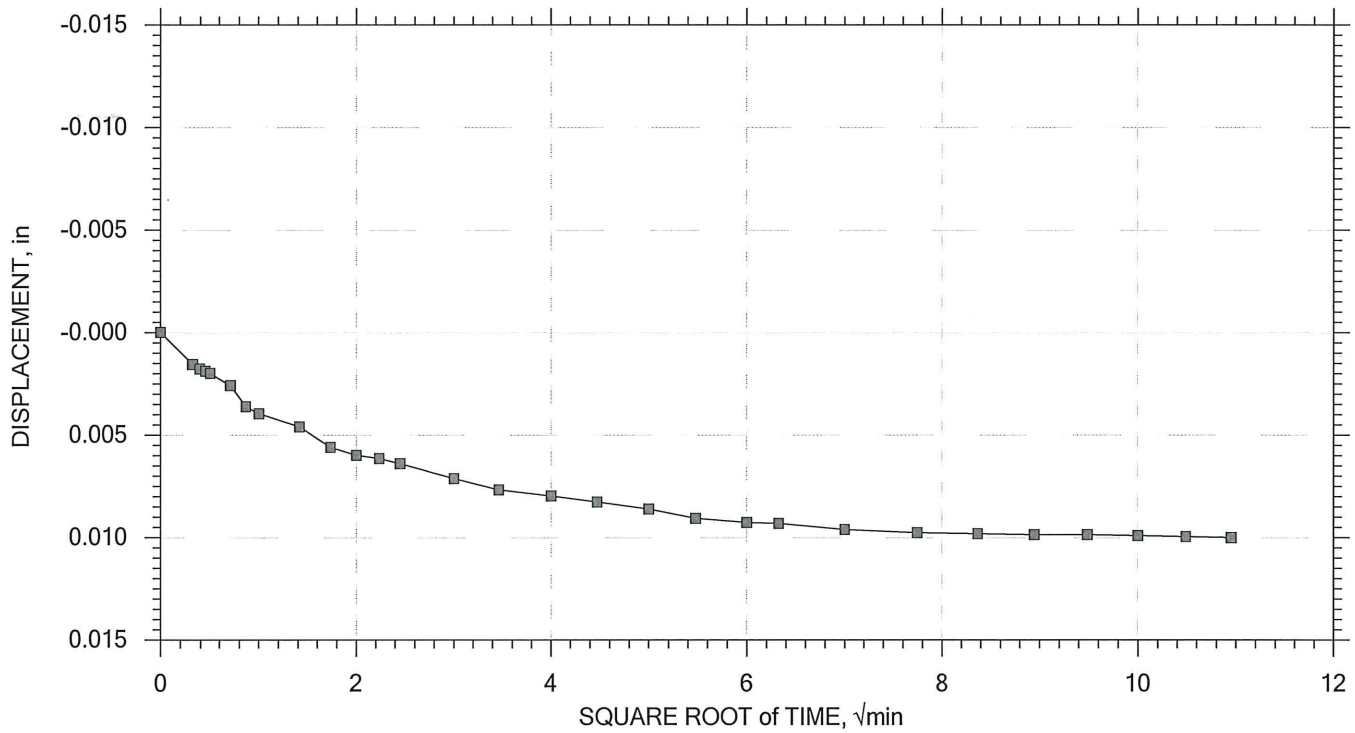
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 1 of 16

Stress: 0.125 tsf





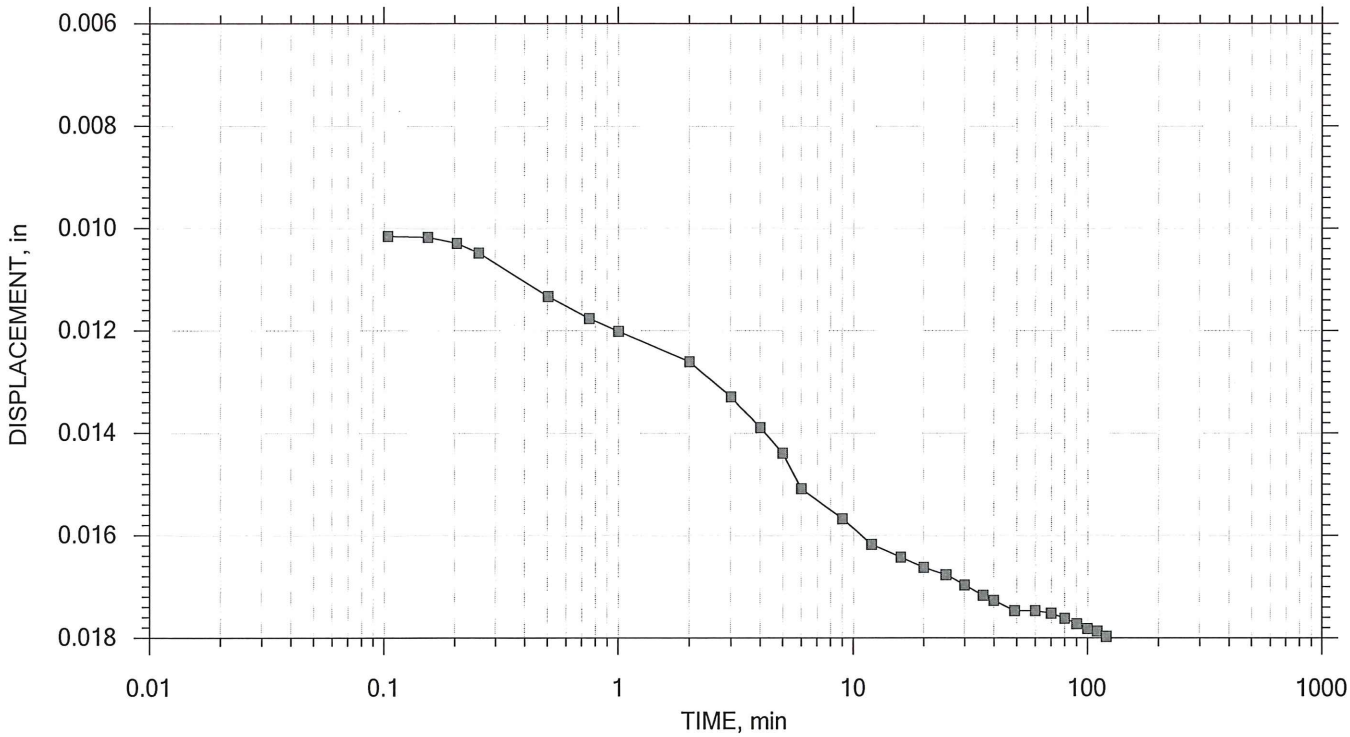
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

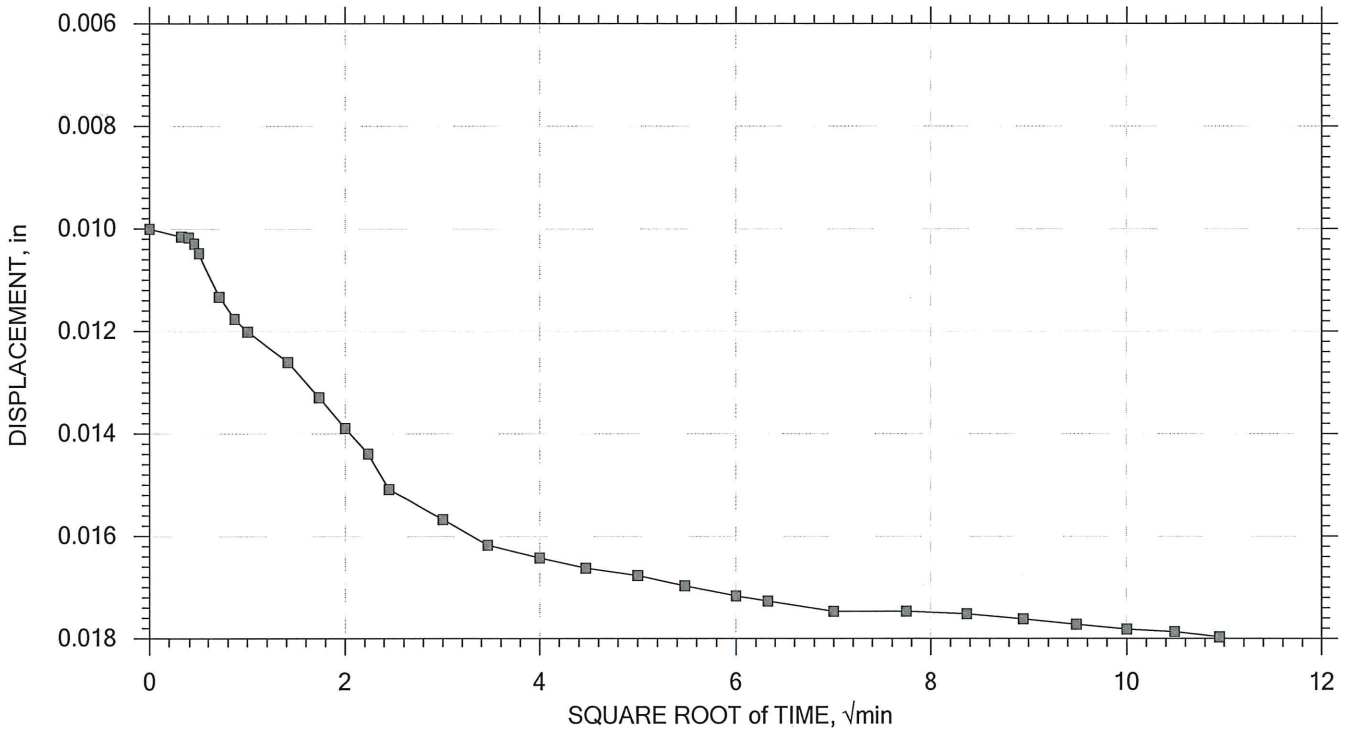
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 2 of 16

Stress: 0.25 tsf





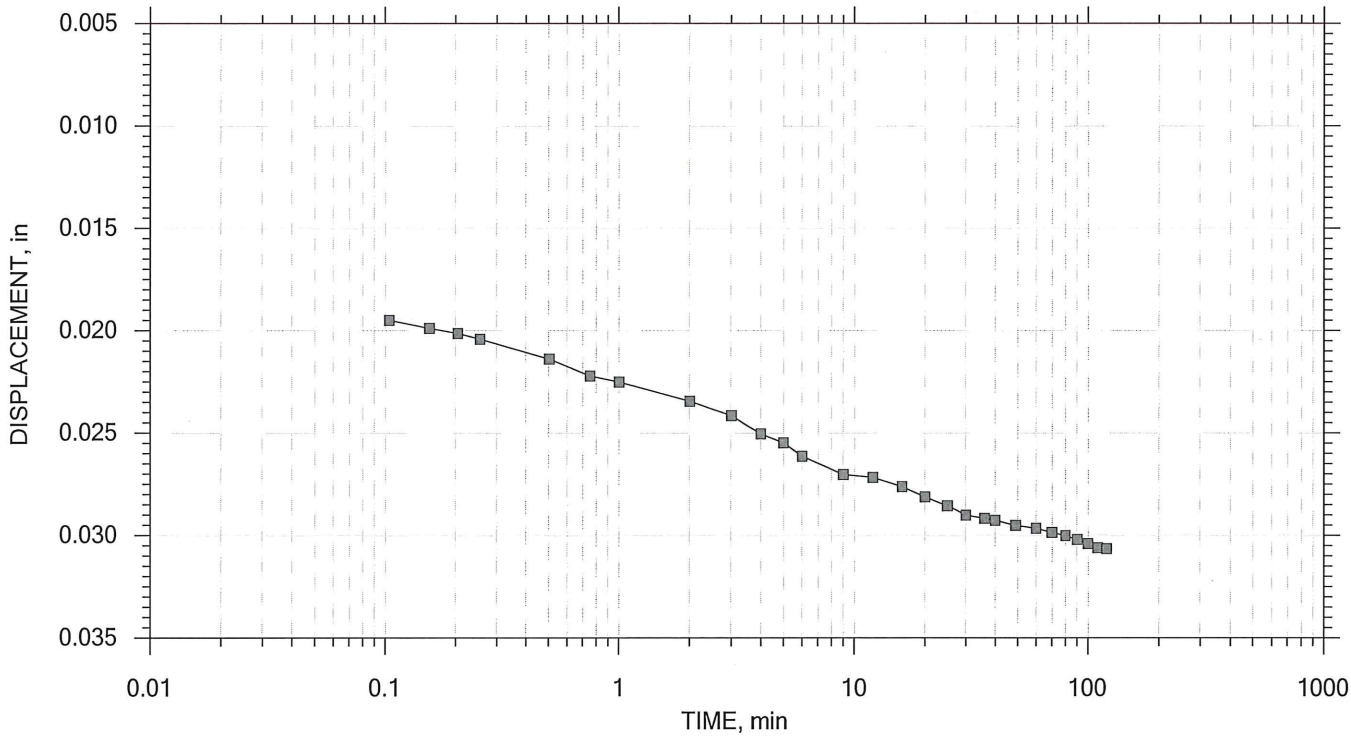
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

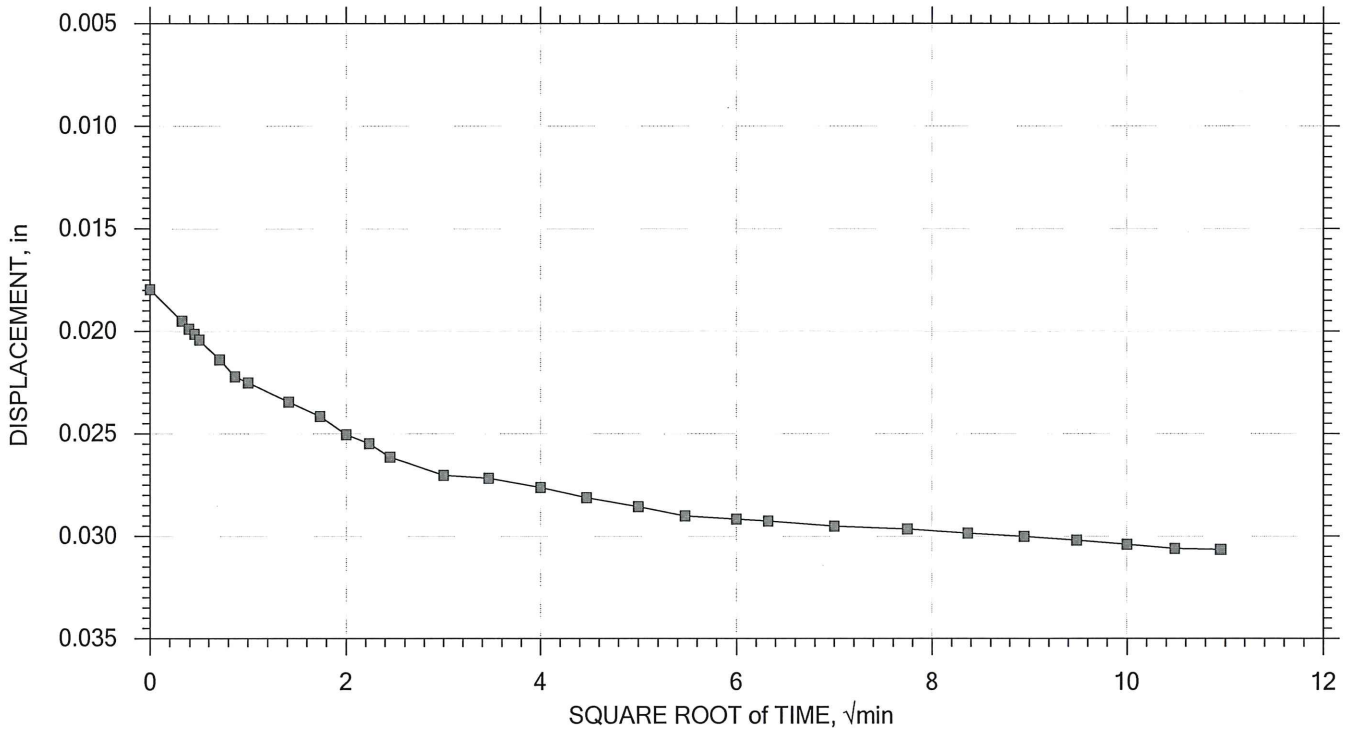
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 3 of 16

Stress: 0.5 tsf





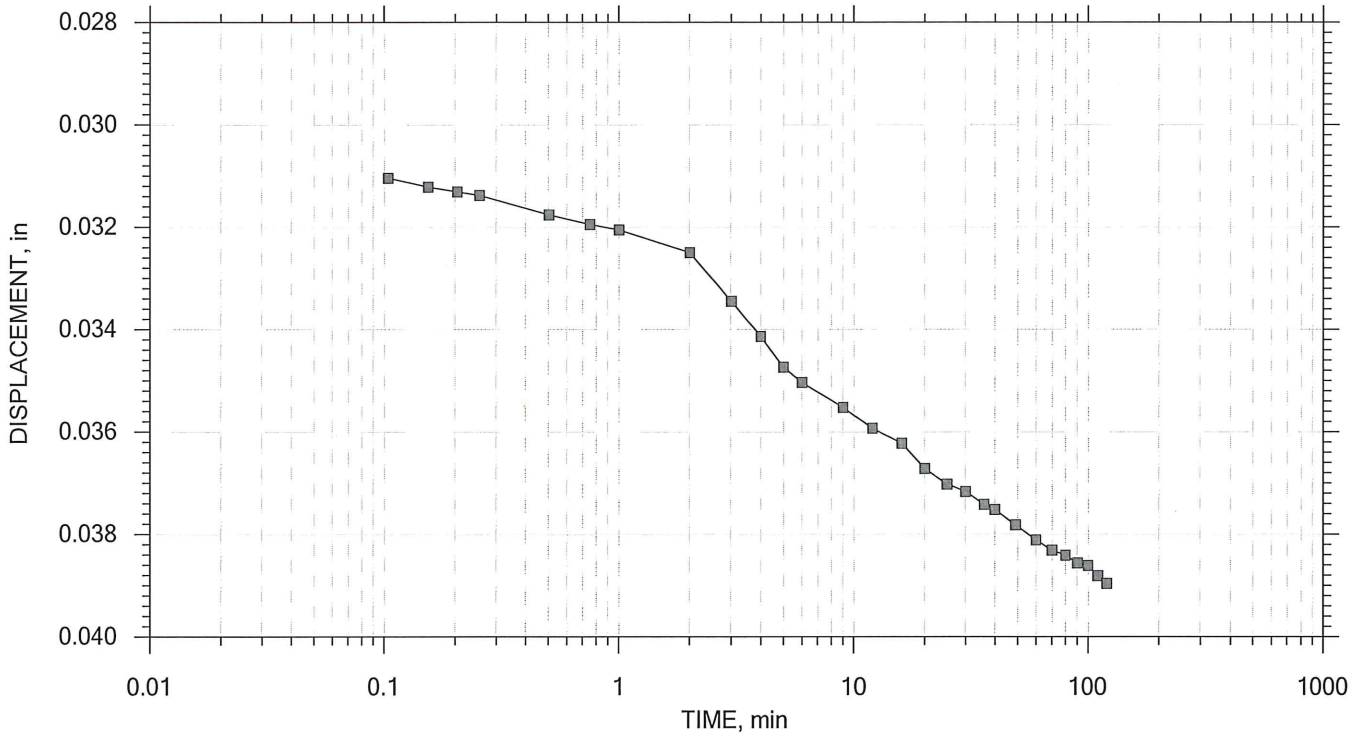
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

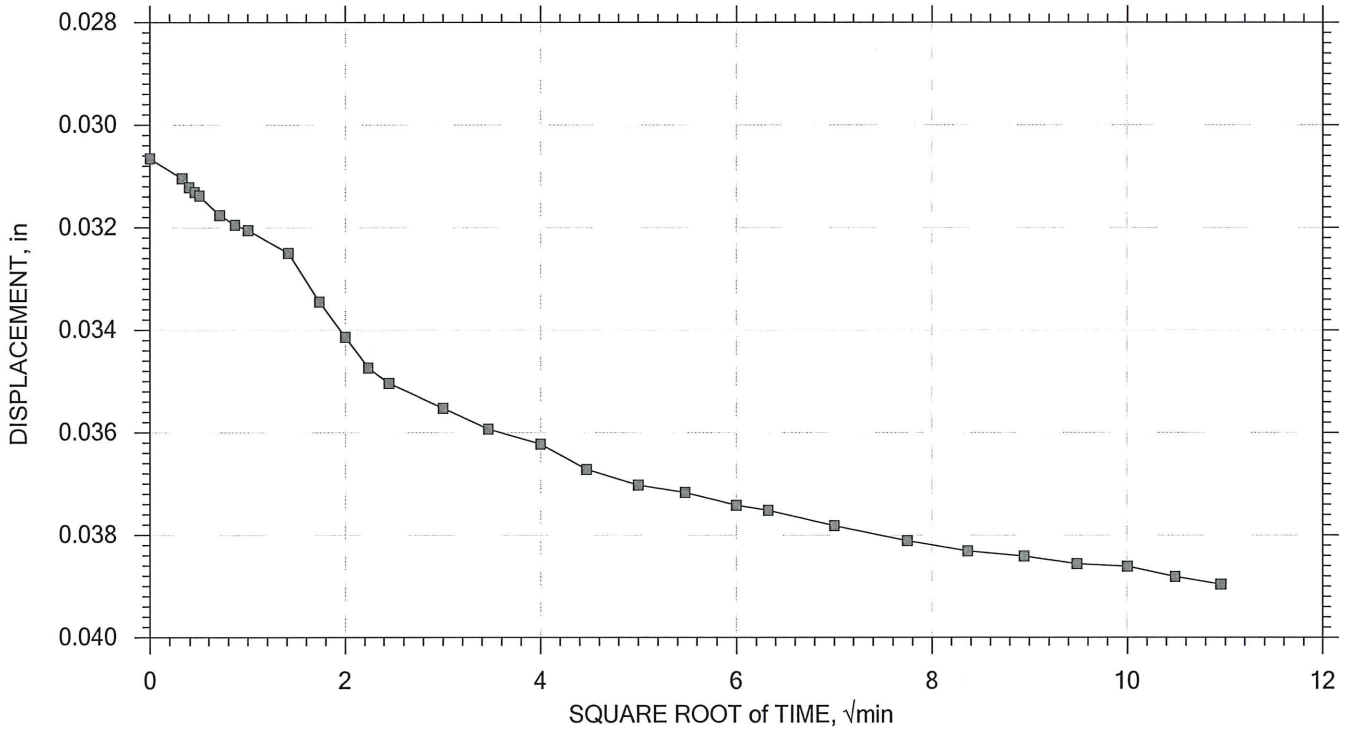
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 4 of 16

Stress: 0.75 tsf





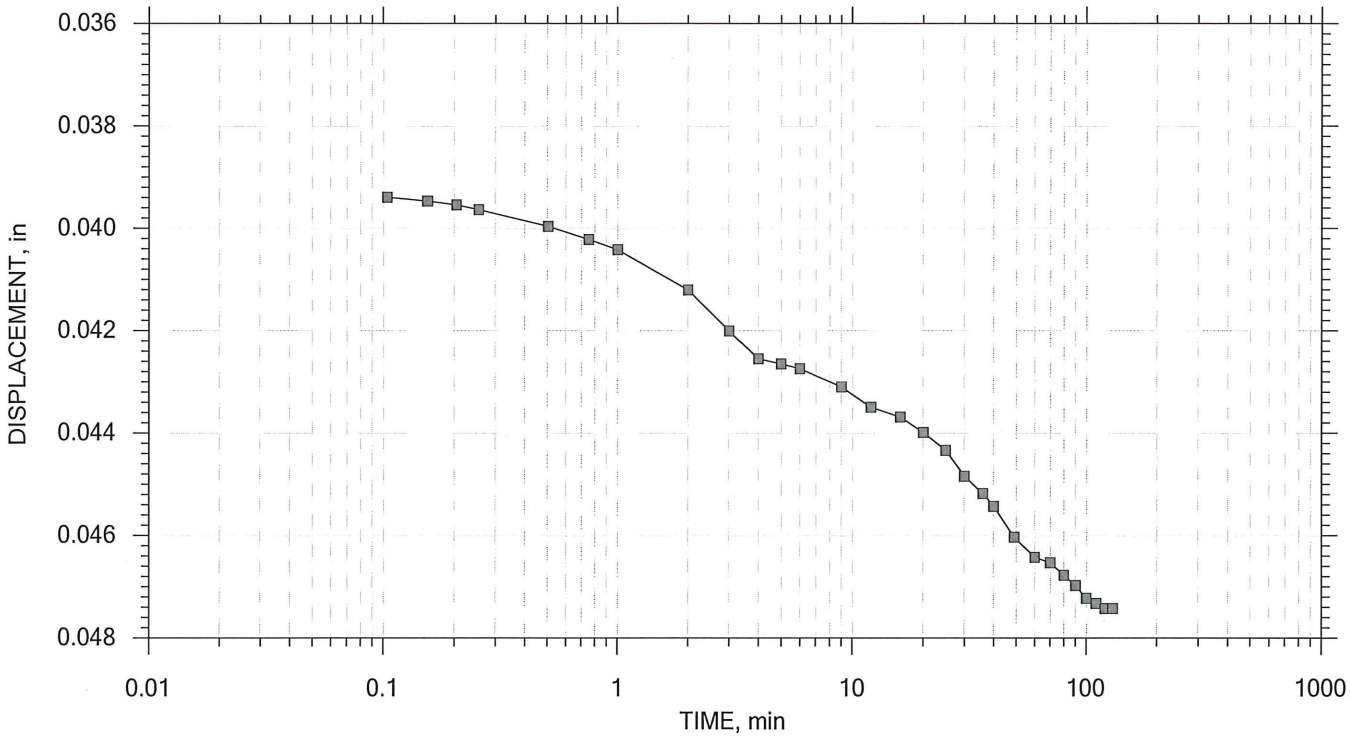
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

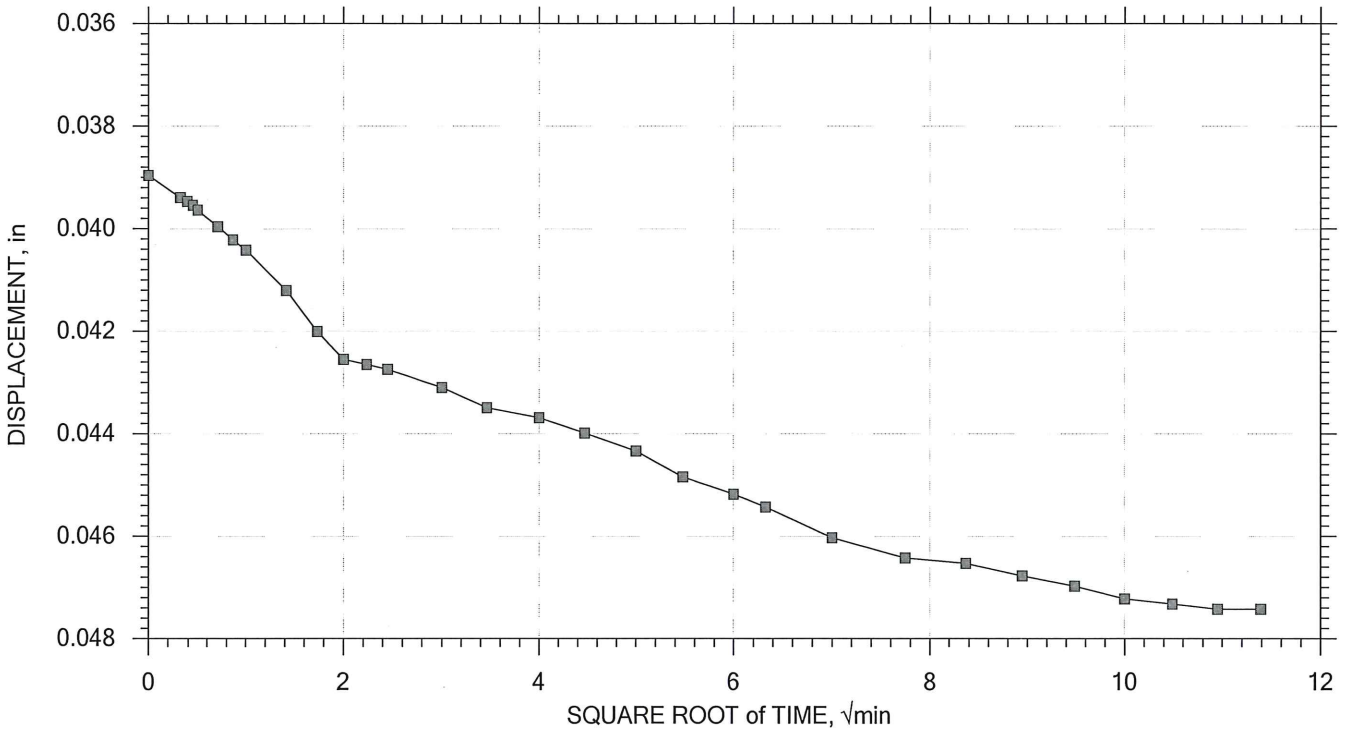
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 5 of 16

Stress: 1 tsf





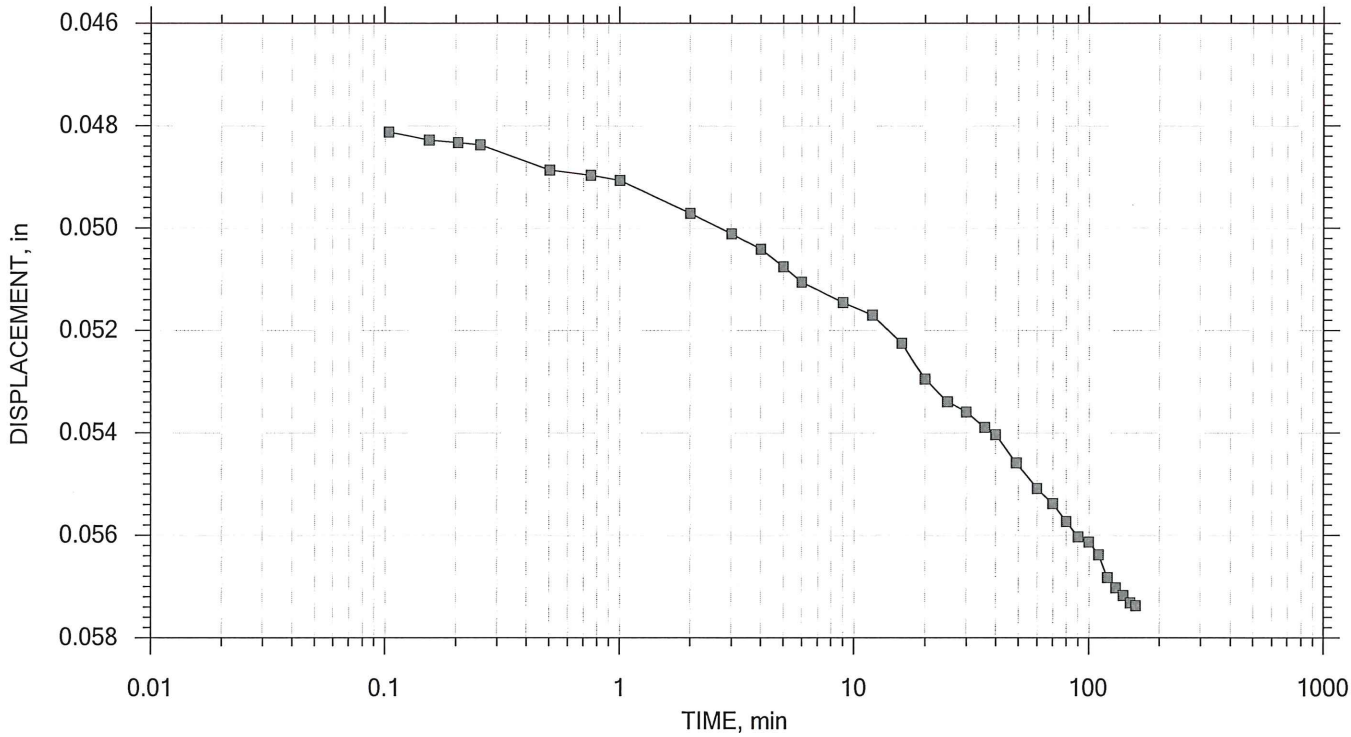
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

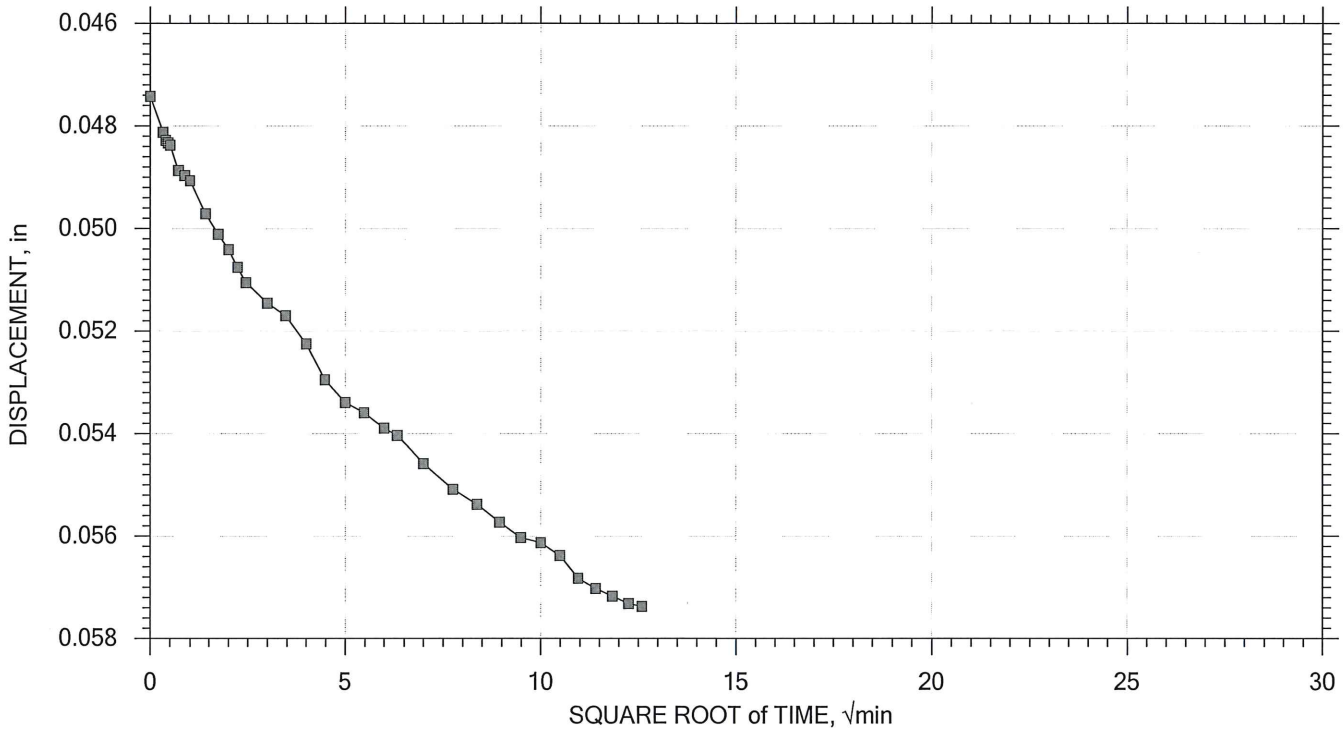
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 6 of 16

Stress: 1.25 tsf





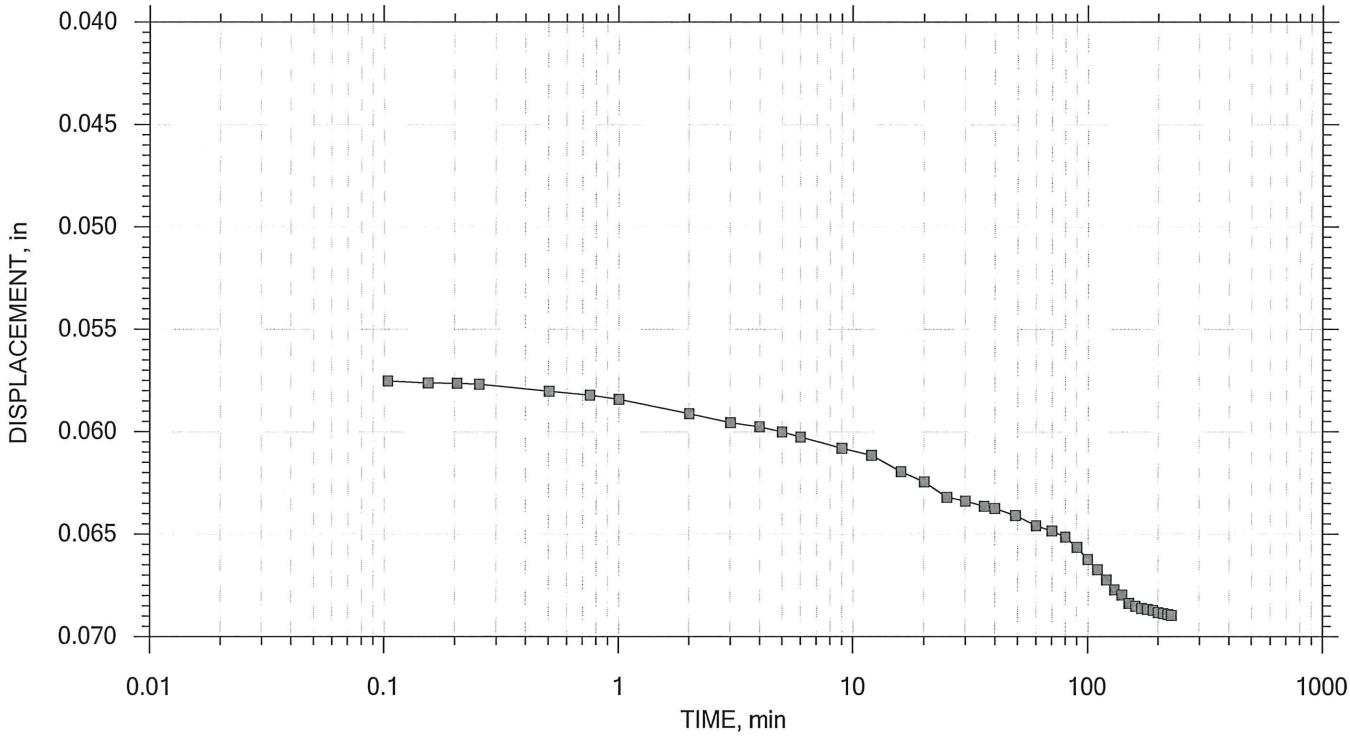
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

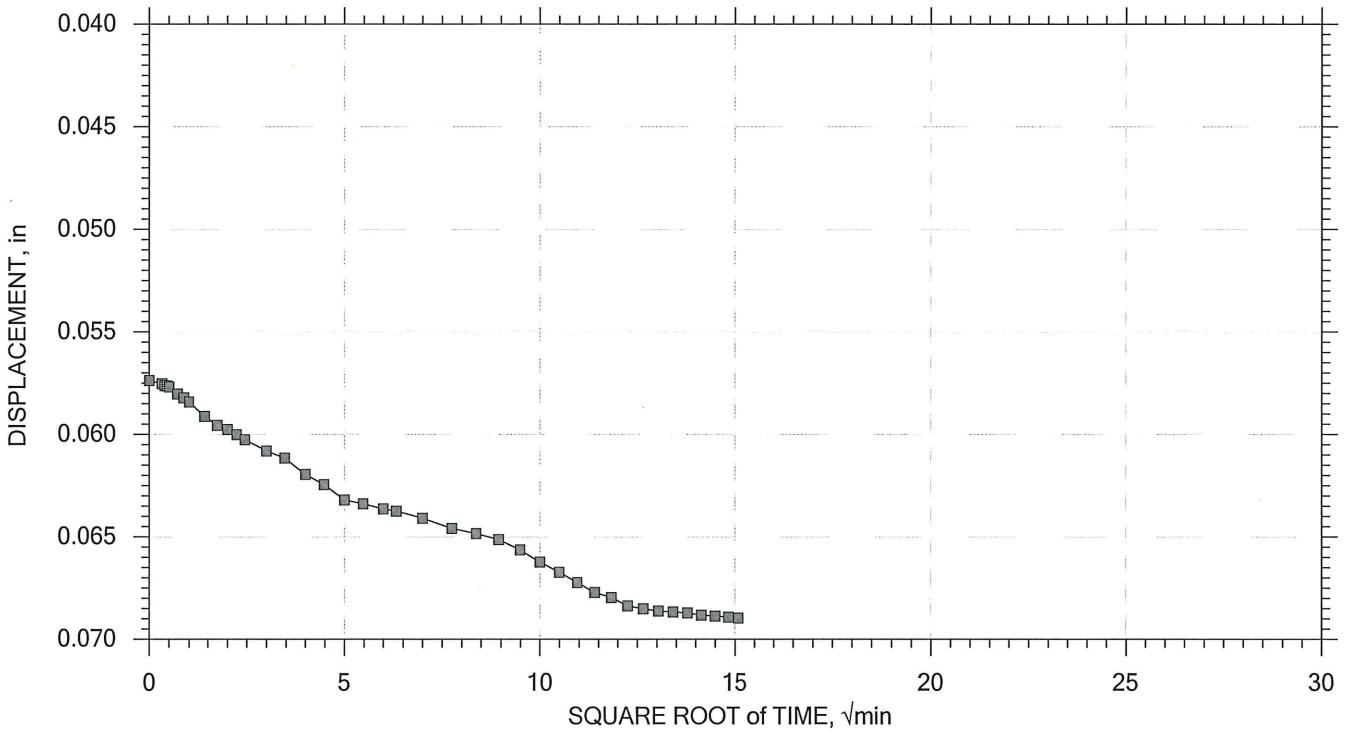
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 7 of 16

Stress: 1.5 tsf





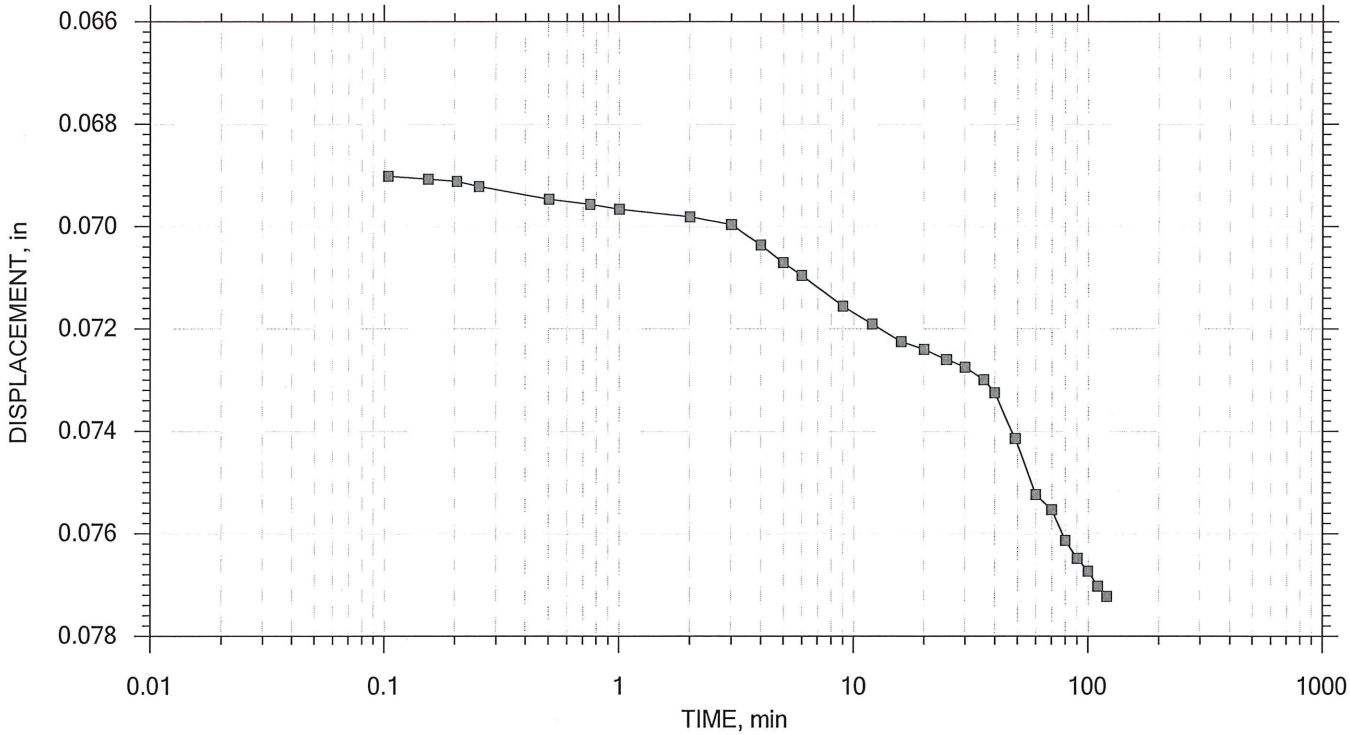
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

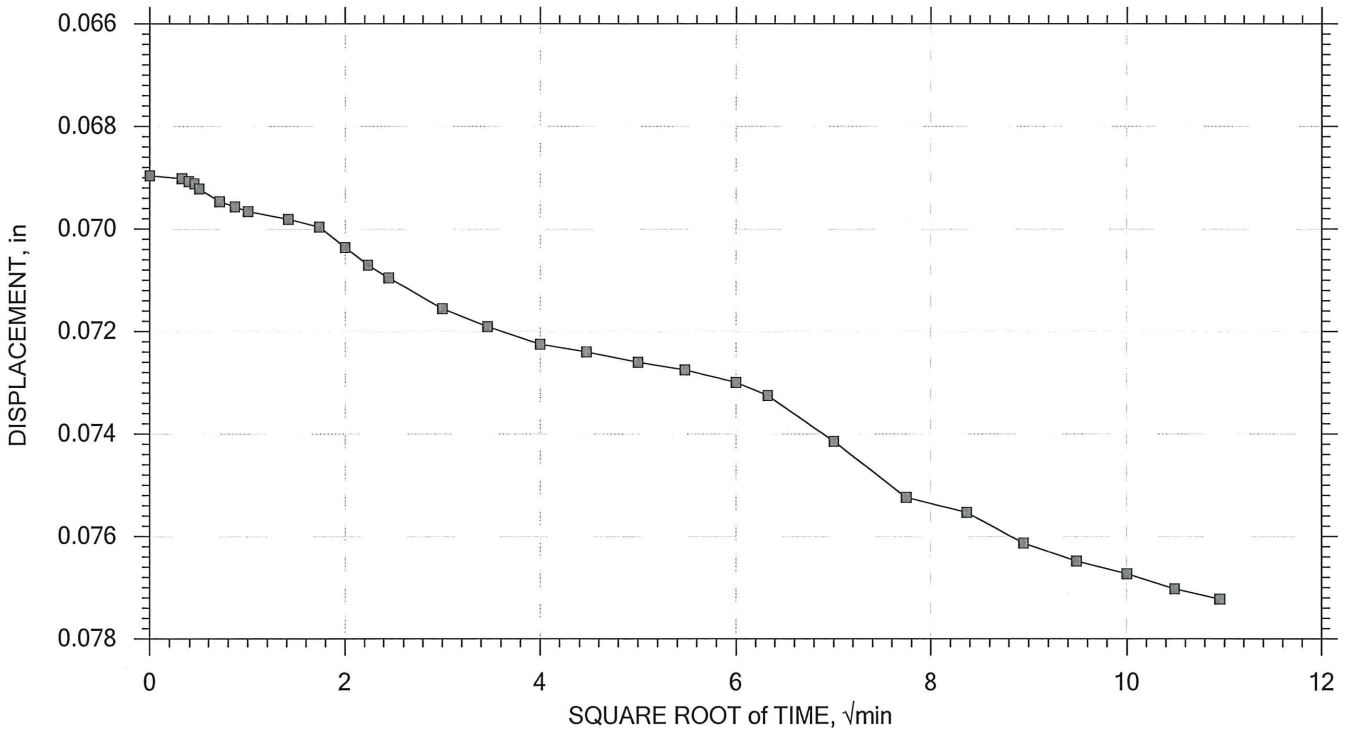
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 8 of 16

Stress: 1.75 tsf





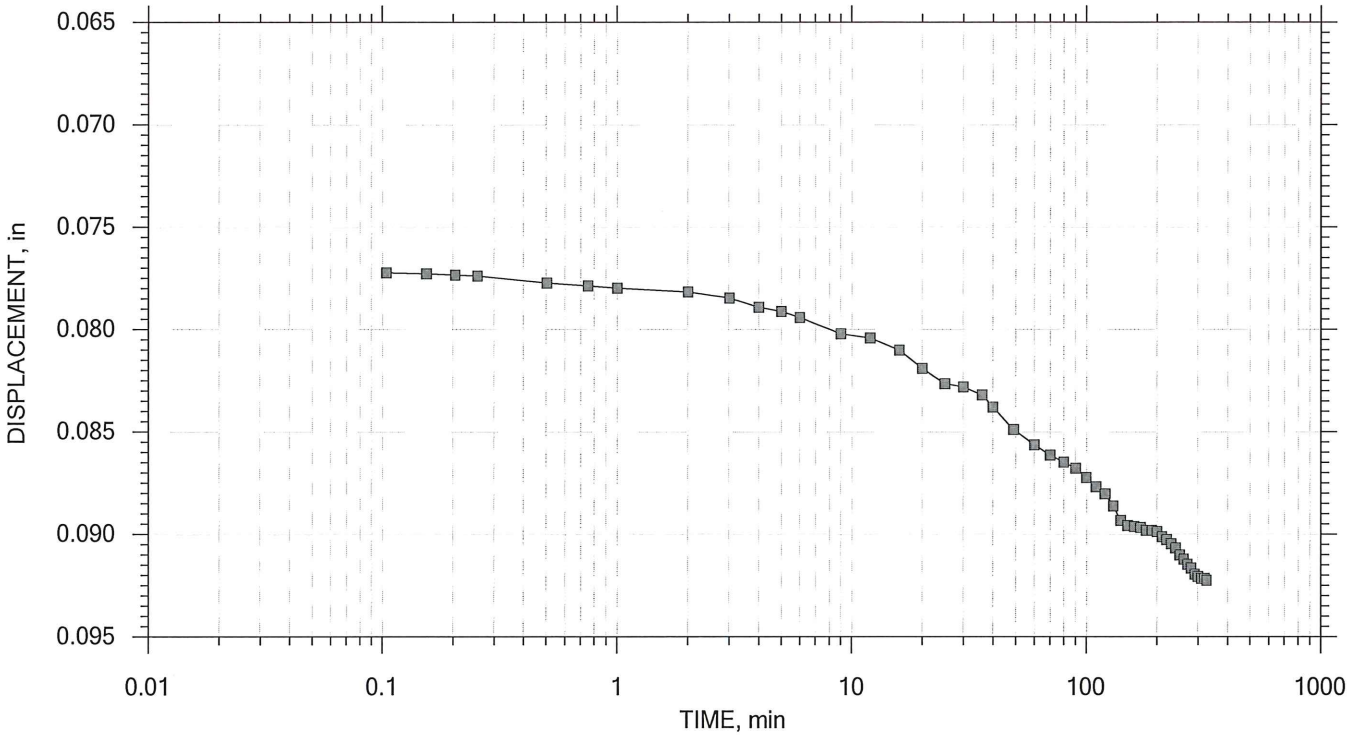
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

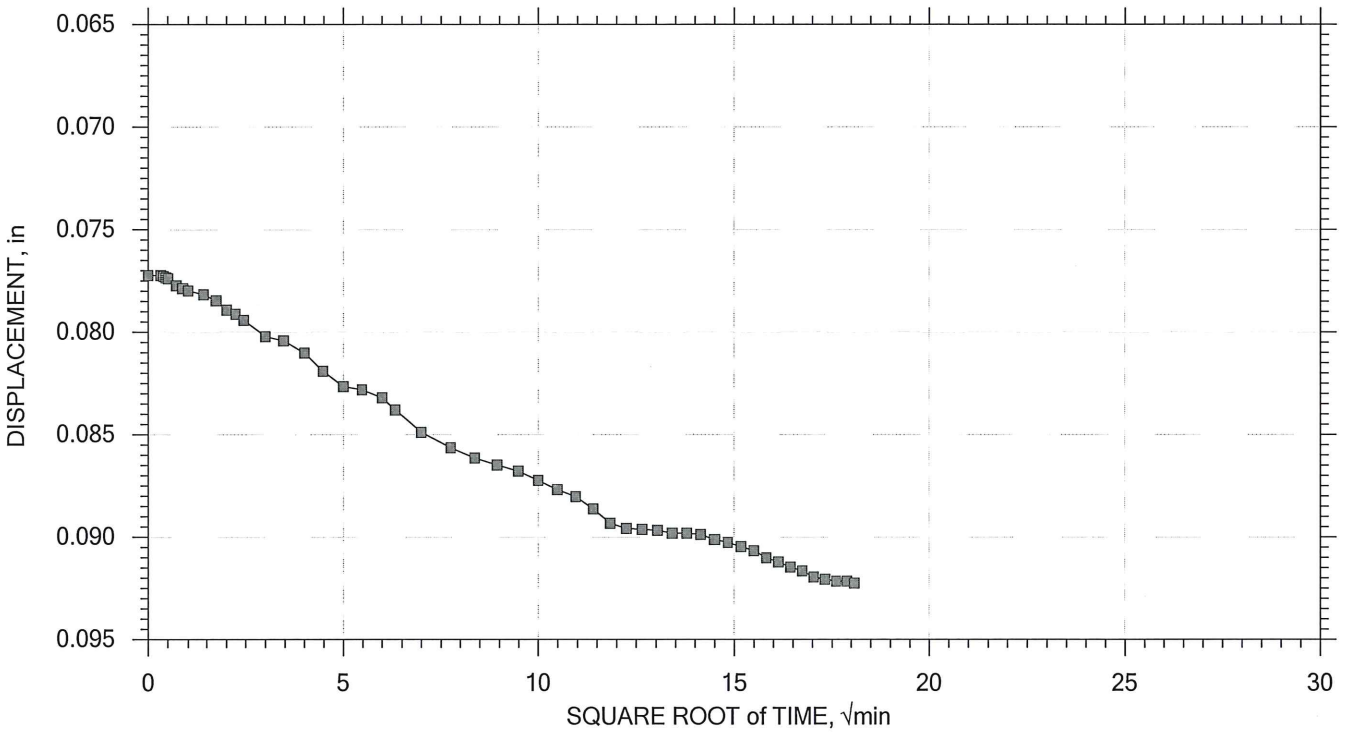
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 9 of 16

Stress: 2 tsf





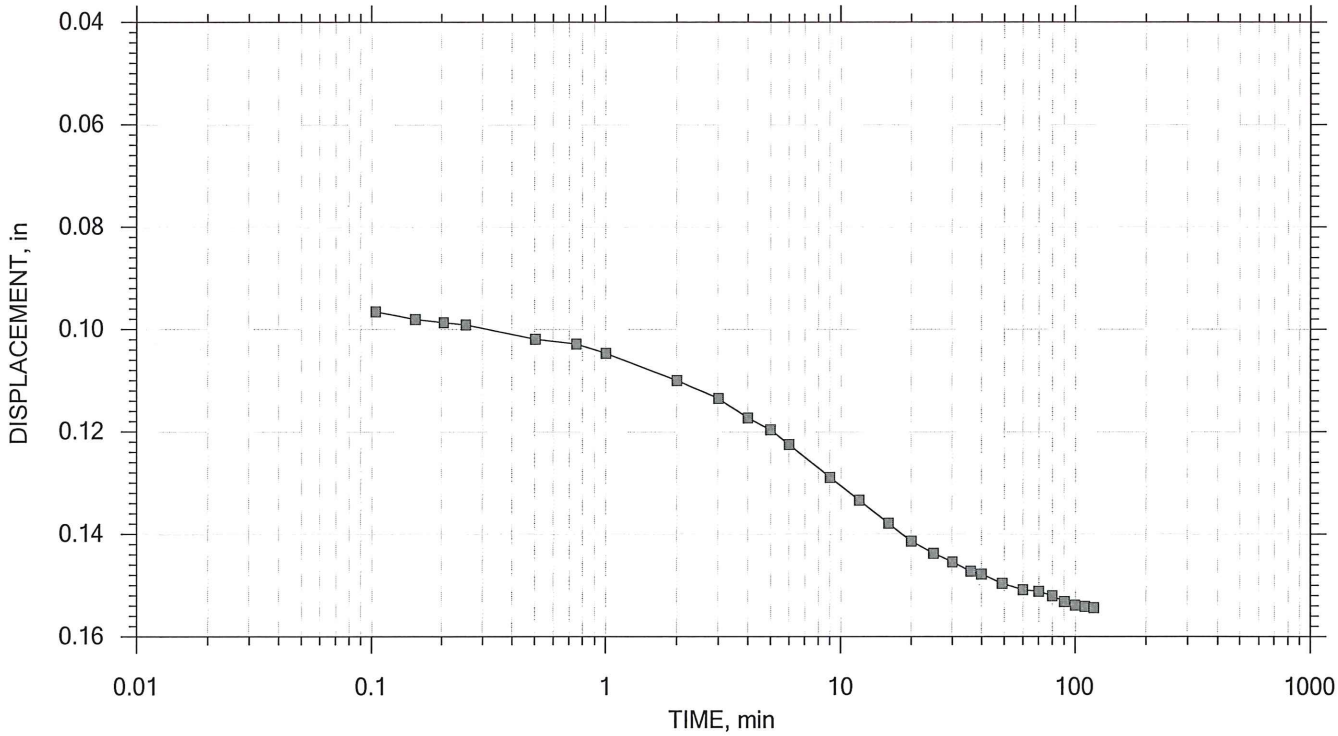
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

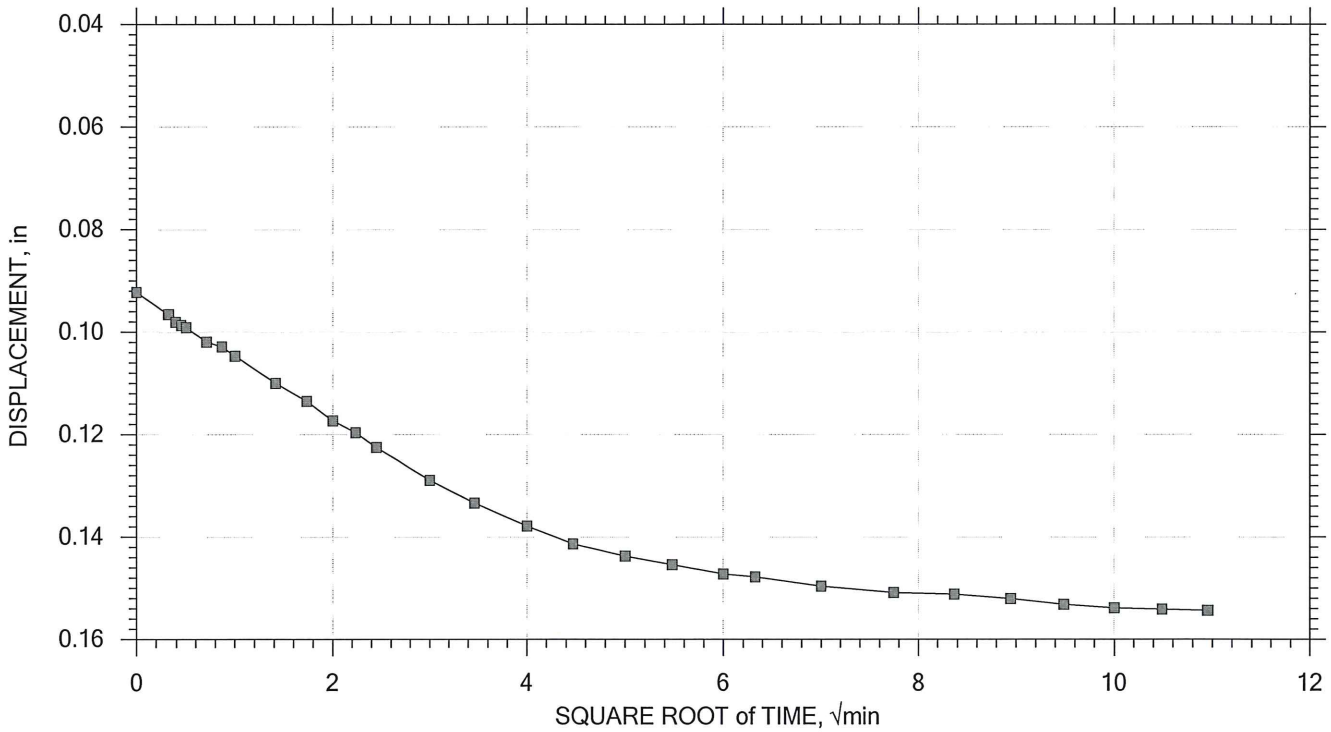
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 10 of 16

Stress: 4 tsf





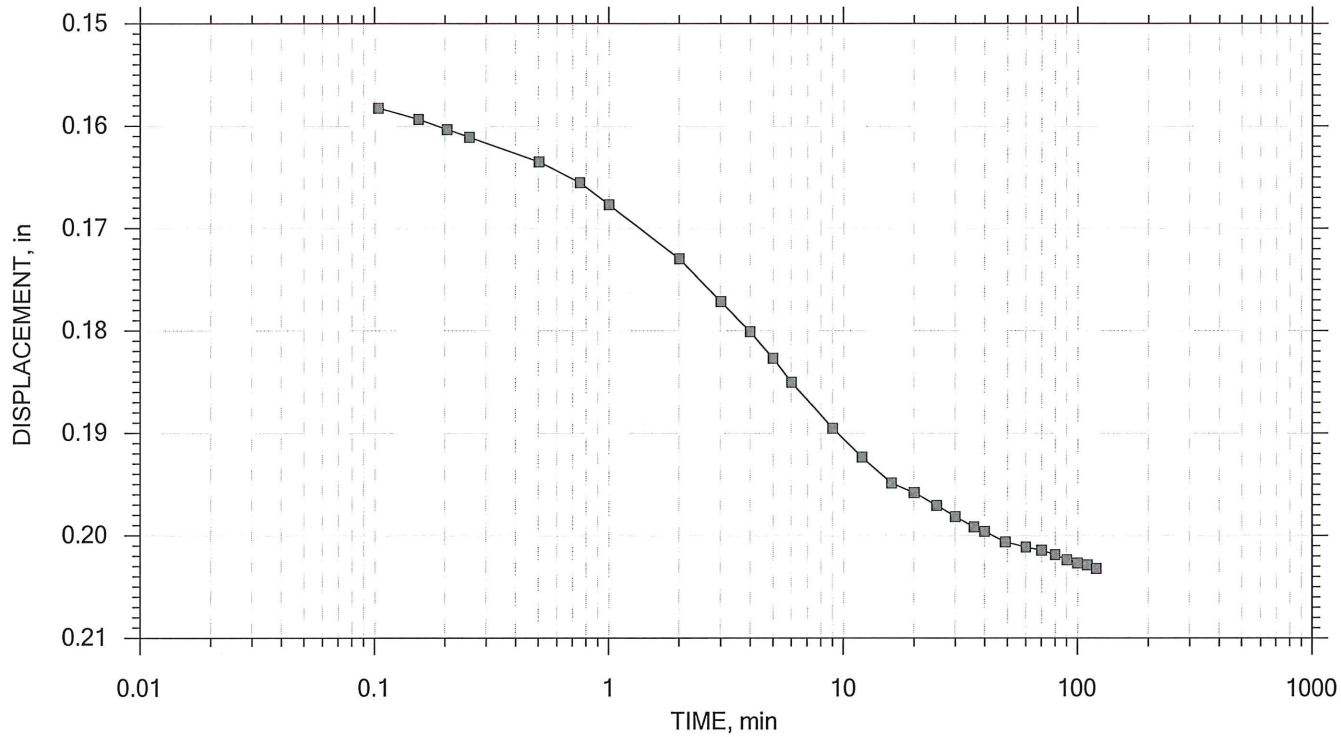
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

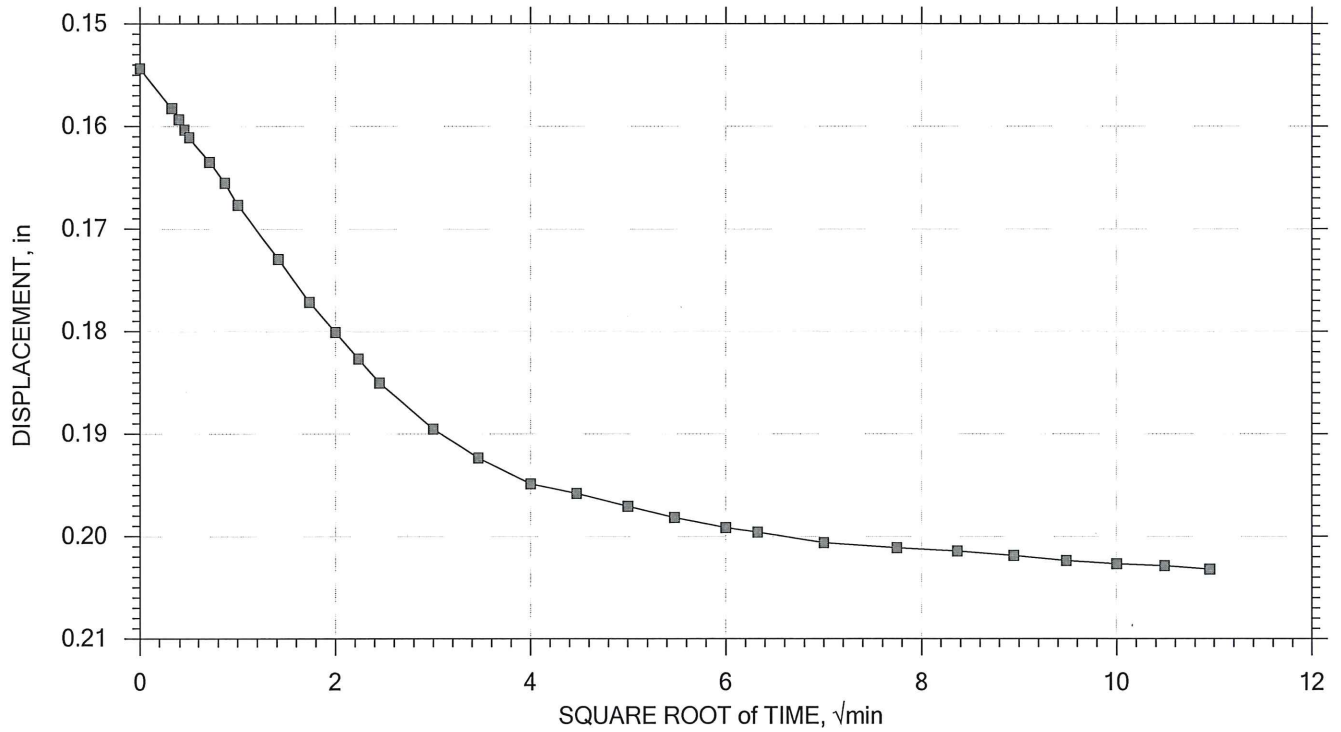
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 11 of 16

Stress: 8 tsf





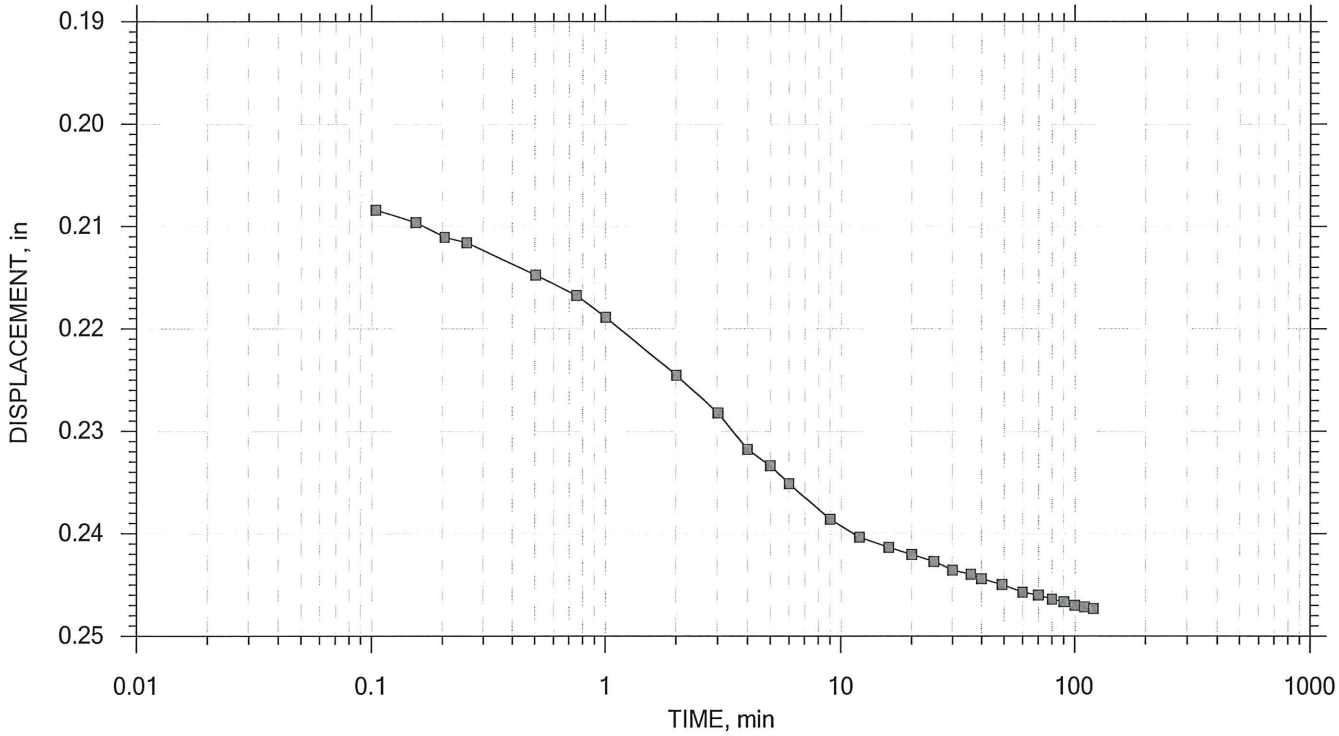
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

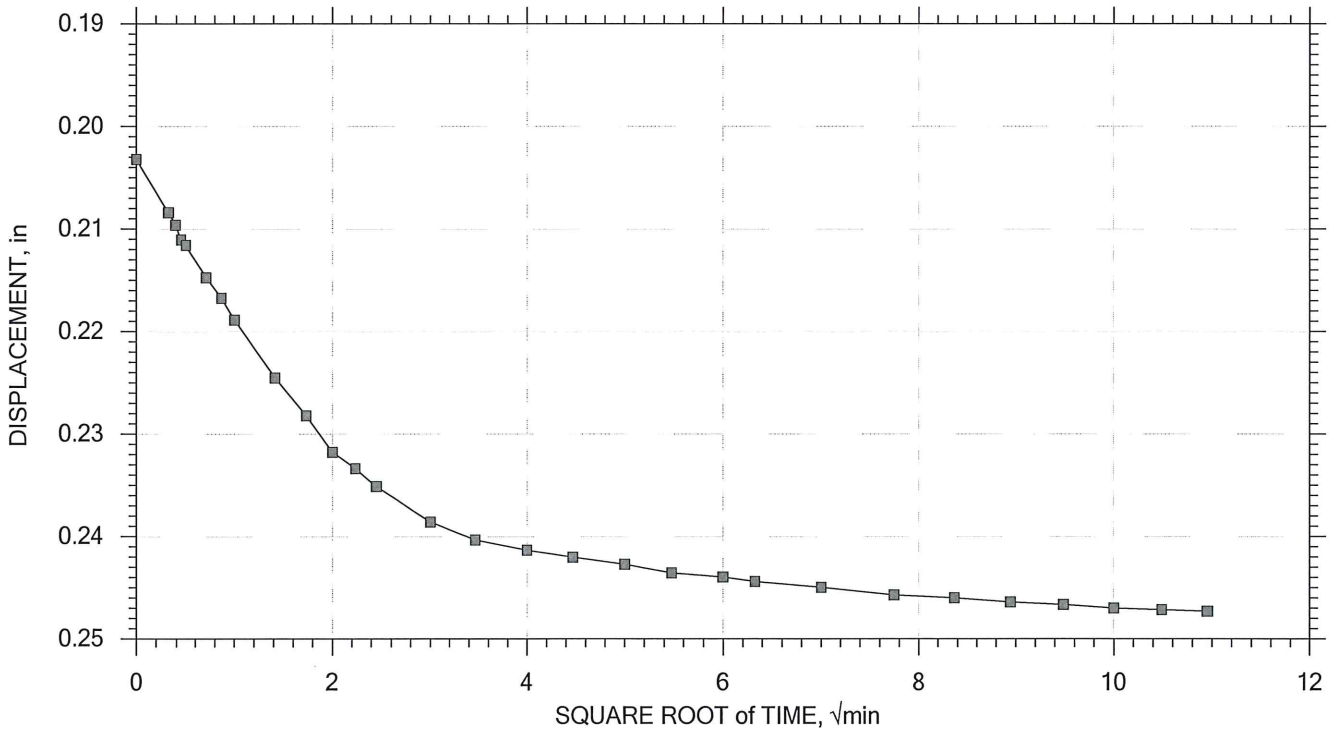
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 12 of 16

Stress: 16 tsf





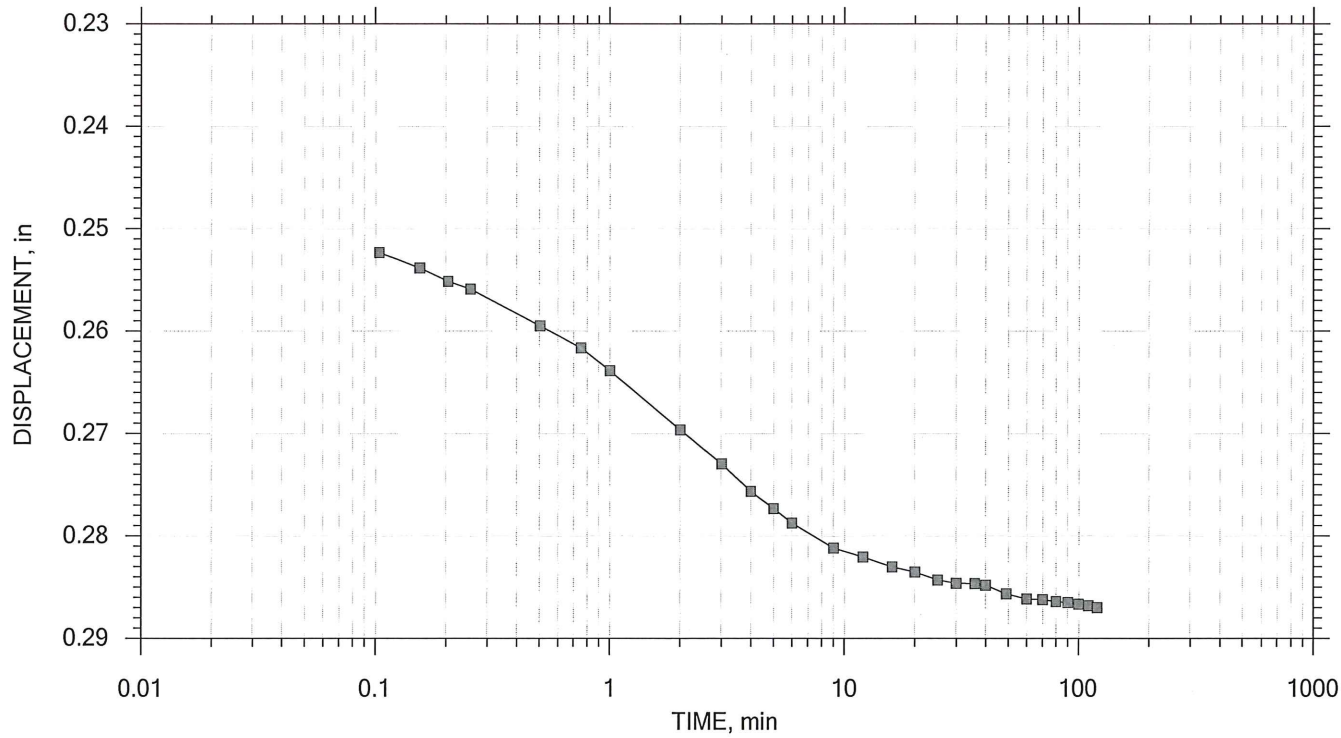
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

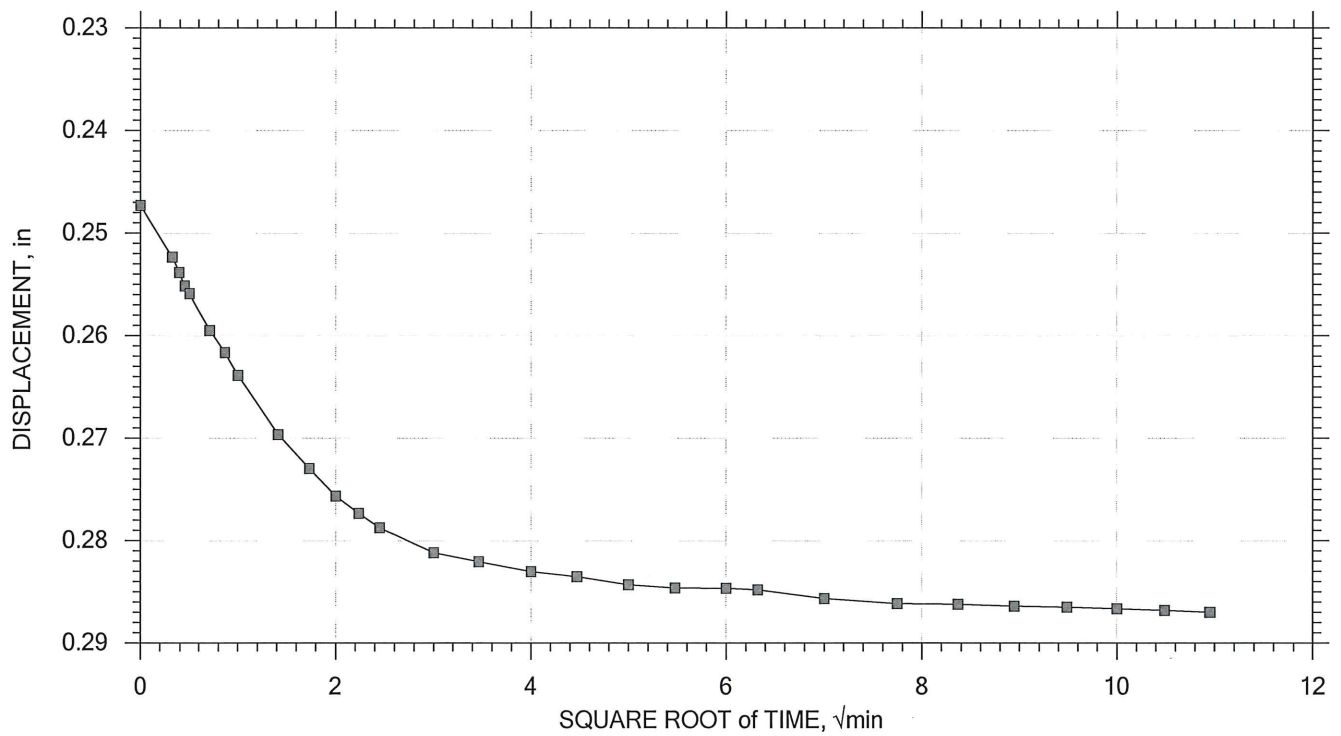
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 13 of 16

Stress: 32 tsf





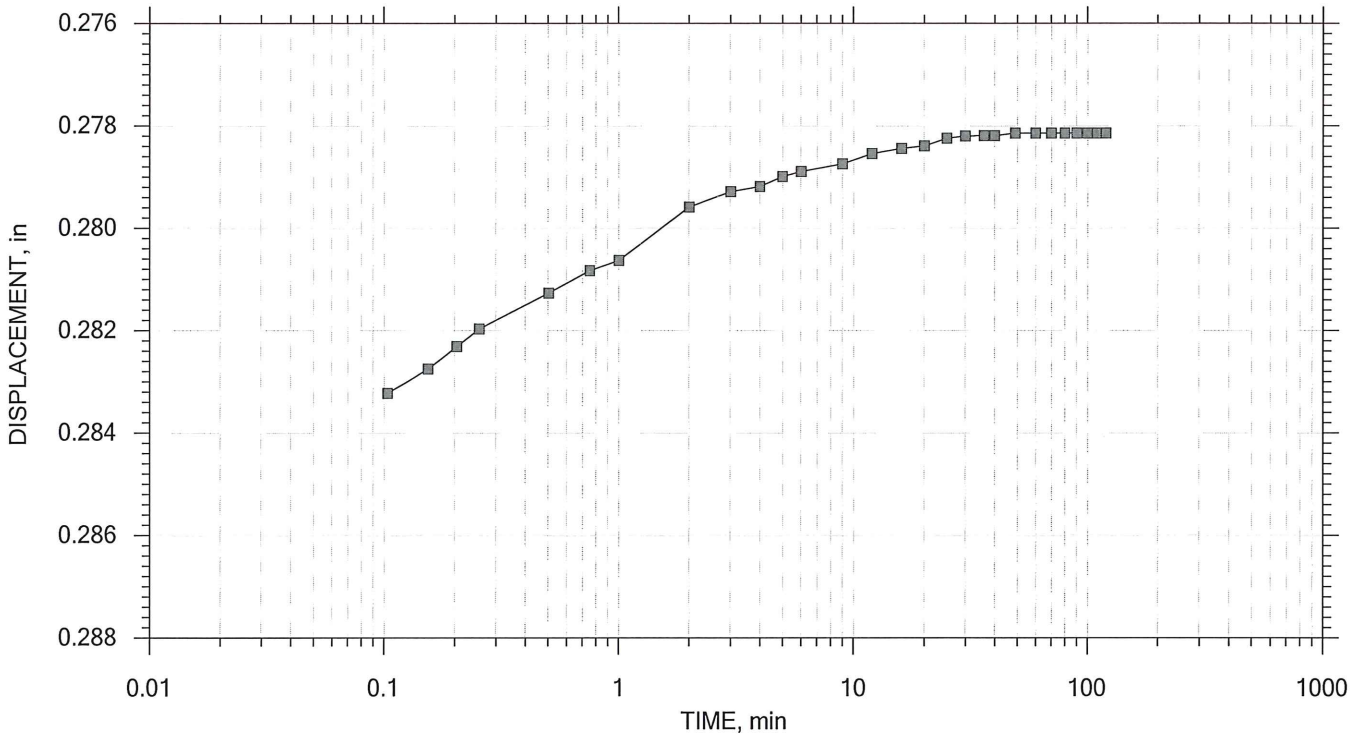
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

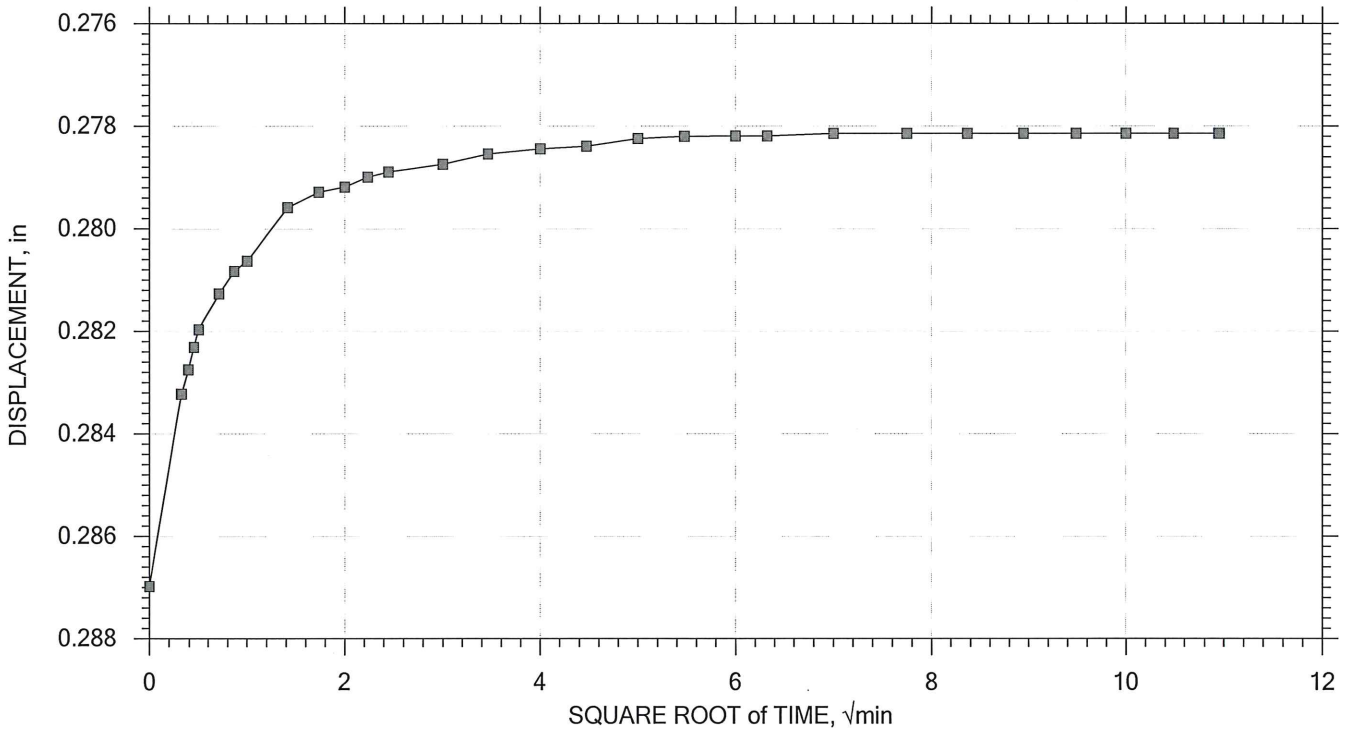
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 14 of 16

Stress: 8 tsf





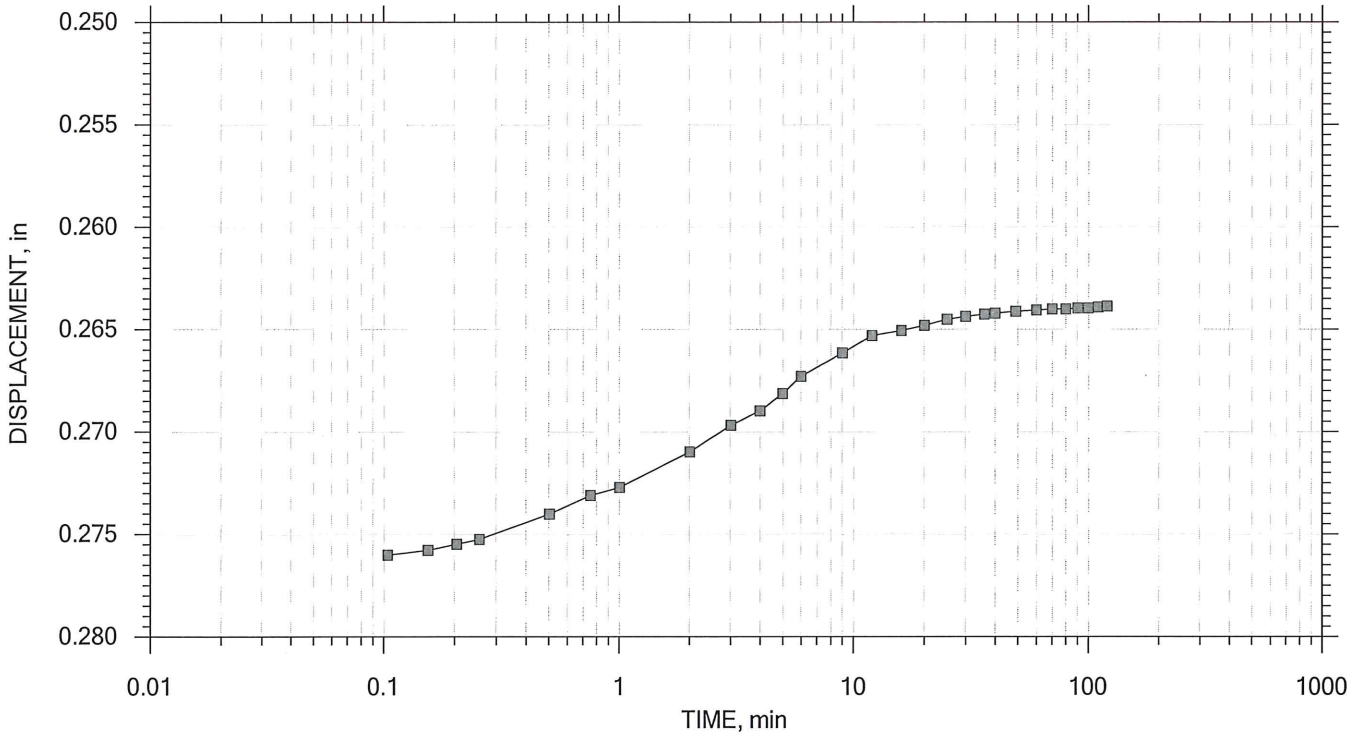
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

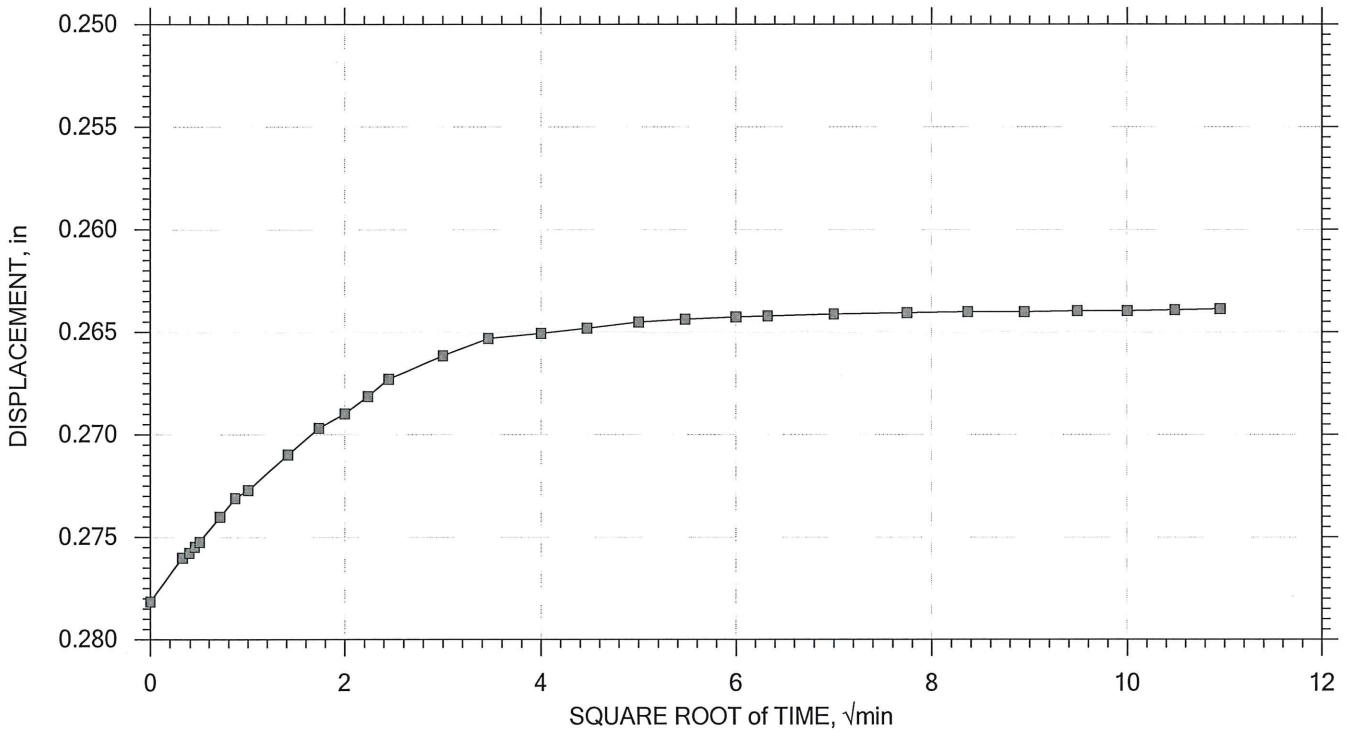
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 15 of 16

Stress: 2 tsf





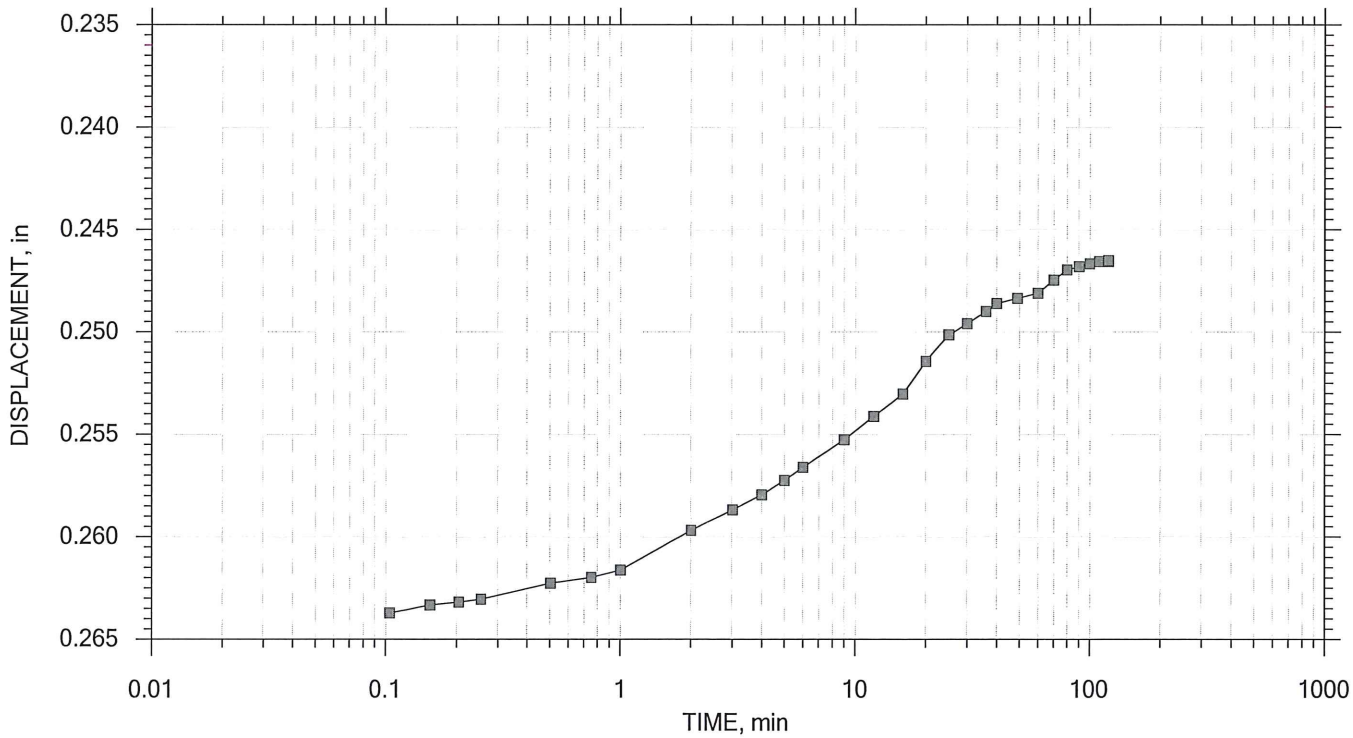
GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

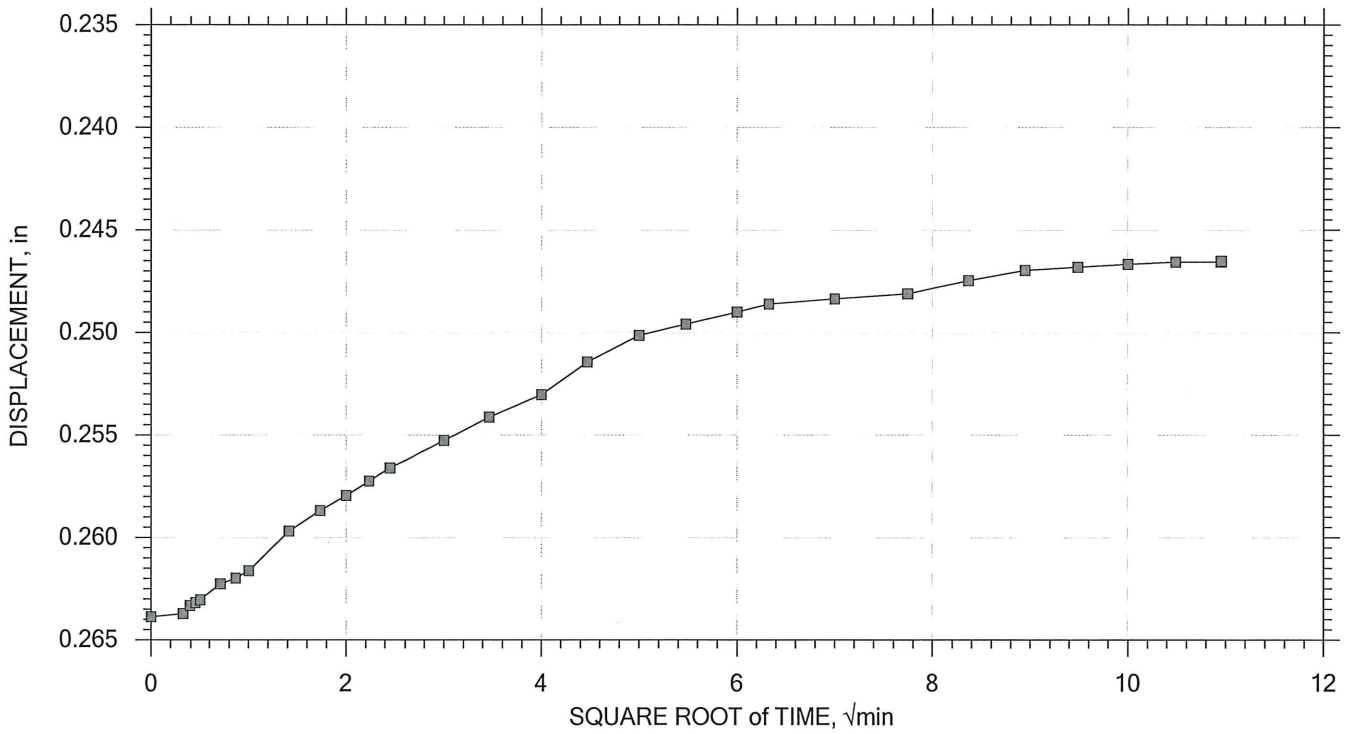
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 16 of 16

Stress: 0.5 tsf

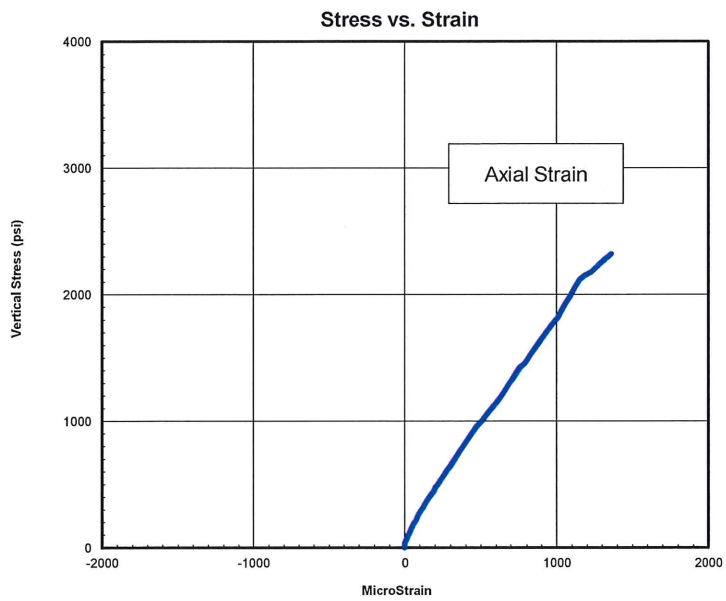




GeoTesting EXPRESS	Project: U.S. Rt. 1 Viaduct Bridge Repl.	Location: Bath, ME	Project No.: GTX-300397
	Boring No.: BB-BROV-103	Tested By: md	Checked By: jdt
	Sample No.: U3	Test Date: 04/02/13	Test No.: IP-2
	Depth: 41.0-43.0 ft	Sample Type: intact	Elevation: ---
	Description: Wet, gray clay		
	Remarks: System Q		

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Viaduct Bridge Replacement
Project Location:	Bath, ME
GTX #:	300397
Test Date:	4/9/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-BROV-101
Sample ID:	R1
Depth, ft:	14.61-14.98
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 2,321 psi

The lateral strain gauges failed and did not produce valid data.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
200-900	1,860,000	---
900-1500	1,630,000	---
1500-2100	1,740,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Haley & Aldrich, Inc.	Test Date:	4/8/2013
Project Name:	U.S. Route 1 Viaduct Bridge Replacement	Tested By:	daa
Project Location:	Bath, ME	Checked By:	mpd
GTX #:	300397		
Boring ID:	BB-BROV-101		
Sample ID:	R1		
Depth:	14.61-14.98 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.39	4.39	4.39
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	625.66		
Bulk Density, lb/ft^3	174	**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	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00020
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00070

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00080

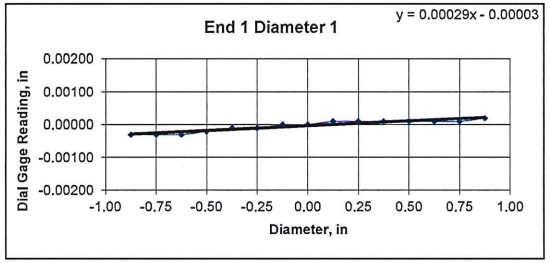
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00020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

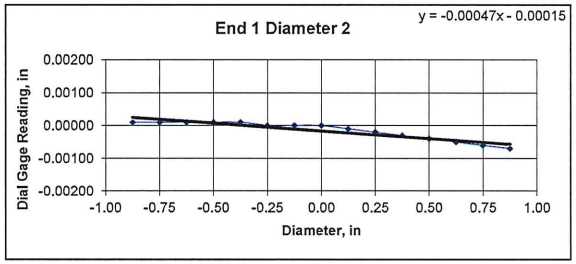
Difference between max and min readings, in:

0° = 0.0004 90° = 0.0008

Maximum difference must be < 0.0020 in. Difference = ± 0.00040

Flatness Tolerance Met? **YES**





DIAMETER 1

End 1:

Slope of Best Fit Line 0.00029

Angle of Best Fit Line: 0.01662

End 2:

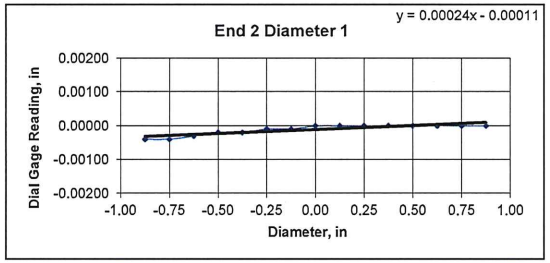
Slope of Best Fit Line 0.00024

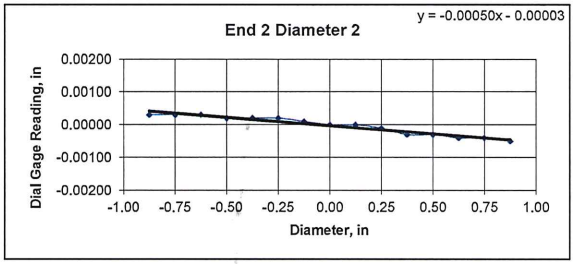
Angle of Best Fit Line: 0.01375

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? **YES**

Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line -0.00047

Angle of Best Fit Line: -0.02693

End 2:

Slope of Best Fit Line -0.00050

Angle of Best Fit Line: -0.02865

Maximum Angular Difference: 0.00172

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	0.00050	1.990	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

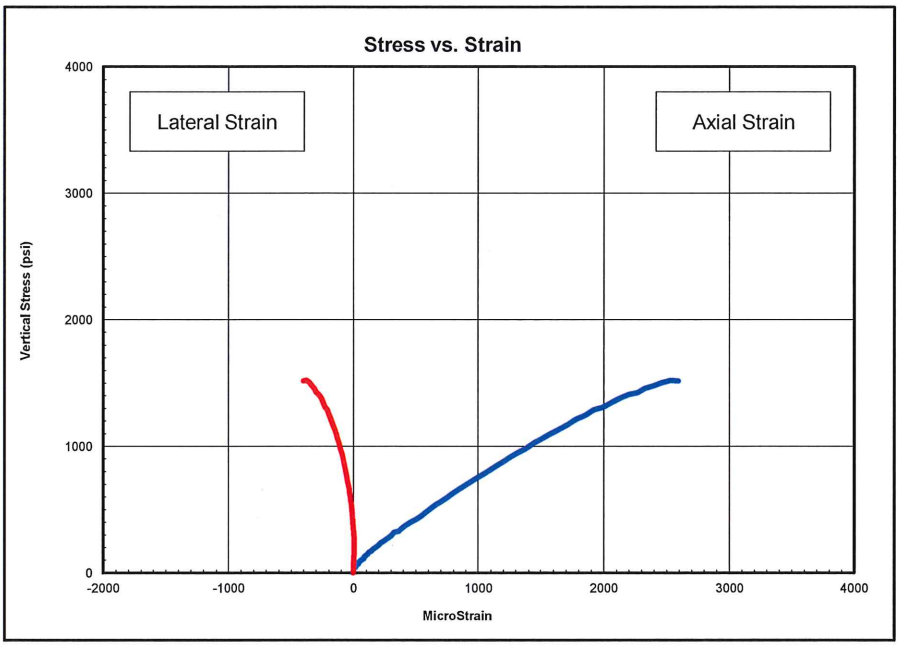
Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Viaduct Bridge Replacement
Project Location:	Bath, ME
GTX #:	300397
Test Date:	4/9/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-BROV-101
Sample ID:	R1
Depth, ft:	14.61-14.98

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Viaduct Bridge Replacement
Project Location:	Bath, ME
GTX #:	300397
Test Date:	4/9/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-BROV-102
Sample ID:	R1
Depth, ft:	13.98-14.35
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 1,522 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
150-600	685,000	0.05
600-950	615,000	0.13
950-1350	532,000	0.20

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	4/8/2013
Project Name:	U.S. Route 1 Viaduct Bridge Replacement	Tested By:	daa
Project Location:	Bath, ME	Checked By:	mpd
GTX #:	300397		
Boring ID:	BB-BROV-102		
Sample ID:	R1		
Depth:	13.98-14.35 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.39	4.39	4.39
Specimen Diameter, in:	1.98	1.99	1.99
Specimen Mass, g:	619.99		
Bulk Density, lb/ft^3	173		
Length to Diameter Ratio:	2.2		

Minimum Diameter Tolerence Met? YES

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	-0.00090	-0.00090	-0.00070	-0.00050	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00030	0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00020	-0.00040	-0.00040

Difference between max and min readings, in:

0° = 0.00120 90° = 0.00050

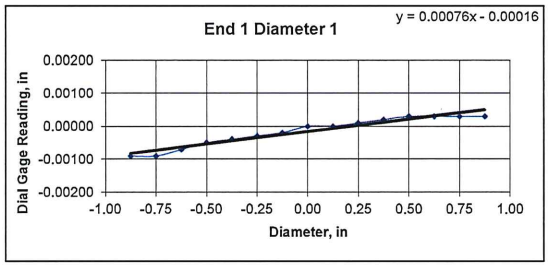
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00080	-0.00070	-0.00050	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00030	0.00040	0.00040	0.00040	0.00040
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050

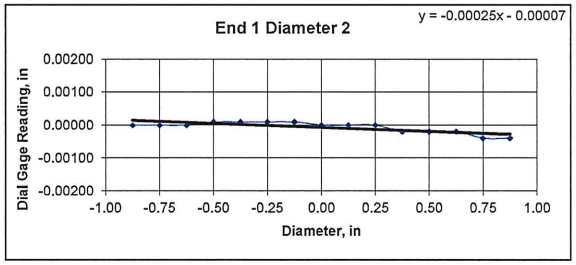
Difference between max and min readings, in:

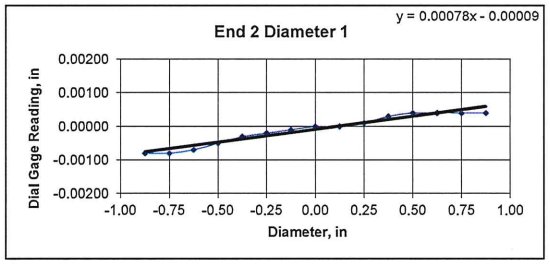
0° = 0.0012 90° = 0.0005

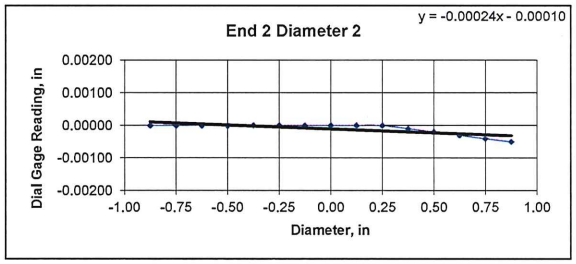
Maximum difference must be < 0.0020 in. Difference = ± 0.00060

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00076
Angle of Best Fit Line:	0.04354

End 2:

Slope of Best Fit Line	0.00078
Angle of Best Fit Line:	0.04469

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	-0.00025
Angle of Best Fit Line:	-0.01432

End 2:

Slope of Best Fit Line	-0.00024
Angle of Best Fit Line:	-0.01375

Maximum Angular Difference: 0.00057

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	0.00120	1.985	0.00060	0.035	YES
Diameter 2, in (rotated 90°)	0.00050	1.985	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00120	1.985	0.00060	0.035	YES
Diameter 2, in (rotated 90°)	0.00050	1.985	0.00025	0.014	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. Route 1 Viaduct Bridge Replacement
Project Location:	Bath, ME
GTX #:	300397
Test Date:	4/9/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-BROV-102
Sample ID:	R1
Depth, ft:	13.98-14.35

After cutting and grinding

After break

GeoTesting EXPRESS

WARRANTY and LIABILITY

GeoTesting Express (GTX) warrants that all tests it performs are run in general accordance with the specified test procedures and accepted industry practice. GTX will correct or repeat any test that does not comply with this warranty. GTX has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

GTX may report engineering parameters that require us to interpret the test data. Such parameters are determined using accepted engineering procedures. However, GTX does not warrant that these parameters accurately reflect the true engineering properties of the *in situ* material. Responsibility for interpretation and use of the test data and these parameters for engineering and/or construction purposes rests solely with the user and not with GTX or any of its employees.

GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any lost benefits or other consequential damages resulting from the use of these test results, even if GTX has been advised of the possibility of such damages. GTX will not be responsible for any liability of the Purchaser to any third party.

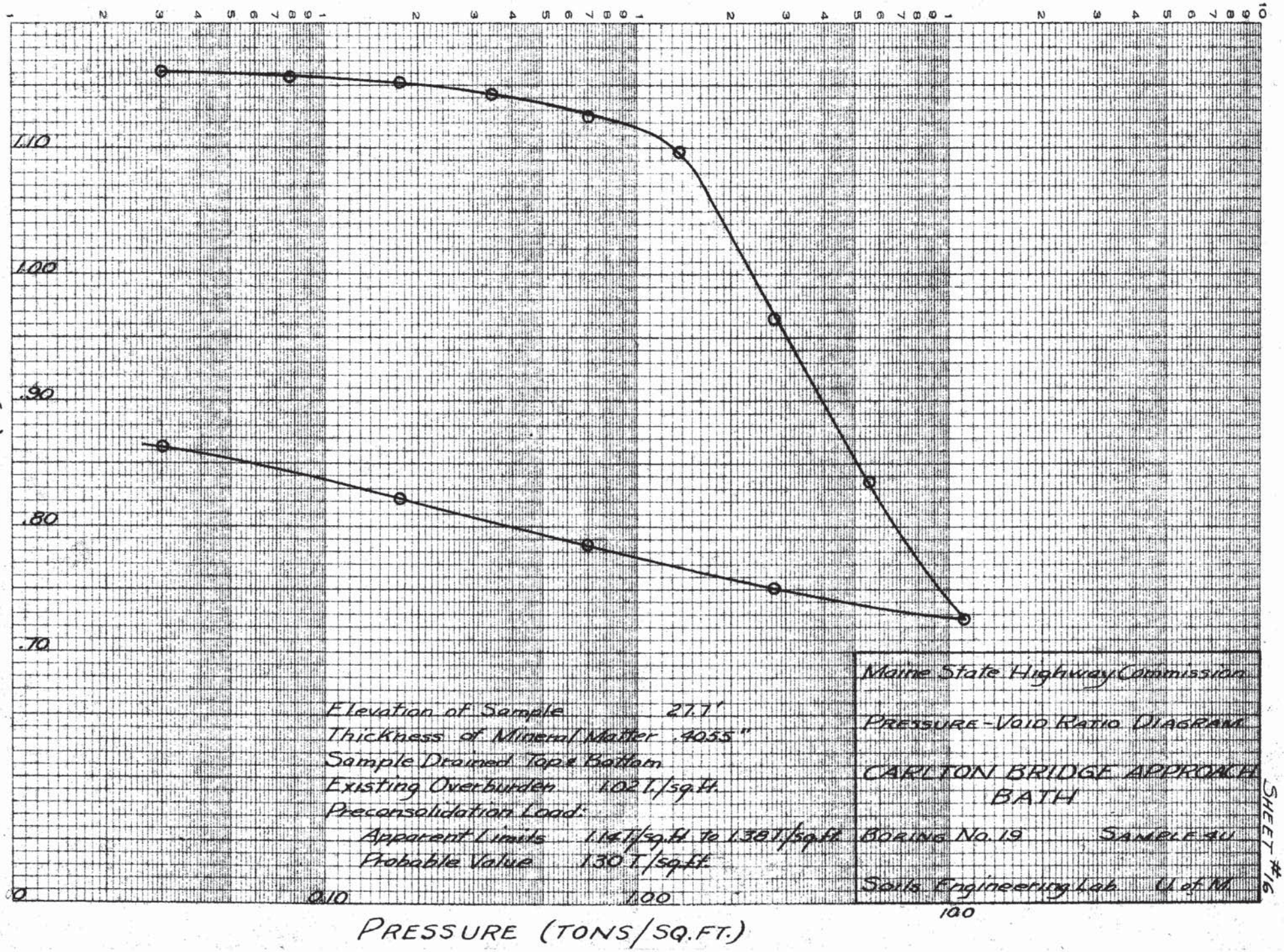
Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
C_c	coefficient of curvature, $(D_{30})^2 / (D_{10} \times D_{60})$
C_u	coefficient of uniformity, D_{60}/D_{10}
C_c	compression index for one dimensional consolidation
C_α	coefficient of secondary compression
c_v	coefficient of consolidation
c	cohesion intercept for total stresses
c'	cohesion intercept for effective stresses
D	diameter of specimen
D_{10}	diameter at which 10% of soil is finer
D_{15}	diameter at which 15% of soil is finer
D_{30}	diameter at which 30% of soil is finer
D_{50}	diameter at which 50% of soil is finer
D_{60}	diameter at which 60% of soil is finer
D_{85}	diameter at which 85% of soil is finer
d_{50}	displacement for 50% consolidation
d_{90}	displacement for 90% consolidation
d_{100}	displacement for 100% consolidation
E	Young's modulus
e	void ratio
e_c	void ratio after consolidation
e_o	initial void ratio
G	shear modulus
G_s	specific gravity of soil particles
H	height of specimen
PI	plasticity index
i	gradient
K_o	lateral stress ratio for one dimensional strain
k	permeability
LI	Liquidity Index
m_v	coefficient of volume change
n	porosity
PI	plasticity index
P_c	preconsolidation pressure
p	$(\sigma_1 + \sigma_3) / 2$, $(\sigma_v + \sigma_h) / 2$
p'	$(\sigma'_1 + \sigma'_3) / 2$, $(\sigma'_v + \sigma'_h) / 2$
p'_c	p' at consolidation
Q	quantity of flow
q	$(\sigma_1 - \sigma_3) / 2$
q_f	q at failure
q_o, q_i	initial q
q_c	q at consolidation
S	degree of saturation
SL	shrinkage limit
s_u	undrained shear strength
T	time factor for consolidation
T	temperature
t	time
U, UC	unconfined compression test
UU, Q	unconsolidated undrained triaxial test
u_a	pore gas pressure
u_e	excess pore water pressure
u, u_w	pore water pressure
V	total volume
V_g	volume of gas
V_s	volume of solids
V_v	volume of voids
V_w	volume of water
V_o	initial volume
v	velocity
W	total weight
W_s	weight of solids
W_w	weight of water
w	water content
w_c	water content at consolidation
w_f	final water content
w_l	liquid limit
w_n	natural water content
w_p	plastic limit
w_s	shrinkage limit
w_o, w_i	initial water content
α	slope of q_f versus p_f
α'	slope of q_f versus p_f'
γ_t	total unit weight
γ_d	dry unit weight
γ_s	unit weight of solids
γ_w	unit weight of water
ε	strain
ε_{vol}	volume strain
ε_h, ε_v	horizontal strain, vertical strain
μ	Poisson's ratio, also viscosity
σ	normal stress
σ'	effective normal stress
σ_c, σ'_c	consolidation stress in isotropic stress system
σ_h, σ'_h	horizontal normal stress
σ_v, σ'_v	vertical normal stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ	shear stress
φ	friction angle based on total stresses
φ'	friction angle based on effective stresses
φ'_r	residual friction angle
φ_{ult}	φ for ultimate strength

Historic Consolidation Test Results

NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

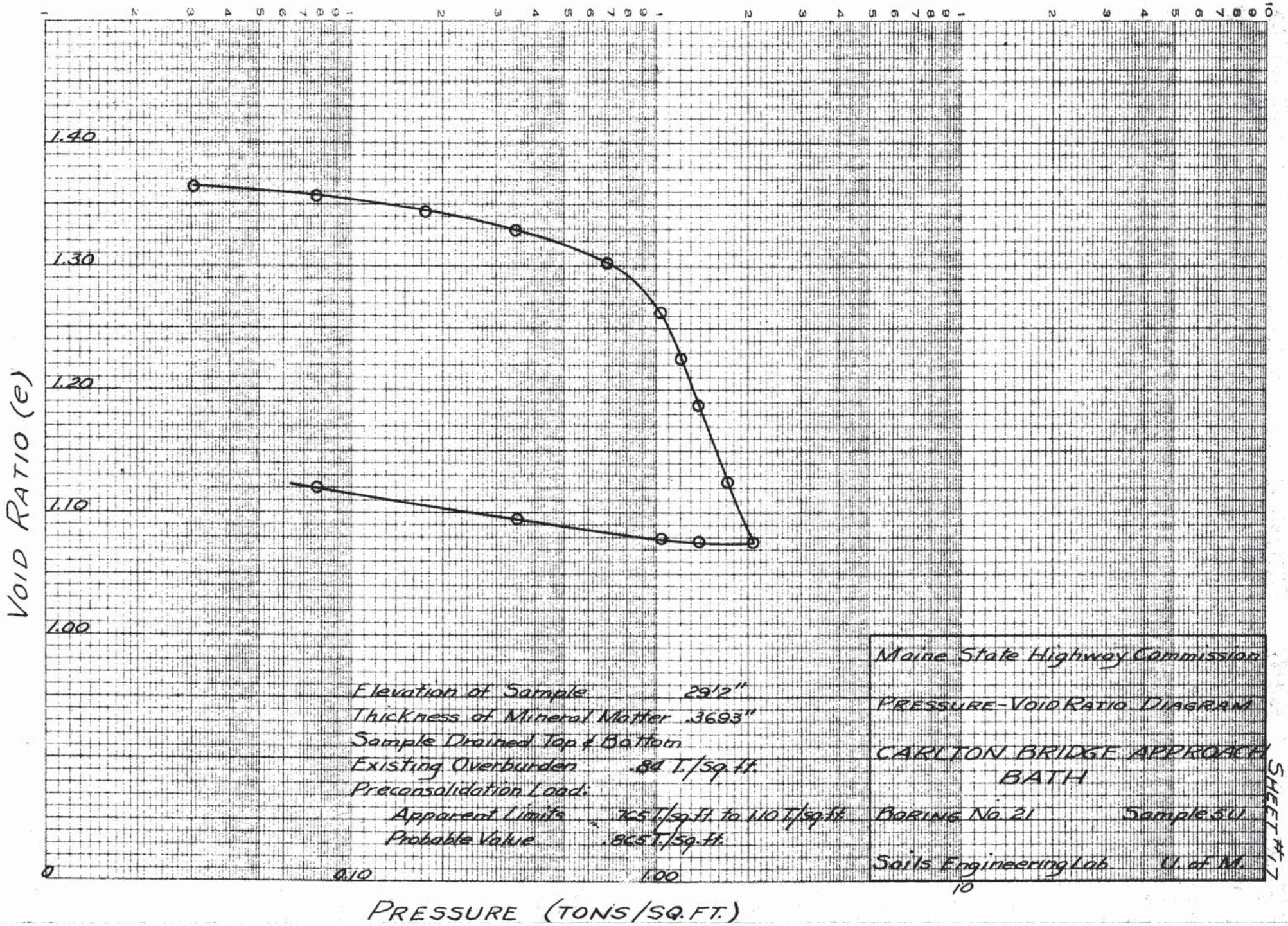
EUGENE DIETZGEN CO.
MADE IN U. S. A.



SHEET #16

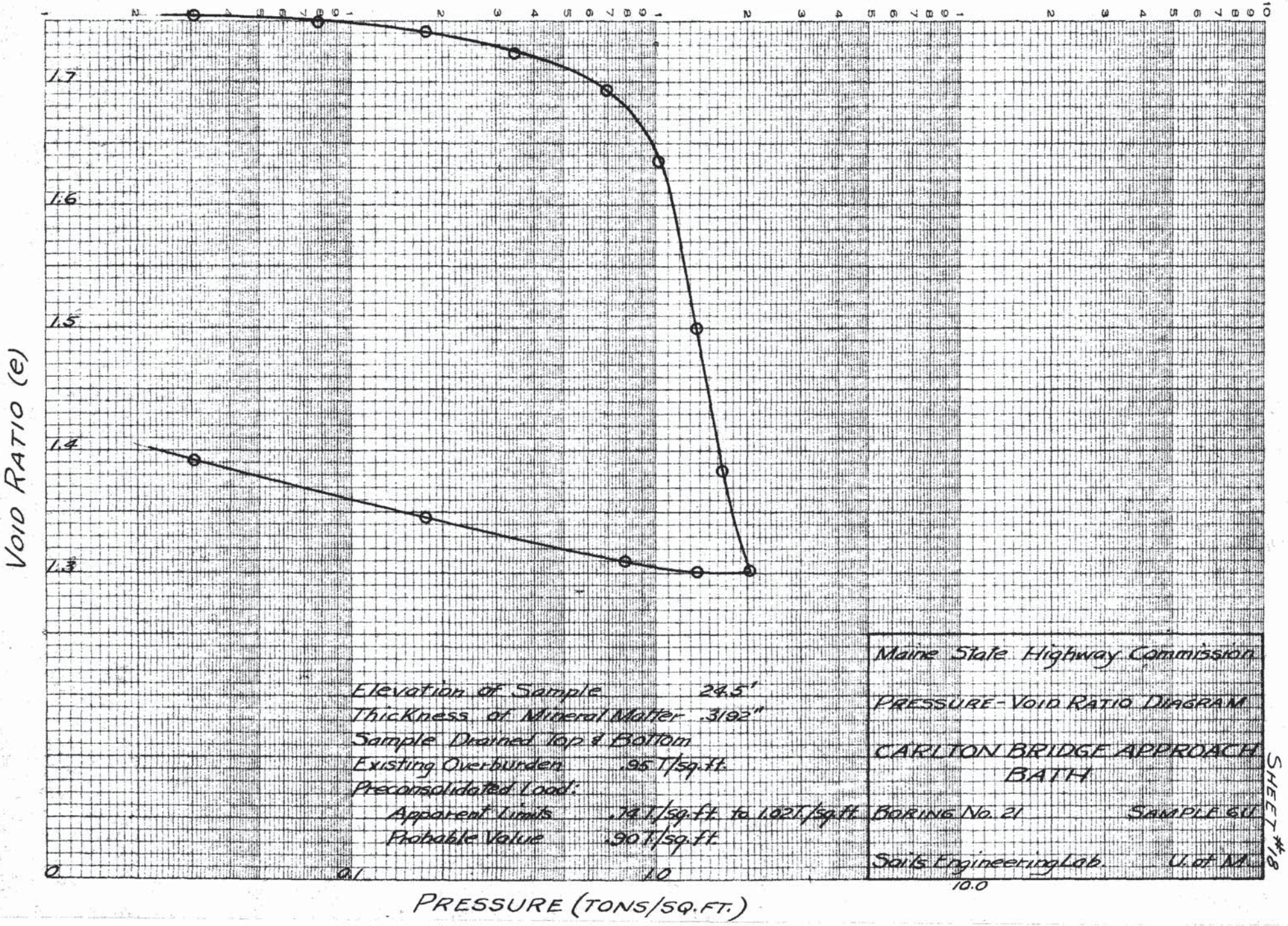
NO. 340-L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 70 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



NO. 340 L410 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
4 CYCLES X 10 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



APPENDIX D

Historic Bridge Drawings

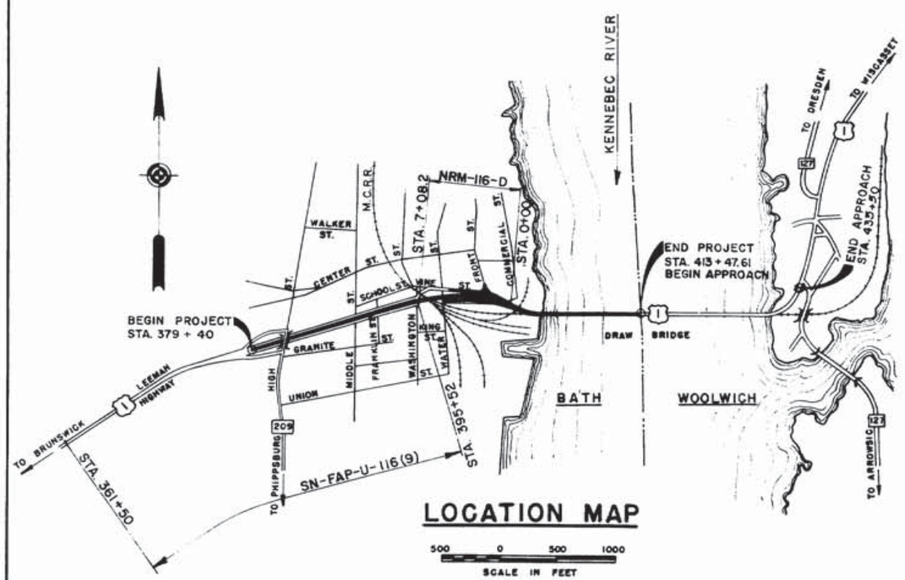
B. P. R. DIV. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	1	148

STATE OF MAINE
STATE HIGHWAY COMMISSION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH AND WOOLWICH
SAGADAHOC COUNTY, MAINE

FEDERAL AID PROJECT NO. U-026-1(8)

LENGTH OF PROJECT 0.645 MILE



LOCATION MAP

A.D.T. 1959 = 2470

A.D.T. 1979 = 3285

D.H.V. = 330

D = 65 %

T = 6 %

V = 40 MPH

CONCRETE CLASSIFICATION

CONCRETE FILL ..... CLASS "B"
ALL OTHER CONCRETE ..... CLASS "A"

LOADING H20-S16-44

fs = 20000 p.s.i.
fc = 1200 p.s.i.
n = 10

SPECIFICATIONS

MAINE STATE HIGHWAY COMMISSION STANDARD SPECIFICATIONS, REVISIONS OF JAN. 1956, EXCEPT AS MODIFIED ON THESE PLANS AND IN THE SUPPLEMENTAL SPECIFICATIONS.

WEST APPROACH
INDEX OF SHEETS

1 ..... TITLE SHEET
2 ..... STANDARD DETAILS - CATCH BASINS, MANHOLES, ETC.
3 ..... STANDARD DETAILS - GUARD RAIL & FITTINGS, ETC.
4 ..... STANDARD DETAILS - MISCELLANEOUS ITEMS
5 ..... GENERAL PLAN - WEST APPROACH
6-7-8-9 ..... SURVEY
10 ..... BORINGS
11 ..... STRATIGRAPHICAL PROFILE
12 ..... ALIGNMENT AND PROFILE STA. 379 TO 387
13 ..... ALIGNMENT AND PROFILE STA. 387 TO 396
14 ..... ALIGNMENT AND PROFILE STA. 396 TO CARLTON BRIDGE
15 ..... TRESTLE PROFILE STA. 379 TO 388
16 ..... TRESTLE PROFILE STA. 388 TO 397
17 ..... TRESTLE PROFILE STA. 397 TO 403 + 50
18 ..... PROFILE - FRONT ST. TO CARLTON BRIDGE
19 ..... PROFILE - VINE STREET
20 ..... PROFILE - RAMP
21 ..... TYPICAL CROSS SECTIONS
22-23-24-25 CROSS SECTIONS
26 ..... TRAFFIC ISLAND STA. 379 + 50 TO 381 + 50
27 ..... PROW A, AND TRAFFIC ISLANDS STA. 382 + 50 TO HIGH ST.
28 ..... PLAN OF MEDIANS AT PIERS 6-7-9-10-14
29 ..... LAYOUT - VINE AND WATER STREETS
30 ..... LAYOUT - VINE AND FRONT STREETS
31 ..... ROADWAY DRAINAGE
32 ..... RELOCATION OF SIDEWALK AT WASHINGTON STREET
33 ..... ABUTMENT NO. 1
34-35 ..... ABUTMENT NO. 1 - RETAINING WALLS
36 ..... PIERS NO. 1 THRU 11
37 ..... PIERS NO. 12 THRU 19
38 ..... ABUTMENT NO. 2
39-40 ..... NORTH AND SOUTH RETAINING WALLS - ABUTMENT NO. 2
41-42 ..... RAMP RETAINING WALL
43 ..... VINE STREET RETAINING WALL
44 ..... VINE STREET SIDEWALK STA. 399 + 25 TO 399 + 94*
45 ..... PROW B
46-47 ..... DETAILS - T-BEAM SPANS 1 TO 6
48-49 ..... DETAILS - T-BEAM SPANS 7 TO 12
50 ..... RAIL AND RAIL CURB LAYOUT - T-BEAM SPANS 1 TO 12
51 ..... REINFORCING STEEL LAYOUT - T-BEAM SPANS 1 TO 6
52 ..... REINFORCING STEEL LAYOUT - T-BEAM SPANS 7 TO 12
53 ..... EXPANSION DAMS AND BEARINGS - T-BEAM SPANS 1 TO 12
54 ..... DRAINAGE DETAILS - T-BEAM SPANS 1 TO 12
55 ..... DRAINAGE DETAILS - STEEL SPANS 13 TO 20
56 ..... STRUCTURAL STEEL - SPANS 13 TO 20
57 ..... EXPANSION DAMS AND BEARINGS - SPANS 13 TO 20
58 ..... SHEAR CONNECTORS - SPANS 13 TO 20
59 ..... SUPERSTRUCTURE SLABS - SPANS 13 THRU 18
60 ..... SUPERSTRUCTURE SLABS - SPANS 19 & 20
61 ..... BLOCKING FOR SPANS 13 THRU 20, OFFSETS FOR SLAB SPANS 19 & 20
62 ..... RAIL LAYOUT - SPANS 13 THRU 20
63 ..... ALUMINUM RAIL - TYPICAL DETAILS, PYLON CAP DETAIL
64 ..... REINFORCING STEEL - ABUTMENT NO. 1 & RETAINING WALLS, VINE STREET RETAINING WALL & PROWS
65 ..... REINFORCING STEEL - ABUTMENT NO. 2 & RETAINING WALLS
66 ..... REINFORCING STEEL - RAMP RETAINING WALL, LIGHT POST BASES
67 ..... REINFORCING STEEL - PIERS 1 THRU 11
68 ..... REINFORCING STEEL - PIERS 12 THRU 19
69 ..... REINFORCING STEEL - SUPERSTRUCTURE, TRESTLE SPANS 1 THRU 20
70 ..... CONDUIT AND LIGHTING LAYOUT
71 ..... CONDUIT DETAIL AT PIER 6, AND SPAN 18
72 ..... CONDUIT DETAIL AND CONCRETE LIGHT BASES
73 ..... CONDUIT DETAIL AND HANGERS
74 ..... LIGHT STANDARDS AND ANCHOR BOLT DETAIL
75 ..... CONDUIT DETAIL - BATH AND WOOLWICH ABUTMENTS

USE SHEET NO. 70 FOR BOTH WEST APPROACH & CARLTON BRIDGE WIDENING
USE SHEETS NO. 73, 74, & 75 FOR CARLTON BRIDGE WIDENING

NOTE – FOR GENERAL PLAN - CARLTON BRIDGE WIDENING & INDEX OF SHEETS SEE SHEET NO. 76

APPROVED:
MAINE CENTRAL RAILROAD
J. W. Higgins
CHIEF ENGINEER

APPROVED:
MAINE STATE HIGHWAY COMMISSION
David H. Stevens
CHAIRMAN
Harold B. Emery
Carl Lawrence Gray
CHIEF ENGINEER

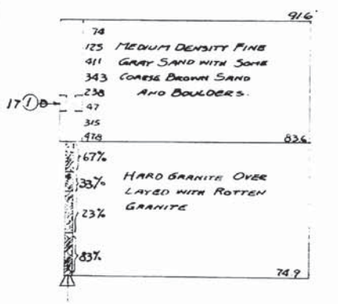
DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
REGION 1

APPROVED:

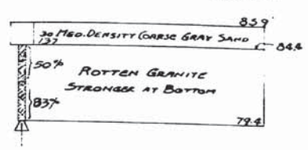
DIVISION ENGINEER ..... DATE

F. A. & B. Sta. No.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
-	MAINE	U-026-1(8)	10	9/48

BORING NO. 1
STA 385+95 CENTER ABUT. #1

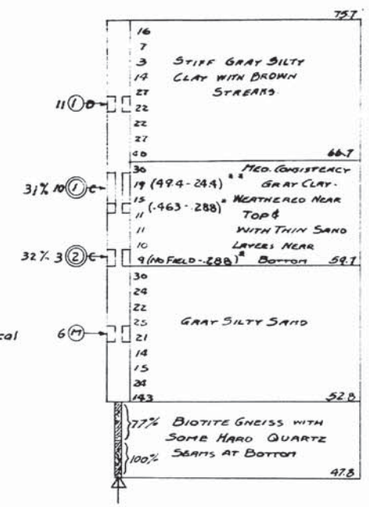


BORING NO. 2
STA. 387+15 CENTER PIER #2

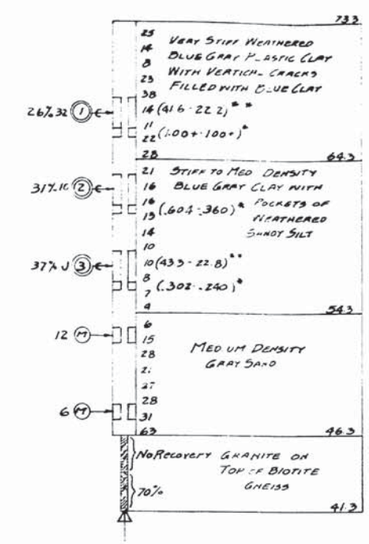


BORING NO. 3
STA. 388+37 CENTER PIER #4

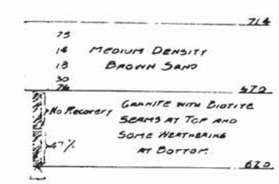
BORING NO. 4
STA 389+51 CENTER PIER #6



BORING NO. 5
STA 391+22 CENTER PIER #7

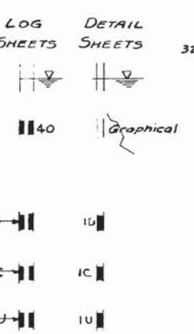


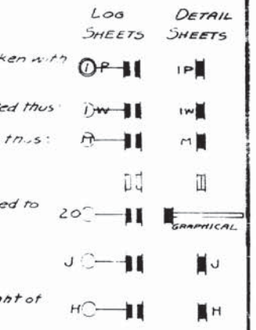
BORING NO. 6
STA 392+65 CENTER PIER #8



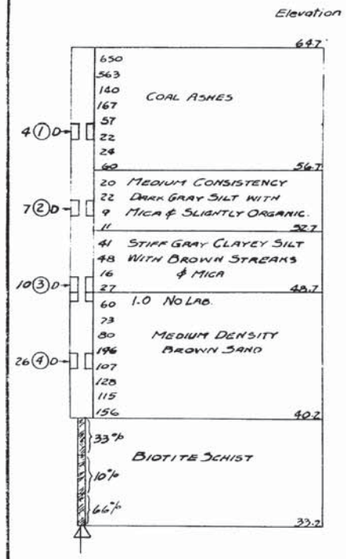
BORING NOTES

1. Scales as Noted on Drawings
2. Casing Size as Noted on Drawings.
3. Ground Water Table indicated thus:
4. Number of Blows of 275# Hammer falling 18 inches required to drive Extra Heavy Casing One Foot thus:
5. W, S, or V indicate respectively that the Hole was Washed, Sampled or Vaned ahead of Casing.
6. Location and designation of dry Samples taken in S.&H. sampler #1290s indicated thus:
7. Location and designation of dry samples taken in 2" O.D. 18 Ga. seamless Tubing indicated thus:
8. Location and designation of dry samples taken in 3½ O.D. 16 Ga. seamless tubing indicated thus:
9. Location and designation of 3½ O.D. dry samples taken with piston sampler indicated thus:
10. Location and designation of wash samples indicated thus:
11. Unsuccessful attempts to secure dry sample indicated thus:
12. Location of field vane test indicated thus:
13. Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus:
14. Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:
15. Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:

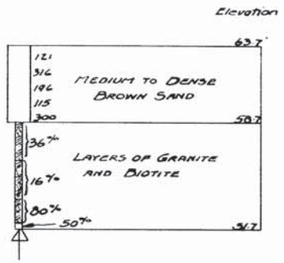




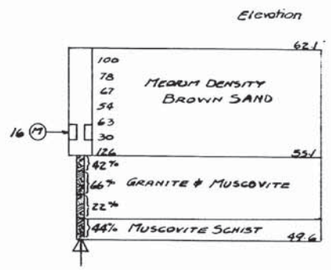
BORING NO. 7
STA 393+40 CENTER PIER #12



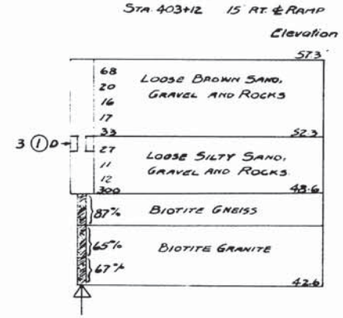
BORING NO. 8
STA 394+04 CENTER PIER #13



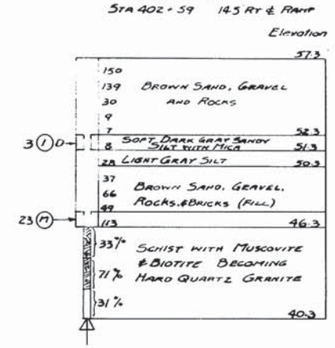
BORING NO. 9
STA 394+68 CENTER PIER #14



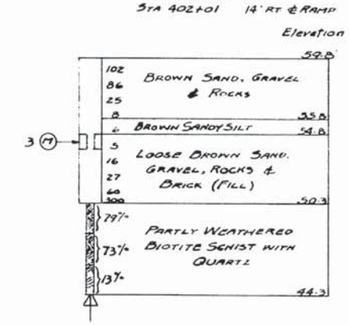
BORING NO. 10
LOCATION "O"
STA 403+12 15' RT. & RAMP



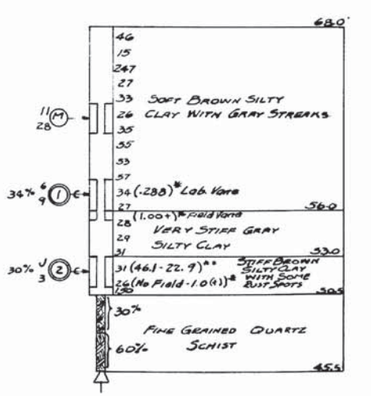
BORING NO. 11
LOCATION "M"
STA 402+59 14.5' RT. & RAMP



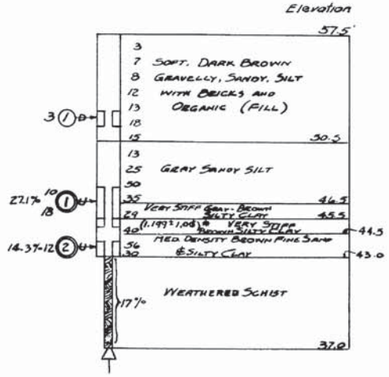
BORING NO. 12
LOCATION "K"
STA 402+01 14' RT. & RAMP



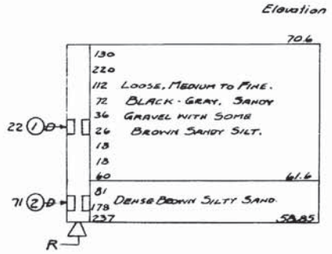
BORING NO. 14
STA 392+13 CENTER PIER #10



BORING NO. 15
LOCATION "E"
STA 400+34 47' RT. & TRESTLE

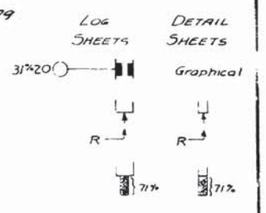


BORING NO. 23
STA. 401+00 15' RT. & TRESTLE



BORING NOTES

16. Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.
17. Natural water contents, given as percent of dry weight are indicated thus:
18. Bottom of boring indicated thus:
19. Refusal of drill rods or casing indicated thus:
20. % recovery of rock core by size E diamond bit thus:



NOTES:

* Field Vane Shear Strength Followed by Laboratory Vane Shear Strength in Tons/sq. ft.

** Liquid Limit Followed By Plastic Limit (Per Cent of Water Content)

SCALE 1" = 5 FEET.

DESIGN - Garrill
TRACE - JONES
CHECK - Langford

WEST APPROACH

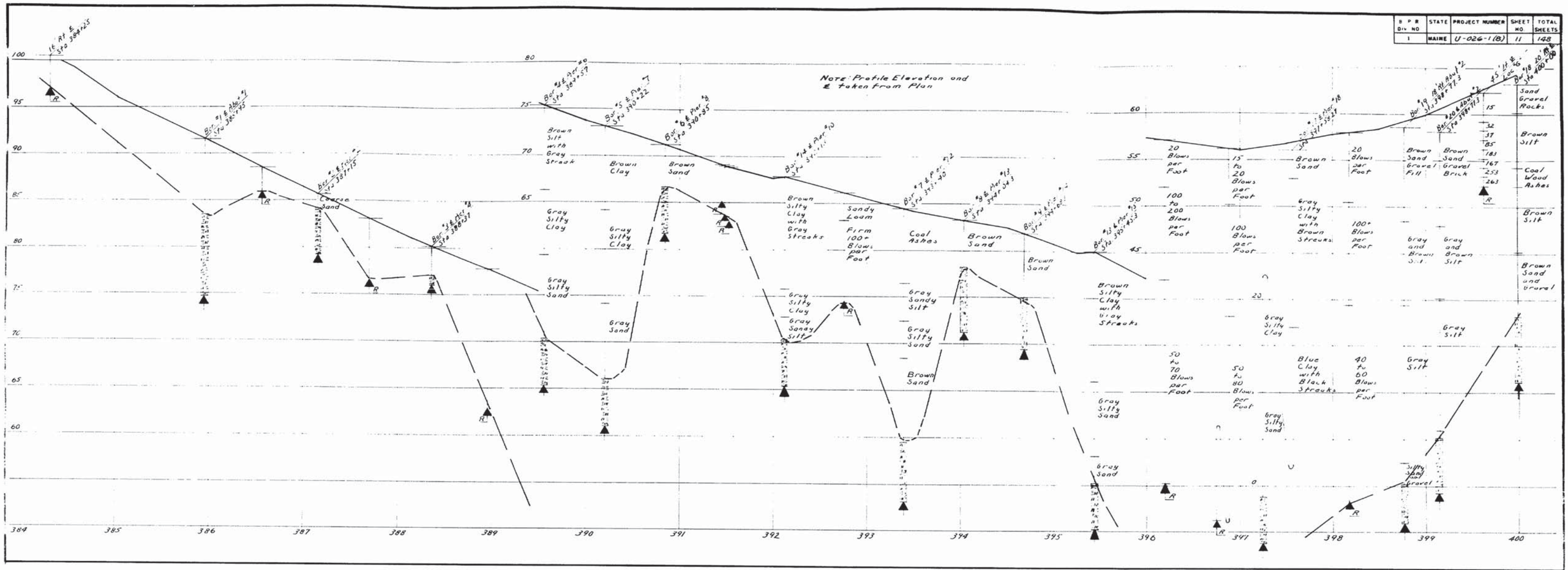
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

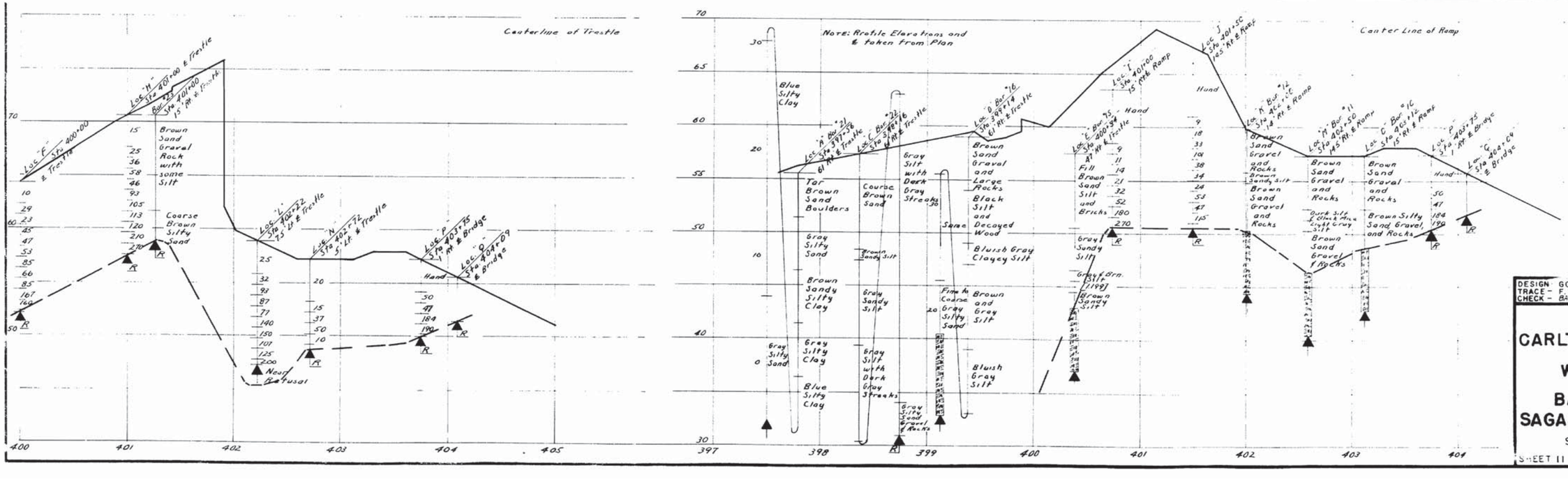
CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

BORINGS

SHEET 10 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	11	148





DESIGN - GORRILL
TRACE - F. SMITH
CHECK - BARNES

WEST APPROACH

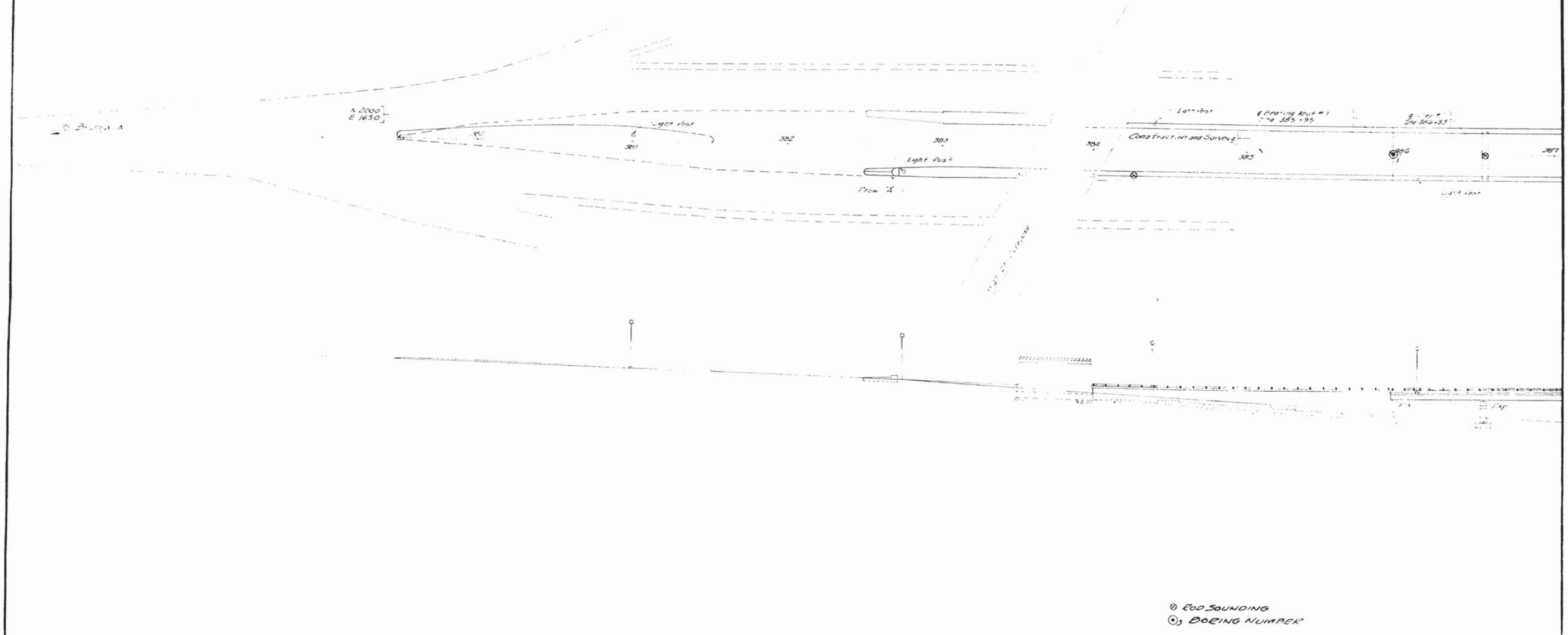
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

STRATIGRAPHICAL PROFILE

SHEET 11 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV. NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	12	148



DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

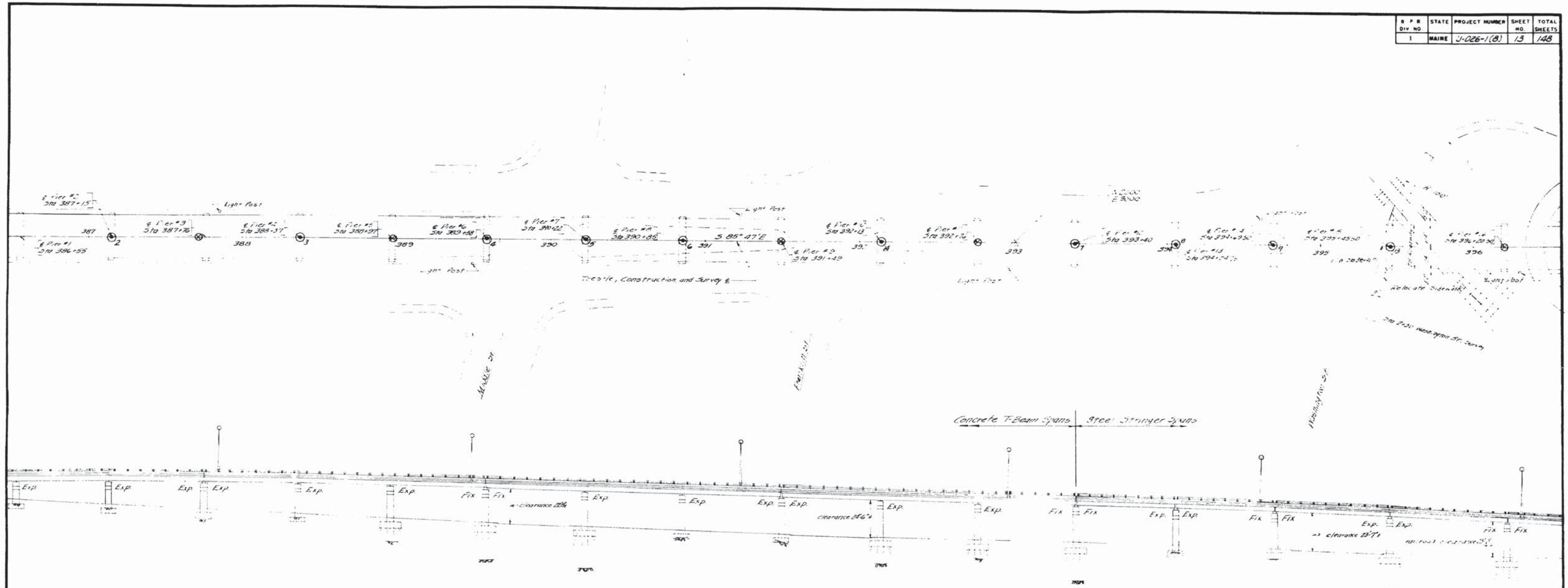
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

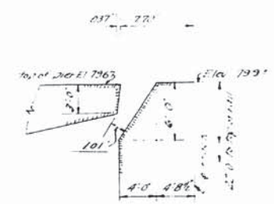
CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ALIGNMENT AND PROFILE
STA. 379 TO STA. 387

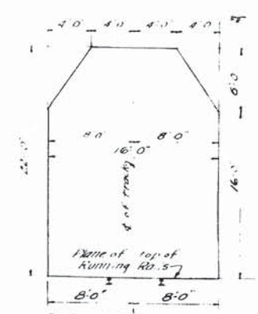
SHEET 12 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	J-026-1(8)	13	148

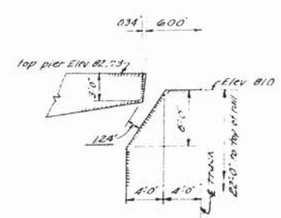




Railroad Clearance at Pier #16



Railroad Clearance Diagram



Railroad Clearance at Pier #15

DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

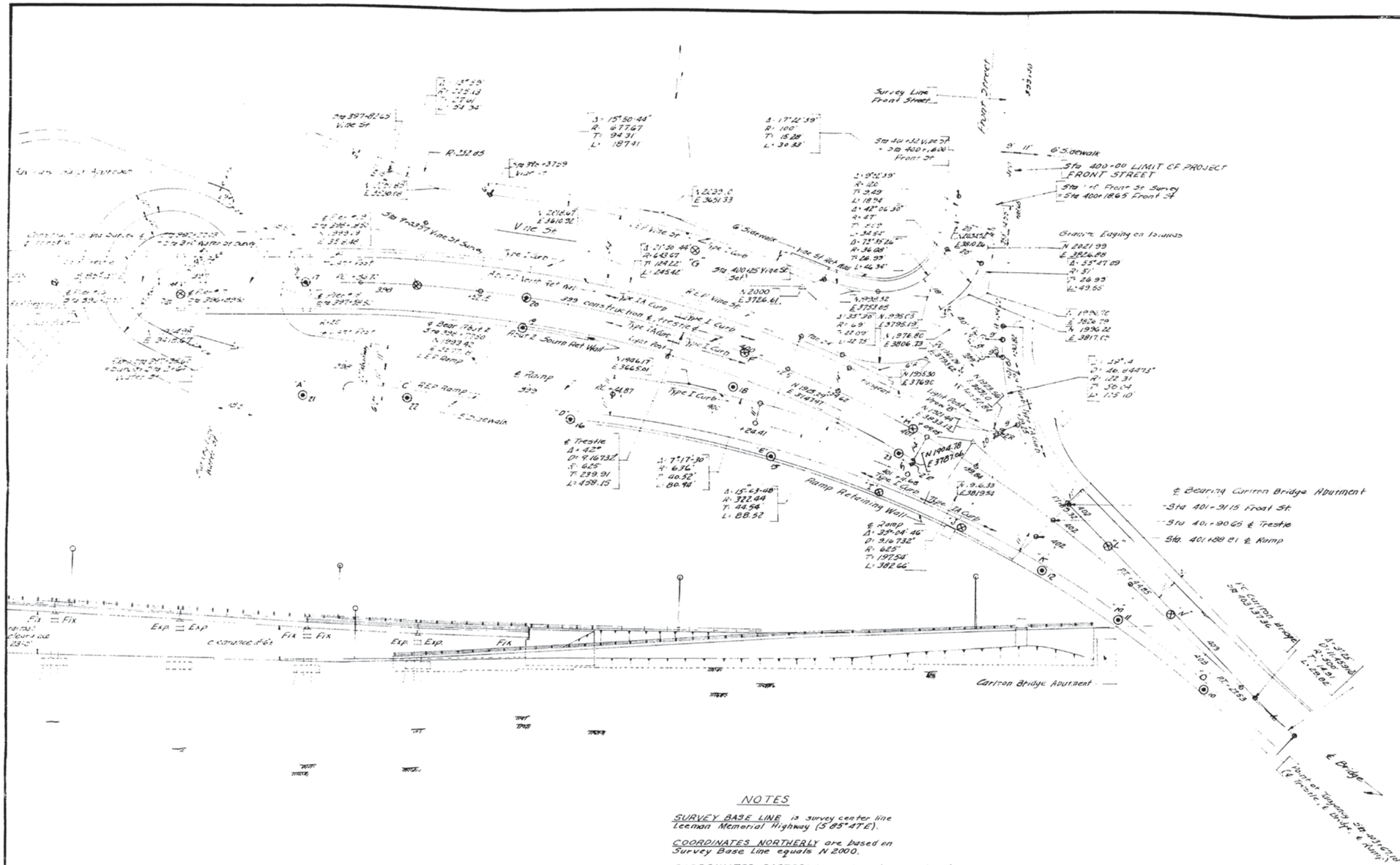
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ALIGNMENT AND PROFILE
STA. 387 TO STA. 396

SHEET 13 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	14	148



NOTES

SURVEY BASE LINE is survey center line Leeman Memorial Highway (S 85°47' E).

COORDINATES NORTHERLY are based on Survey Base Line equals N 2000.

COORDINATES EASTERLY are based on line at right angles to Survey Base Line at Sta. 393+00 equals E 3000.

All curves are arc length.
Scale: 1 inch = 30 feet

Construction center line and center line of trestle are the same.

Construction center line and survey center line are the same to Sta. 397+06.70, P.C. of curve.

DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

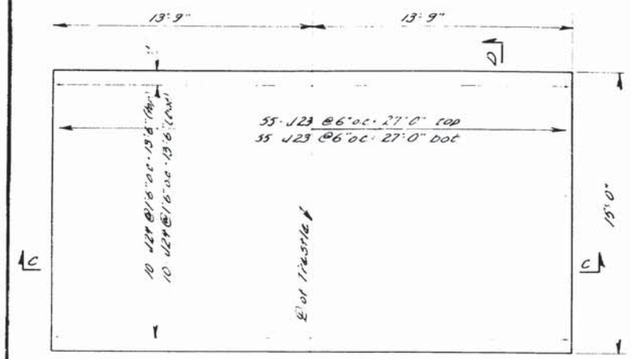
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

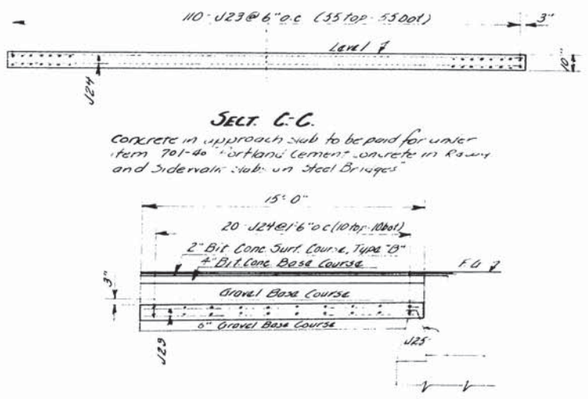
ALIGNMENT AND PROFILE
STA. 396 TO CARLTON BRIDGE

SHEET 14 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (A)	33	148



PLAN - APPROACH SLAB



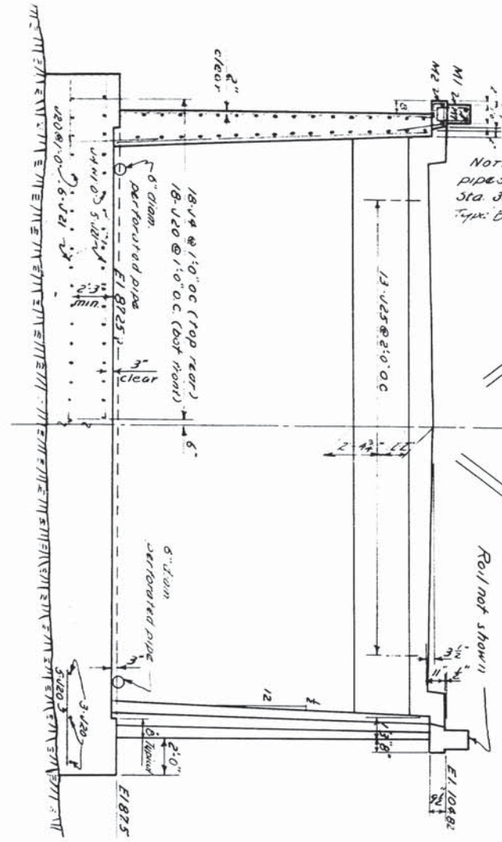
SECT. C-C.

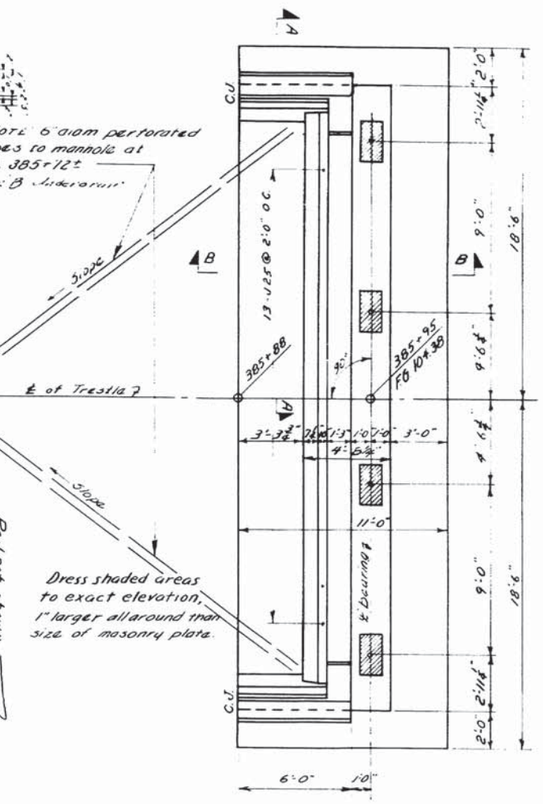
Concrete in approach slab to be paid for under item 701-40 "Portland Cement Concrete in Rdwy and Sidewalk Slab on Steel Bridges"

SECT. D-D.

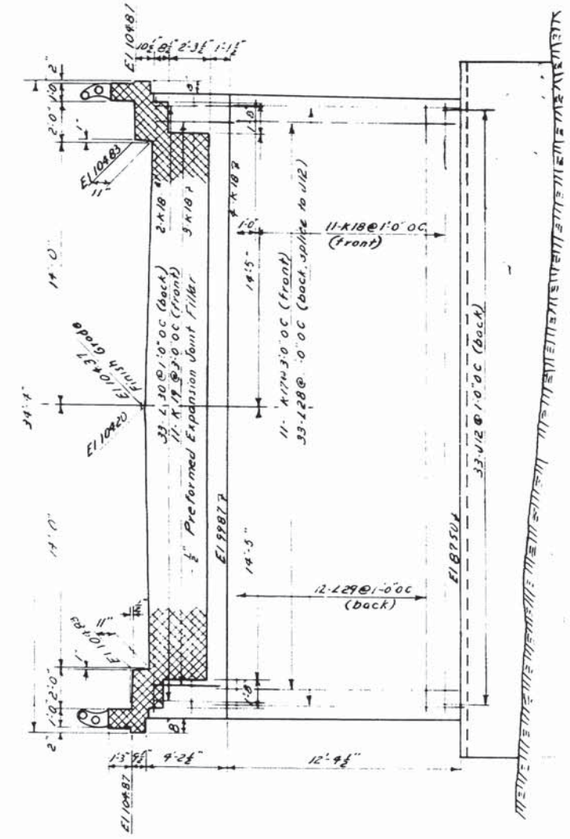
SECTION A-A

HALF REAR ELEVATION

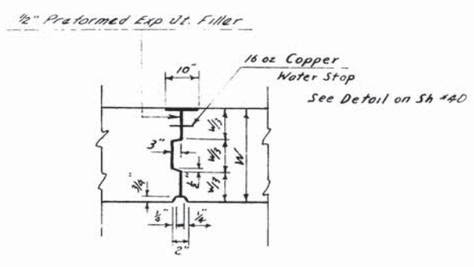




PLAN

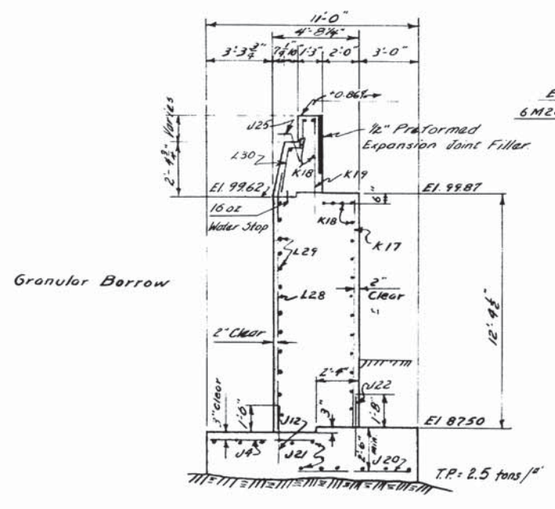


FRONT ELEVATION

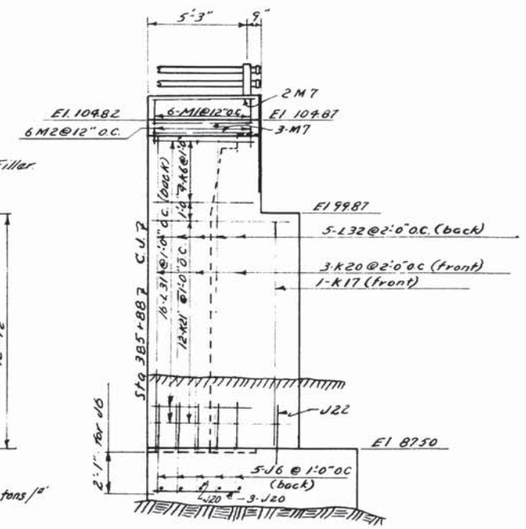


VERT GROOVE AND KEY AT CONSTRUCTION JOINTS

Cover constructruction joints on the back side with two layers of heavy roofing, 10" wide. Coat the surface of the concrete and the contact surface of roofing with a suitable grade of roofing cement as applied. The area to be covered shall be recessed 3/8". Do not construct Key in Coping and Rail Curb. Cover the horizontal construction joint on the rear of the breastwall at El. 99.62 in the same manner as described above. - Omit groove.



SECTION B-B



END ELEVATION

DESIGN - SHAILER
TRACE - E.V. SMITH
CHECK - [illegible]

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH

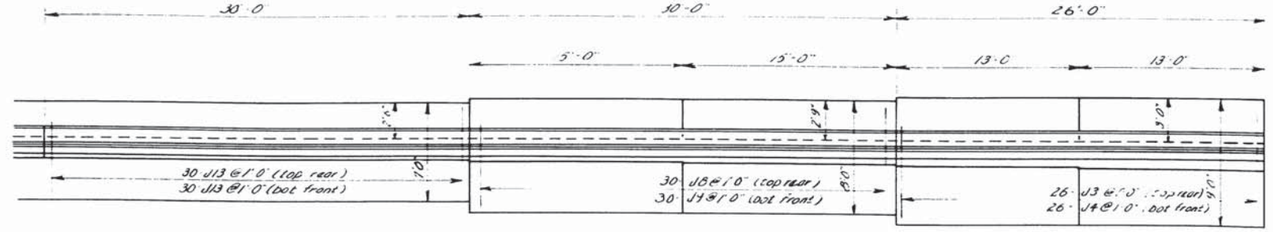
BETWEEN

BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ABUT. NO. 1

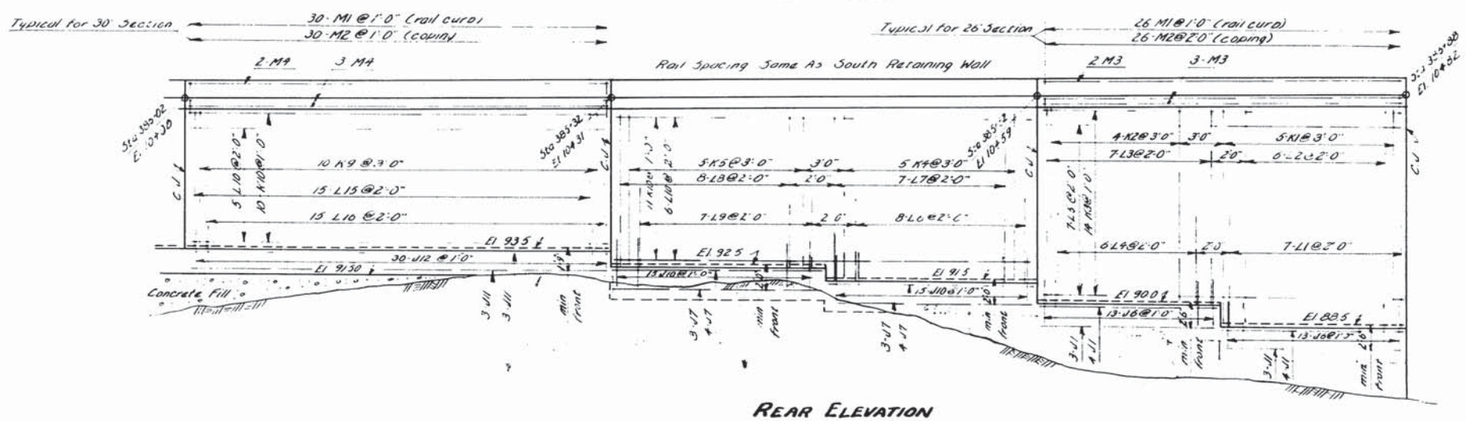
SHEET 33 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	34	148



PLAN

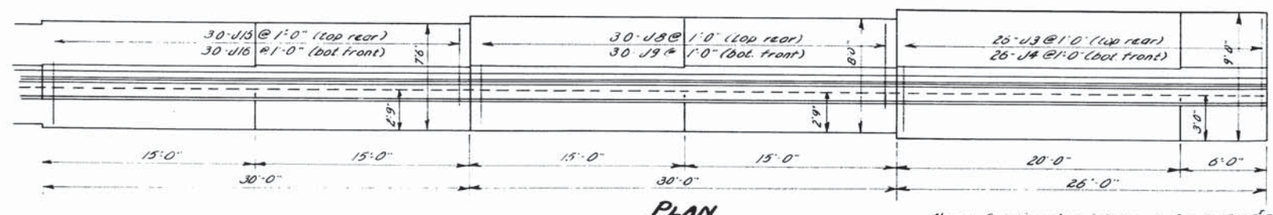
Rail not shown



REAR ELEVATION

NORTH RETAINING WALL

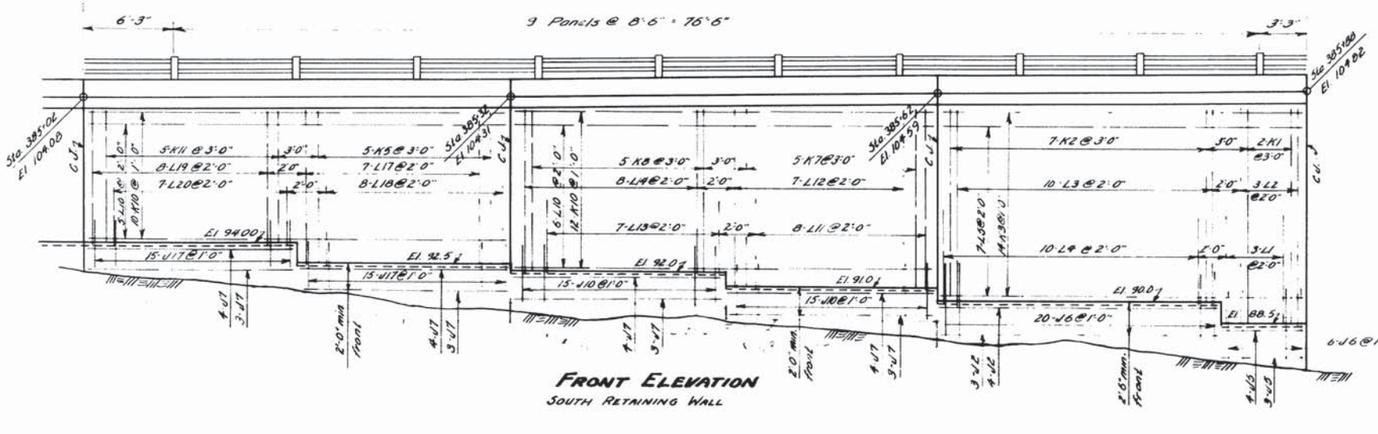
Rail not shown



PLAN

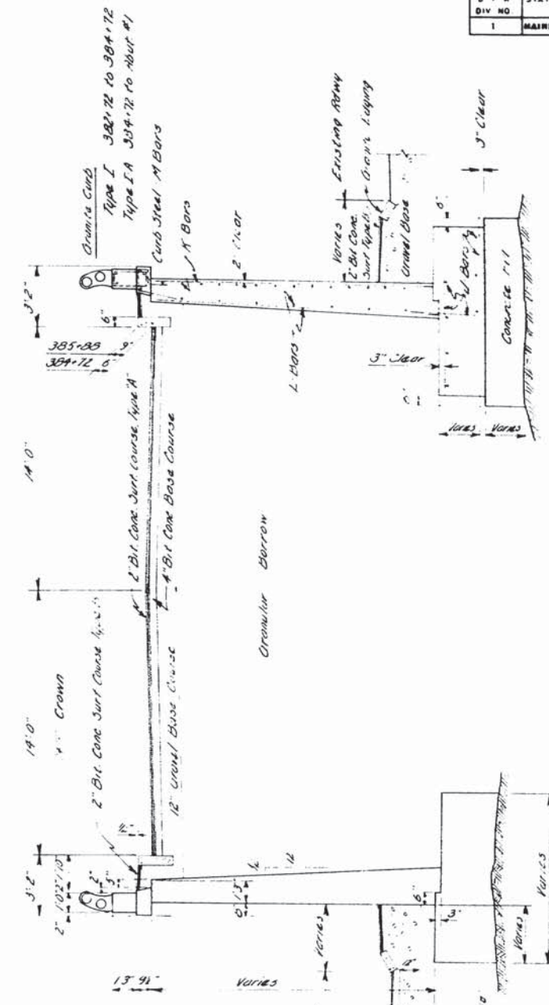
Rail not shown

Note: Existing drop inlets at Sta 385+65 Rt. and Sta. 385+89 Lt with connecting 12" R.C. Pipe to remain in place.



FRONT ELEVATION

SOUTH RETAINING WALL



TYPICAL TRANSVERSE SECTION

Note: See Sheet #40 for Construction Joint and Water Stop Details.

Key To Reinforcing Steel Location

J-Bars - Footing

L-Bars - Rear of Wall

K-Bars - Front of Wall

M-Bars - Rail Curb and Coping

DESIGN - SHAILER
TRACE - SHAILER
CHECK - Hamilton

WEST APPROACH

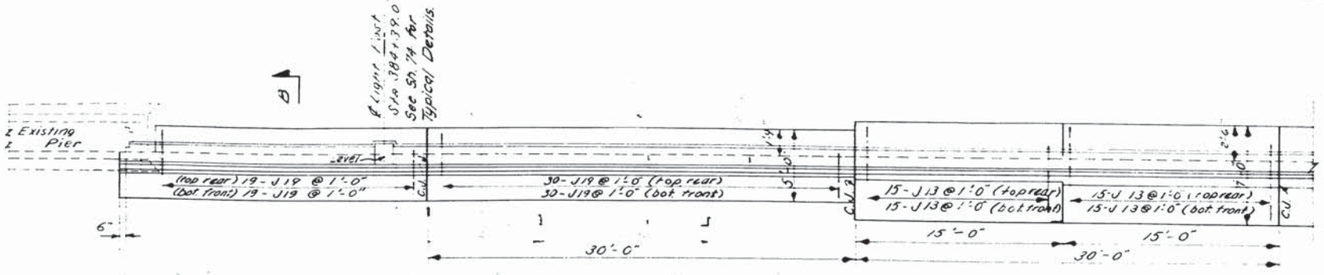
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

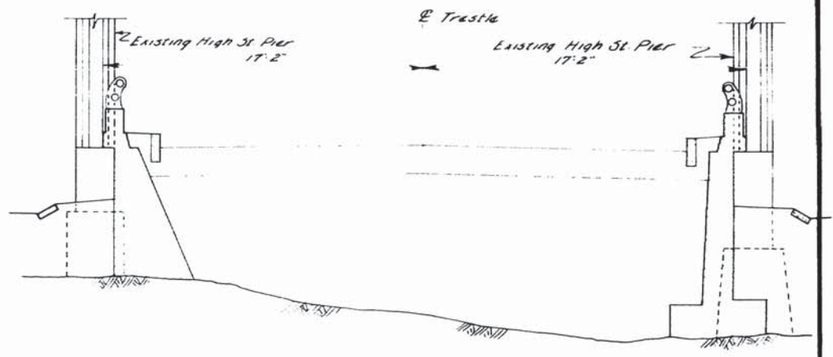
ABUT. NO. 1 RETAINING WALLS

SHEET 34 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	35	148

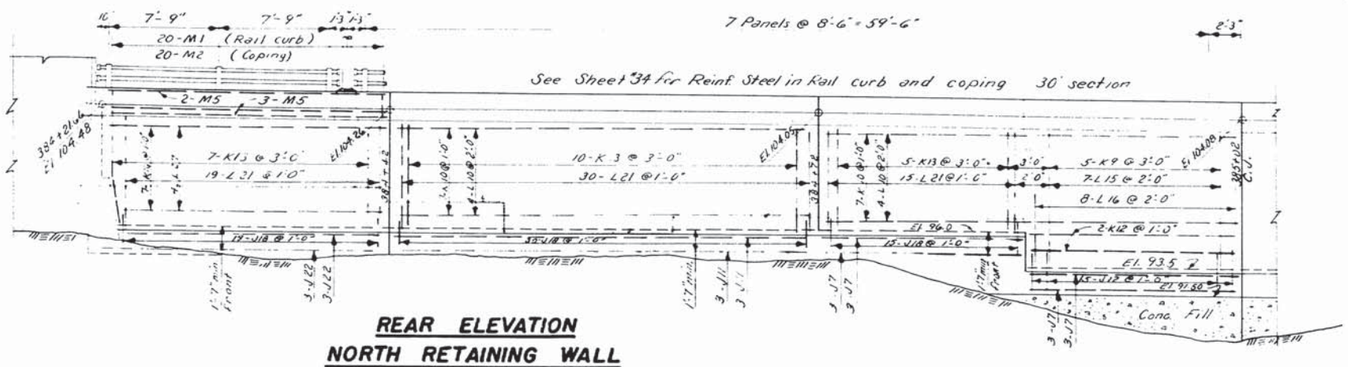


PLAN

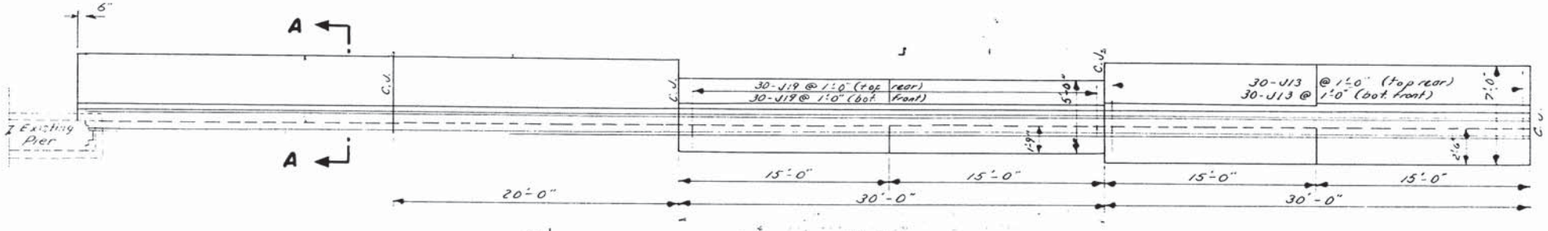


SECTION B-B

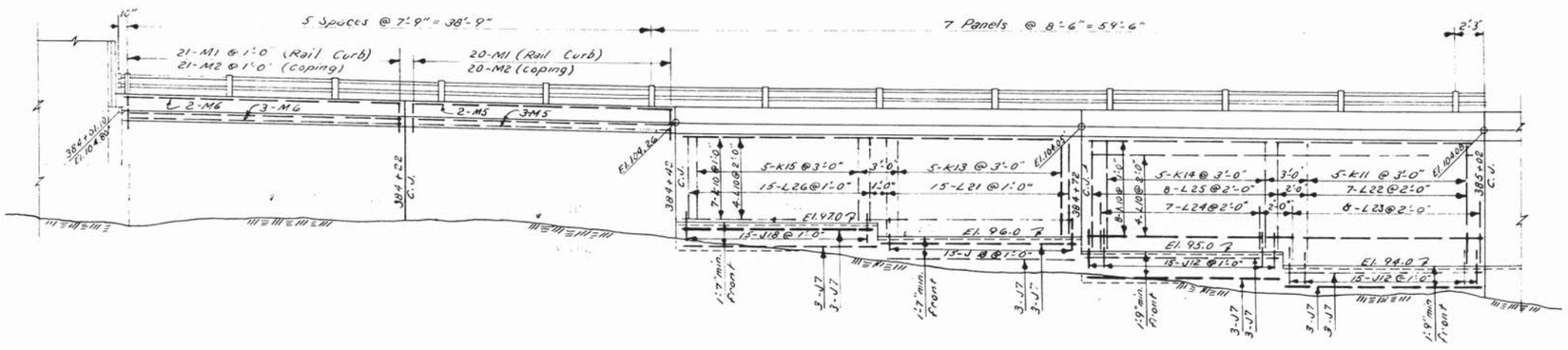
Note: Break the bond between the Existing Piers and the new concrete with a coating of roofing cement.



REAR ELEVATION
NORTH RETAINING WALL

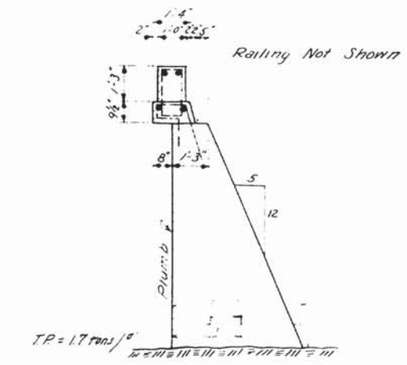


PLAN



FRONT ELEVATION
SOUTH RETAINING WALL

Note: Ledge to be excavated where necessary to obtain minimum depth footing.



SECTION "A-A"

Typical from Existing Pier to Sta. 384+42, South Retaining Wall only.

DESIGN - SHAILER
TRACE - C.M.
CHECK - Gamble

WEST APPROACH

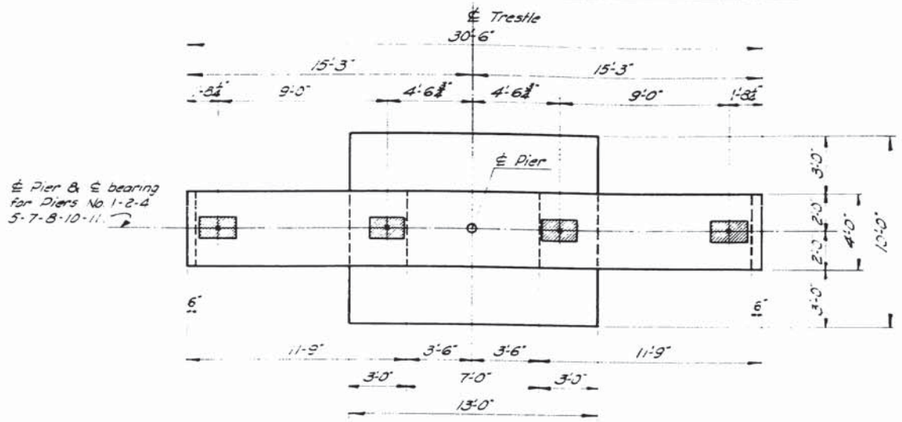
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

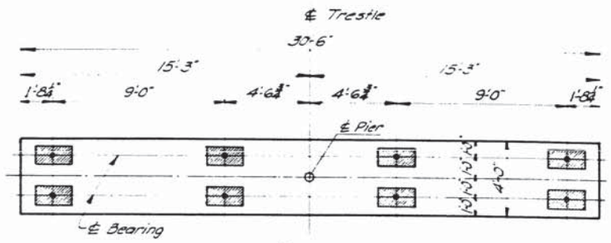
ABUTMENT NO. 1 RETAINING WALLS

SHEET 35 OF 148 AUGUSTA, MAINE JAN. 1957

S P R DIV NO	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	U-026-1 (8)	36	148



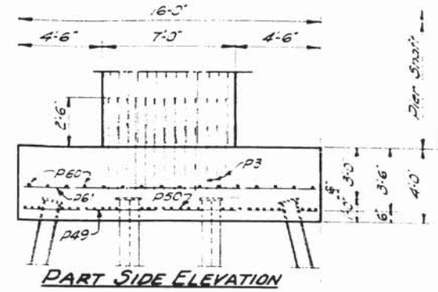
PLAN



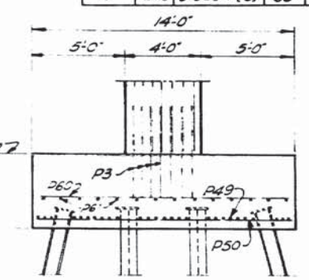
PLAN

Showing location of bearing plates for Piers No. 3-6-9.

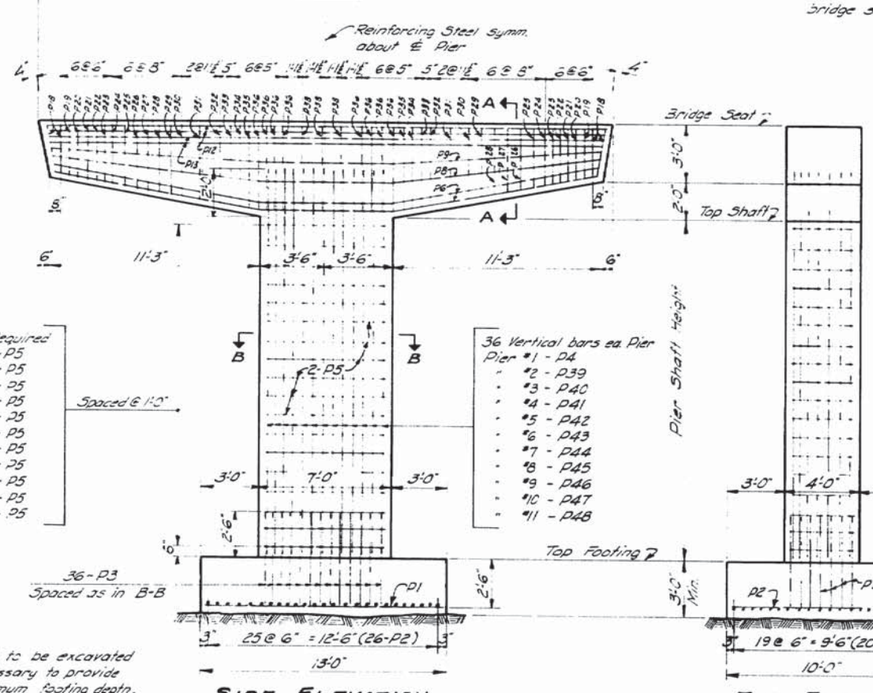
NOTE: Shaded bearing areas to be dressed 1" larger all around than masonry plates to exact bridge seat elevations.



PART SIDE ELEVATION



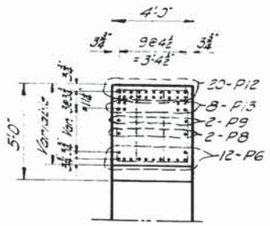
PART END ELEVATION



NOTE: Rock to be excavated if necessary to provide 3'-0" minimum footing depth.

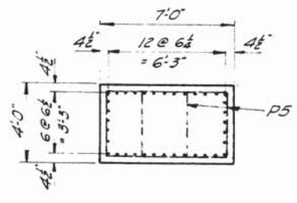
SIDE ELEVATION

END ELEVATION

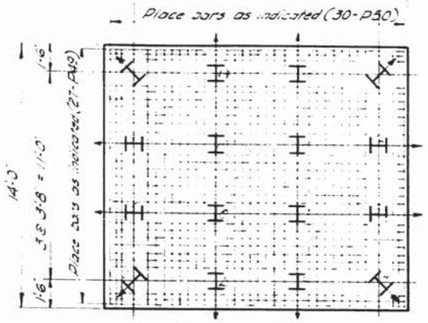


SECTION A-A

Place reinforcing steel in bridge seats to clear anchor bolts.



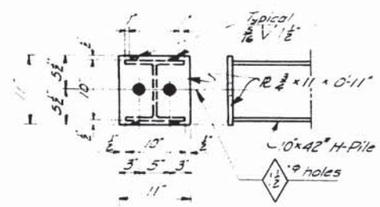
SECTION B-B



FOOTING PLAN

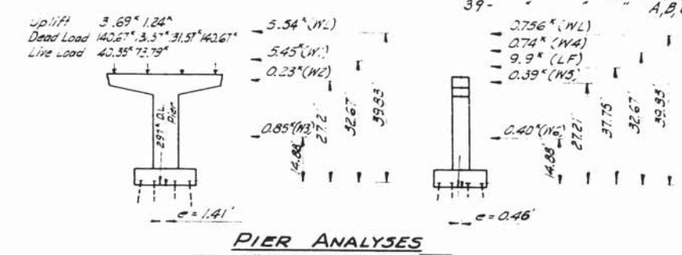
P60 & P61 @ 1'-0" ctrs 6" above top of piles

NOTE: Piles shown ⊢⊣ to be battered 3" ft in direction indicated by arrow.



PILE CAP DETAILS

16-Required for each pier
63- " " Walls O, P, Q, R, V.
39- " " " A, B, C, D, E, F.

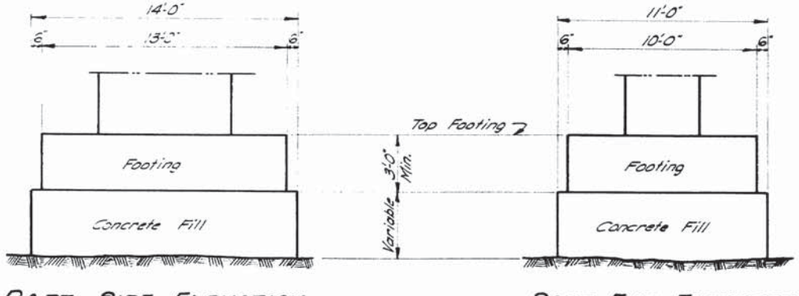


PIER ANALYSES

WL = Wind on moving Live Load
W1, W4 = Wind on superstructure
W2, W5 = Wind on pier cap
W3, W6 = Wind on pier shaft
L.F. = 5% of Live Load
— Group III Loading Wind Skew 15°

Maximum load on corner pile = 92.4ᴷ
Allowable load = 6ᴷ × 12.35 × 125% = 92.6ᴷ

NOTE: Pier footing on ledge.
Maximum pressure at corner = 13.7ᴷ



PART SIDE ELEVATION

PART END ELEVATION

Pier No.	Station ℄ Pier	Bridge Seat Elev.	Top Pier Shaft Elev.	Height of Pier Shaft	Top of Footing Elev.	Bottom of Footing Elev.	Footing Depth	Steel H-Piles 10×42# Req'd.	Pile cut-off Elev.	Est. Length of Piles	Remarks
1	386+55	100.28	95.28	9'-7"	85.69	82.69	3'-0"	—	—	—	Rock excavation required
2	387+15	100.45	95.45	12'-7"	82.86	79.86	3'-0"	—	—	—	" " "
3	387+76	100.22	95.22	14'-9"	80.47	77.47	3'-0"	—	—	—	Concrete fill required
4	388+37	99.61	94.61	16'-11"	77.69	74.69	3'-0"	—	—	—	Rock excavation required
5	388+97	98.63	93.63	18'-4"	75.29	71.29	4'-0"	16	72.29	15'-0"	16-Pile Caps required
6	389+58	97.38	92.38	19'-6"	72.88	68.88	4'-0"	16	69.88	25'-0"	16- " " "
7	390+22	95.86	90.86	19'-11"	70.94	66.94	4'-0"	16	67.94	30'-0"	16- " " "
8	390+85	94.45	89.45	20'-7"	68.86	65.86	3'-0"	—	—	—	Rock excavation required
9	391+49	93.01	88.01	21'-1"	66.92	63.92	3'-0"	—	—	—	" " "
10	392+13	91.58	86.58	21'-2"	65.41	61.41	4'-0"	16	62.41	20'-0"	16-Pile Caps required
11	392+76	90.16	85.16	21'-3"	63.91	59.91	4'-0"	16	60.91	15'-0"	16- " " "

DESIGN – HAMILTON
TRACE – CLARK
CHECK – A.B.P.

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH

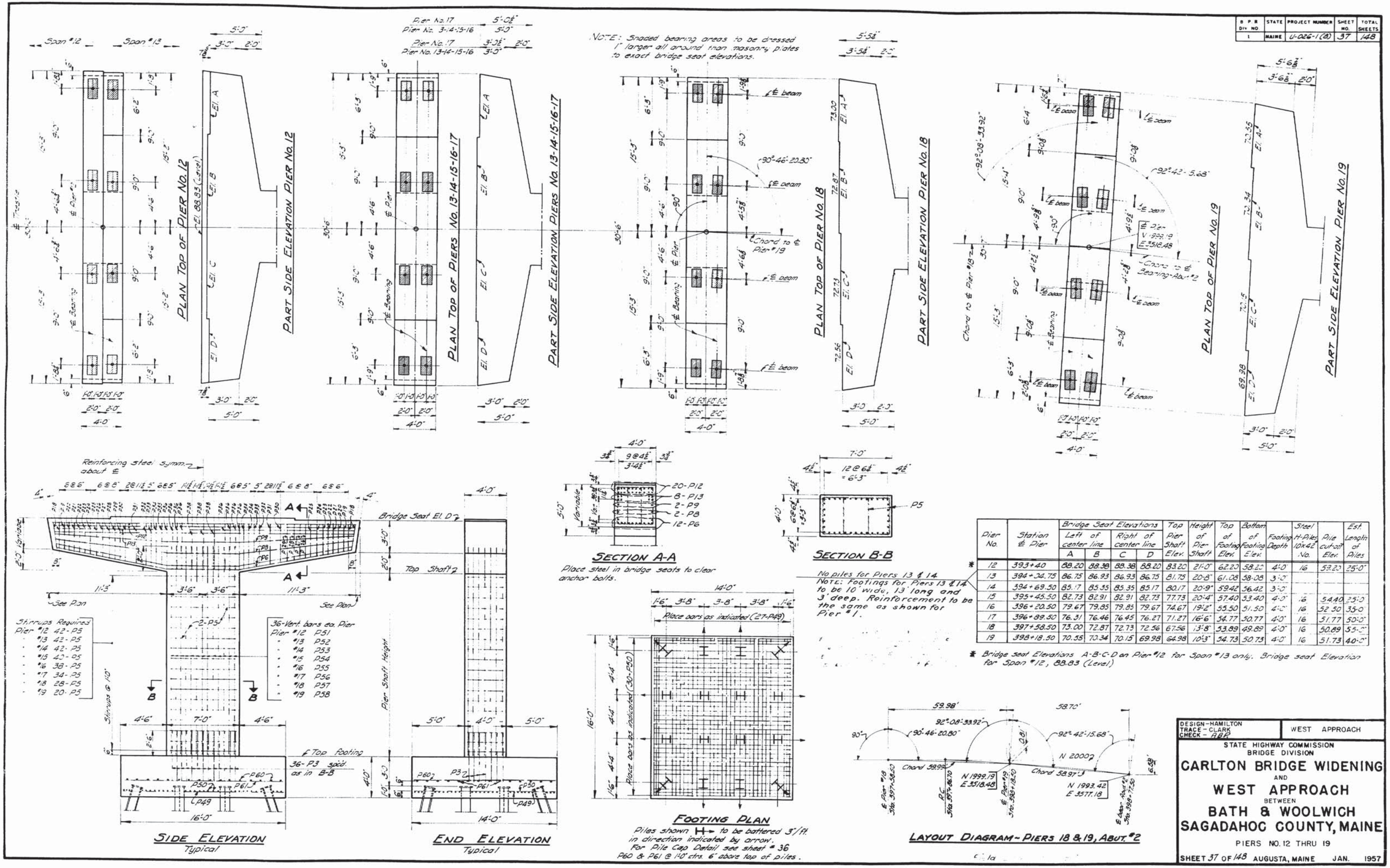
BETWEEN

BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

PIERS NO. 1 THRU 11

SHEET 36 OF 148 AUGUSTA, MAINE JAN. 1957

B.P.R. DIV. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	37	148



Pier No.	Station ℄ Pier	Bridge Seat Elevations Left of center line A	B	Right of center line C	D	Top Pier Shaft Elev.	Height of Pier Shaft	Top of Footing Elev.	Bottom of Footing Elev.	Footing Depth	Steel H-Piles 10x42 No.	Pile cut-off Elev.	Est. Length of Piles
* 12	393+40	88.20	88.38	88.38	88.20	83.20	21'-0"	62.20	58.20	4'-0	16	53.20	25'-0"
13	394+34.75	86.75	86.93	86.93	86.75	81.75	20'-8"	61.08	58.08	3'-0			
14	394+69.50	85.17	85.35	85.35	85.17	80.17	20'-9"	59.42	56.42	3'-0			
15	395+45.50	82.73	82.91	82.91	82.73	77.73	20'-4"	57.40	53.40	4'-0	16	54.40	25'-0
16	396+20.50	79.67	79.85	79.85	79.67	74.67	19'-2"	55.50	51.50	4'-0	16	52.50	35'-0
17	396+89.50	76.31	76.46	76.45	76.27	71.27	16'-6"	54.77	50.77	4'-0	16	51.77	50'-0"
18	397+58.50	73.00	72.87	72.73	72.56	67.56	13'-8"	53.89	49.89	4'-0	16	50.89	55'-0"
19	398+18.50	70.35	70.34	70.15	69.98	64.98	10'-3"	54.73	50.73	4'-0	16	51.73	40'-0"

* Bridge seat Elevations A-B-C-D on Pier #12 for Span #13 only. Bridge seat Elevation for Span #12, 88.83 (Level)

DESIGN - HAMILTON
TRACE - CLARK
CHECK - ABP

WEST APPROACH

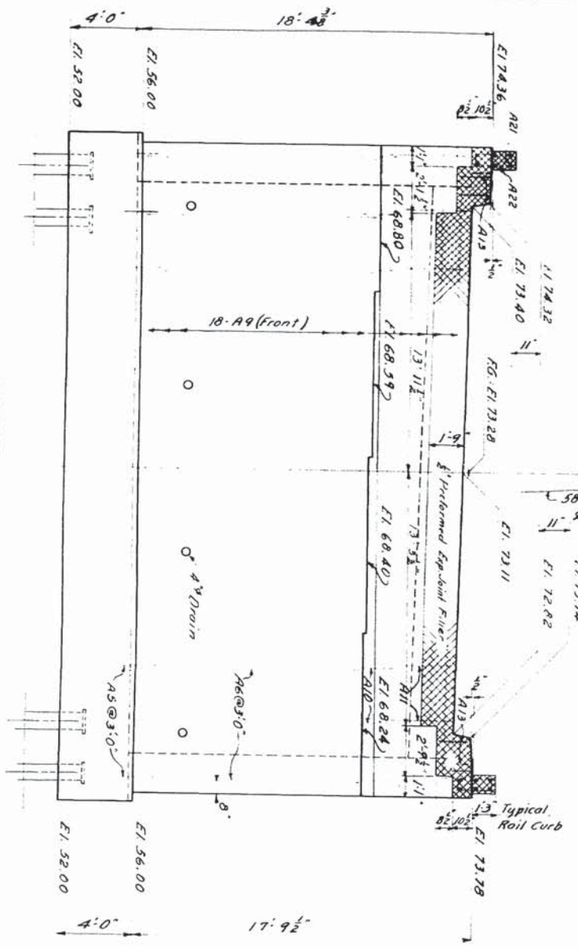
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

PIERS NO. 12 THRU 19

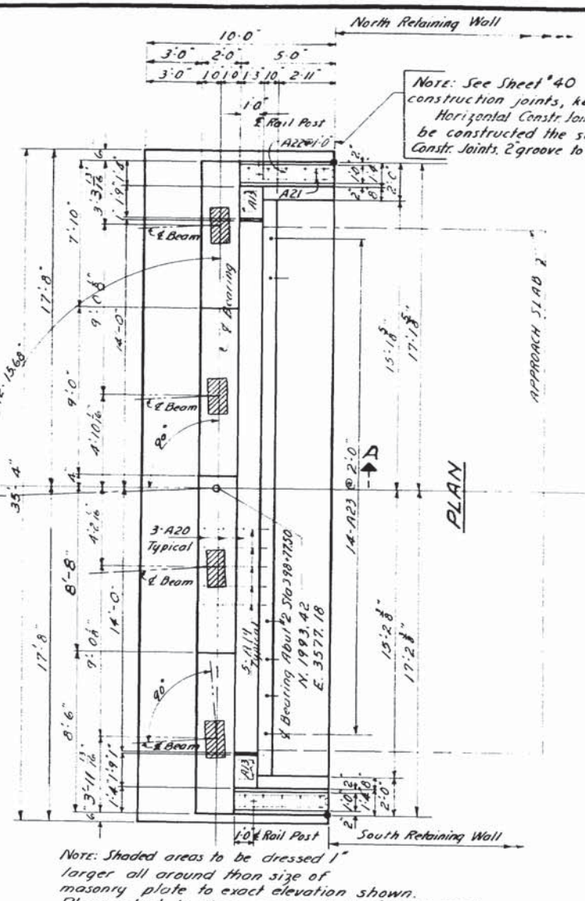
SHEET 37 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div. No.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	38	148



FRONT ELEVATION

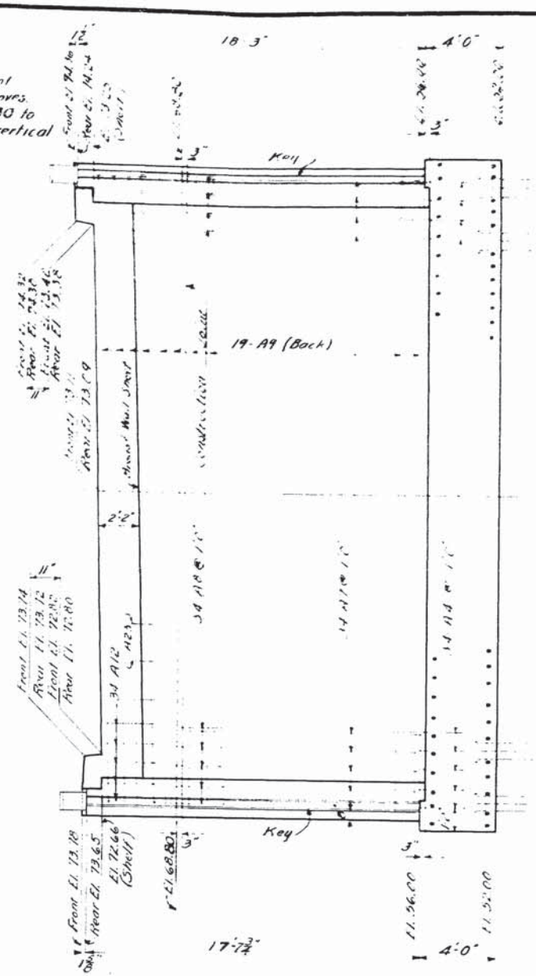
NOTE: Exact location of 4" drains to be determined by the Engineer in the field. Place stone conforming to Stone for French Drains, Special all abutment drains. To be paid for as Granular Backfill.



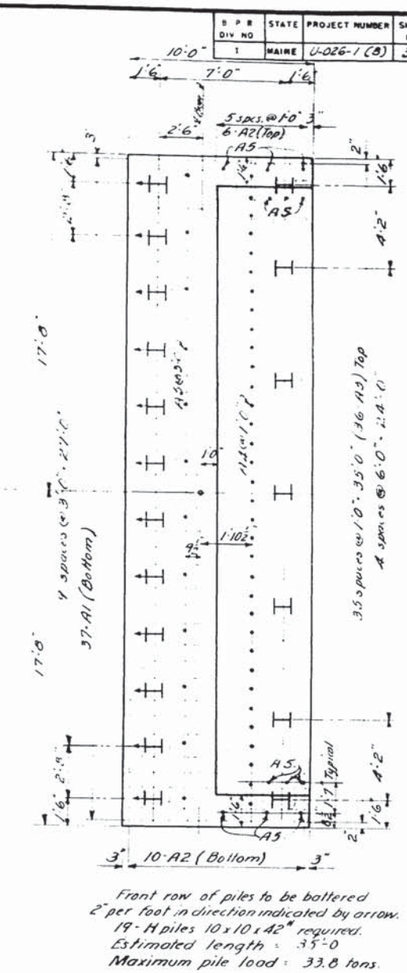
Note: See Sheet #40 for detail of construction joints, key and 2" grooves. Horizontal Constr. Joint at El. 68.80 to be constructed the same as the vertical Constr. Joints. 2" groove to be omitted.

PLAN

Note: Shaded areas to be dressed 1" larger all around than size of masonry plate to exact elevation shown. Place steel to clear anchor bolts (A19 & A20)



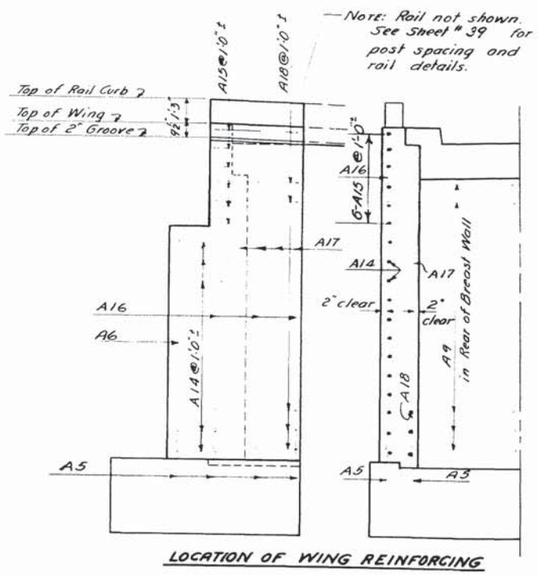
REAR ELEVATION



FOOTING PLAN-PILE AND STEEL LAYOUT

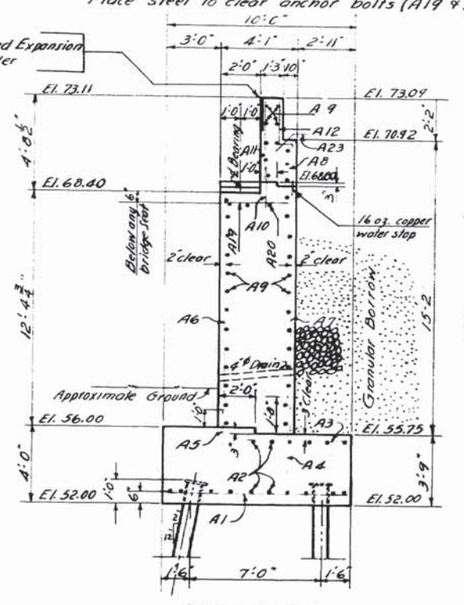
Bend Alinere necessary to clear rail bars.

Front row of piles to be battered 2" per foot in direction indicated by arrow.
19-H piles 10x10x42# required.
Estimated length = 35'-0
Maximum pile load = 33.8 tons.



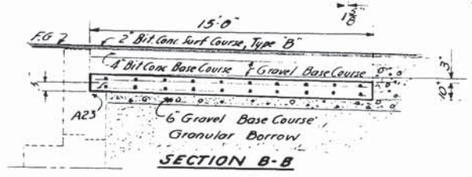
Note: Rail not shown. See Sheet # 39 for rail post spacing and rail details.

LOCATION OF WING REINFORCING

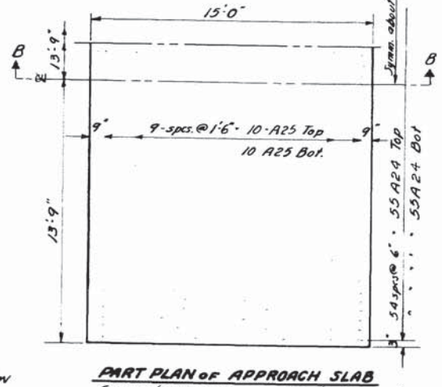


SECTION A-A

Note: Bars A19 & A20 to be placed 6" below bridge seat elevation at beam bearing areas.

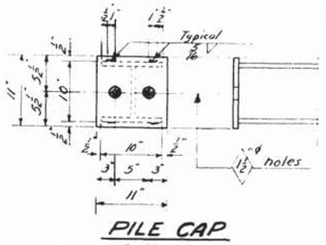


SECTION B-B



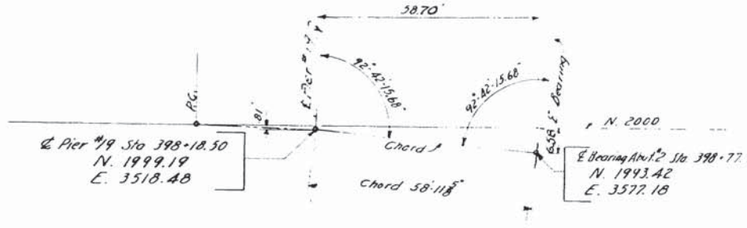
PART PLAN OF APPROACH SLAB

Concrete in approach slab to be paid for under Item 701-40 "Portland Cement Concrete in Roadway & Sidewalk Slabs on Steel Bridges"



PILE CAP

Plate 11" x ¾" x 0'-11" - 19 req'd.



LAYOUT ABUTMENT NO. 2

DESIGN - HAMILTON
TRACE - SAVAGE
CHECK -

WEST APPROACH

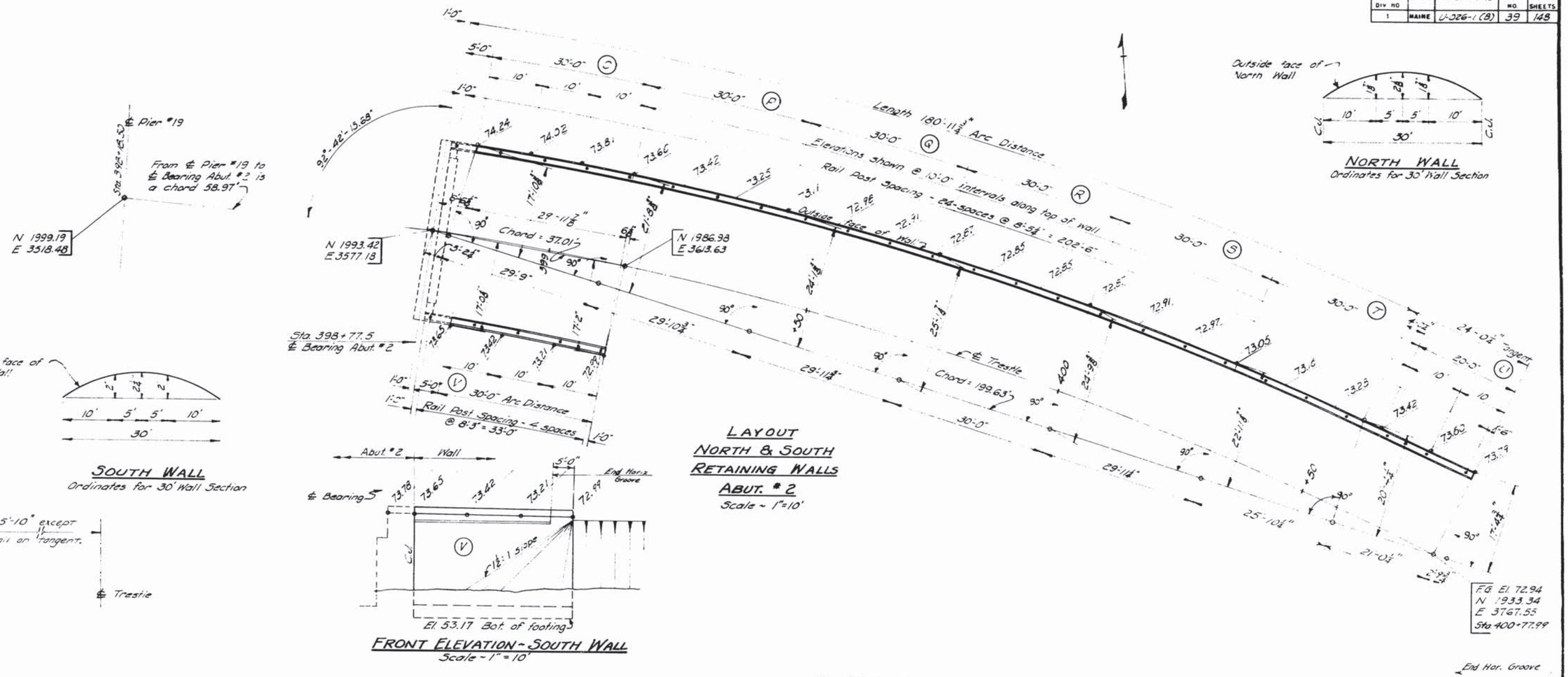
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

ABUTMENT NO. 2

SHEET 38 OF 148 AUGUSTA, MAINE JAN. 1957.

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	U-326-1 (8)	39	148



DESIGN - HAMILTON
TRACE - CLARK
CHECK - [illegible]

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

NORTH AND SOUTH RETAINING WALLS
ABUTMENT NO. 2

SHEET 39 OF 148 AUGUSTA, MAINE JAN. 1957

APPENDIX E

Calculations

Bearing Resistance Calculations

HALEY & ALDRICH

CALCULATIONS

		File No.	38751-030
		Sheet	1 of 3
Client	Maine Department of Transportation	**Date**	4/8/2013
Project	Route 1 Viaduct Bridge Replacement - Bath, Maine	**Computed By**	BCS
Subject	Abutment/Pier Footing Bearing Resistance Evaluation	**Checked By**	EFW

OBJECTIVE:

Evaluate bearing resistance for existing bridge abutments/piers bearing on bedrock in accordance with AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG).

REFERENCES:

1 AASHTO LRFD Bridge Design Specifications 6th Edition 2012.
2 MaineDOT Bridge Design Guide 2003, with Interim Revisions through August 2008.
3 Report entitled, "Subsurface Investigations For Carlton Bridge and West Approach in Bath, Maine," prepared by the State Highway Commission Soils Division, dated April 1957.
4 Historic bridge plans entitled, "Carlton Bridge Widening and West Approach between Bath & Woolwich, Sagadahoc County, Maine," Sheet Nos. 36 through 39 of 148, prepared by the State Highway Commission Bridge Division, dated January 1957.

SUMMARY OF EXISTING BRIDGE STRUCTURE:

Based on review of historic information provided by MaineDOT (references 3 and 4), the existing viaduct bridge is supported on two abutments and 19 piers. Select abutments/piers are supported on spread footings.

SUMMARY OF BEDROCK DATA:

BEDROCK CORE DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Rock Core Length (ft)	Recovery		RQD	
					in.	%	in.	%
BB-BROV-101	R1	12.3	17.2	4.9	59	100%	35	59%
BB-BROV-102	R1	10.7	15.9	5.2	62	100%	53	85%
BB-BROV-103	R1	49.0	54.0	5.0	60	100%	31	52%

LABORATORY TEST DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Peak Compressive Stress (psi)	Bulk Density (pcf)
BB-BROV-101	R1	14.6	15.0	2,321	174
BB-BROV-102	R1	14.0	14.4	1,522	173

BEDROCK DESCRIPTION:

Moderately hard, slightly weathered, medium to fine-grained chlorite/biotite SCHIST with frequent quartz and pegmatite intrusions. Primary joints dipping at low to moderate angles, one steep secondary joint, occasional moderately weathered joints, very close to closely spaced, tight to open.

Moderately hard, slightly weathered, medium to fine-grained GNEISS. Primary joints dipping at low to moderate angles, few steep secondary joints, very close to moderately closely spaced, tight to open with occasional silt coatings on joint surfaces.

For additional details refer to test boring logs.

BEARING RESISTANCE EVALUATION:

SERVICE LIMIT STATE:

Determine service limit state bearing resistance based on presumptive values provided in Table C10.6.2.6.1-1 (attached).

For foliated metamorphic rock (schist) values range from 60 to 80 kips per square foot (ksf).

For service limit state design use 60 ksf (low end of range).

STRENGTH LIMIT STATE:

Calculate upper and lower bound nominal bearing resistance for spread footings bearing on bedrock in accordance with Section 10.8.3.5.4c.

Intact/Tightly Jointed Rock: upper bound value

qp = 2.5 x qu (Eqn. 10.8.3.5.4c-1)

Where: qu = unconfined compressive strengh of rock (ksf)

From previous page, 1.522 ksi < qu < 2.321 ksi…… average = 277 ksf

qu = 277 ksf

qp = 692 ksf

Jointed Rock: lower bound solution

$qp = ((s)^{1/2} + (m(s)^{1/2} + s)^{1/2}) \times qu$ (Eqn. 10.8.3.5.4c-2)

Where: s,m = fractured rock mass parameters specified in Table 10.4.6.4-4.

parameters s and m are a function of rock quality as determined use the Rock Mass Rating (RMR) system.
Determine rock quality in accordance with Tables 10.4.6.4-1 and 10.4.6.4-2 (see attached spreadsheet summary).

Test Boring No.	Sample No.	Recovery (%)	RQD (%)	RMR
BB-BROV-101	R1	100%	59	31
BB-BROV-102	R1	100%	85	50
BB-BROV-103	R1	100%	52	31

For SCHIST and GNEISS bedrock assume Rock Types B and E, respectively. Say Rock Type B is predominant….

BEARING RESISTANCE EVALUATION: (CONT.)

STRENGTH LIMIT STATE: (CONT.)

Jointed Rock: lower bound solution

Calculate rock mass constants (m and s) directly from Eqns. 18 and 19 (Hoek and Brown, 1988)

m / mi = e^((RMR-100/14))…. m = mi x e^((RMR-100)/14)

s = e^((RMR-100)/6))

For Rock Type B and intact rock from Table 1 (Hoek and Brown, 1988)….. mi = 10

Calculate m for RMR = 41 (average of 31 and 50)

m = 0.15

s = 0.000054

qp = $((s)^{1/2} + (m(s)^{1/2} + s)^{1/2})$ x qu (Eqn. 10.8.3.5.4c-2)

qp = 11.4 ksf

Based on review of rock core samples, rock more closely resembles "jointed rock" condition. However, the calculated nominal bearing resistance appears low based on experience. Compare to the presumptive summarized previously. Calculate average of presumptive value and nominal value for "jointed rock"….

Calculate average nominal bearing resistance = 41.4 ksf

Resistance factor for bearing resistance of footings on rock, ϕ = 0.45

Calculate factored bearing resistance = 18.6 ksf

Recommend abutment footings bearing on bedrock be designed based on a factored bearing resistance (Strength Limit State) equal to 20 ksf.

EXTREME EVENT LIMIT STATE:

Recommend abutment footings bearing on bedrock be designed based on a factored bearing resistance (Extreme Event Limit State) equal to 44 ksf.

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. _____ X G/S ☐ Mudline

Core Barrel Type NQ Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name Route 1 Viaduct Bridge Replacement

Drilling Contractor Maine Test Borings

Project Location Bath, Maine

Drill Rig Type Mobile Drill B-53 Truck-Mounted Drill Rig

File Number 38751-030

Client Maine Department of Transportation

Haley & Aldrich Rep. Marleigh Snow

Date 4/19/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-BROV-101	R1	12.3	17.2	15.1	10.2	Schist	334	2	59	100%	34	58%	13	2 in	12 in	10			6	-7	7	**31**
BB-BROV-102	R1	10.7	15.9	6.5	1.3	Gneiss	219	2	62	100%	53	85%	17	2 in	39 in	25			6	-7	7	**50**
BB-BROV-103	R1	49.0	54.0	-41.2	-46.2	Schist	277	2	60	100%	31	52%	13	2 in	12 in	10			6	-7	7	**31**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Condition of Joints

	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions

	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: **BB-BROV-101,102,103**

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR, as specified in Article 10.4.6.4. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-101
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.4	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	NW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	NW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131000, N393314 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.8
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							1		-BITUMINOUS CONCRETE-	
	1D	24/12	1.0 - 3.0	6/7/6/25	13	13	SSA	26.4	1.0 Brown, dry, medium dense, medium to fine SAND, little coarse sand, trace silt, little gravel -FILL-(SW) Note: Probable cobble from approximately 3.0 to 4.3 ft.	
	2D	24/22	4.3 - 6.3	4/6/6/5	12	12		23.1	4.3 Brown-gray, dry to moist, stiff mottled SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5	3D	24/22	6.3 - 8.3	6/6/6/5	12	12		21.1	6.3 Brown-gray, moist, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
	4D V1A	24/22	8.3 - 10.3	1/2/3/3 ~~Su>1940 psf~~	5	5	4	18.1	Brown, moist, medium stiff, silty CLAY with sand layers 55x110 mm vane raw torque reading: V1A: >500 in/lbs	
10							13		9.3 Brown, clayey, medium to fine SAND, little coarse sand, little gravel -GLACIAL TILL-(SC/SM)	
							42	15.4	12.0	
	R1	59/59	12.3 - 17.2	RQD = 59%			26(2.0")	15.1	-WEATHERED BEDROCK- 12.3 Top of Bedrock at El. 15.1 Gray, medium to fine-grained chlorite, biotite, SCHIST with frequent quartz and pegmatitic intrusions. Moderately hard, slightly weathered with occasional moderately weathered joints, low angle to moderately dipping, one steep secondary joint, very close to close, tight to open Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 12.3-13.3' (1:45); 13.3-14.3' (2:33); 14.3-15.3' (1:53); 15.3-16.3' (2:26); 16.3-17.2' (3:04)	GTX#300397 qp=2,321 psi
15								10.2	17.2 **Bottom of Exploration at 17.2 feet below ground surface.**	
20										
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-102
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	17.2	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131318, N393408 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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 = Insitu Vane Shear Test
MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.5 - 2.5	18/16/7/6	23	23	SSA	16.8	-BITUMINOUS CONCRETE- 0.4	
									Brown, dry, medium dense, fine to medium SAND, some coarse sand, trace silt, little gravel -FILL-(SW) 2.5	
	2D	24/14	2.5 - 4.5	5/10/11/16	21	21		14.7	Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	3D	24/22	4.5 - 6.5	9/10/10/10	20	20			Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5										
	4D	24/20	6.5 - 8.5	13/14/20/21	34	34			Brown-gray, dry to moist, hard, clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	5D	20/2	8.5 - 10.2	9/16/28/40(3.0")	44	44			Note: Recovered 2-in. brown, dry, dense, weathered rock.	
10	R1	62/62	10.7 - 15.9	RQD = 85%				7.2	10.0	
								6.5	-WEATHERED BEDROCK- 10.7	
									Top of Bedrock at El. 6.5 Tan-white-gray, medium to fine-grained GNEISS. Moderately hard, slightly weathered, joints low angle to moderately dipping, few steep secondary joints, very close to moderately close, tight to open, occasional silt coatings on joint surfaces. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 10.7-11.7' (2:08); 11.7-12.7' (1:08); 12.7-13.7' (1:19); 13.7-14.7' (1:31); 14.7-15.9' (1:00)	GTX#300397 qp=1,522 psi
15								1.3	15.9	
20										
25										
30										

Bottom of Exploration at 15.9 feet below ground surface.

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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

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

Boring No.: BB-BROV-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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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MV = Unsuccessful Insitu 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 = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.5 - 2.5	6/20/30/6	50	50	SSA	7.7	-BITUMINOUS CONCRETE- 0.1	
									Brown, dry, dense, medium to fine SAND, little coarse sand, little gravel, trace silt -FILL-(SW)	
	2D	24/10	2.5 - 4.5	3/2/2/1	4	4		4.8	3.0	
									Dark brown, moist, soft, SILT, little organics (woody/fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	3D	24/24	4.5 - 6.5	1/1/1/1	2	2			Dark brown, moist, soft, SILT with organics, trace fine sand -ORGANIC DEPOSIT-(ML)	
5										
	4D	24/24	6.5 - 8.5	2/2/3/5	5	5			Dark brown, moist, medium stiff, SILT with organics (fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	5D	24/24	8.5 - 10.5	3/5/7/8	12	12		-0.7	8.5	
									Brown-gray, moist, stiff, mottled clayey SILT with organics (fibrous) -MARINE DEPOSIT-(ML)	
10	6D	24/24	10.5 - 12.5	13/12/14/12	26	26	36		Brown-gray, moist, very stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							35			
	7D	24/12	12.5 - 14.5	4/5/6/7	13	13	34		Note: Attempted field vane shear test at 12.0 ft, could not push. Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
							49			
15	8D	24/24	15.0 - 17.0	3/4/5/4	9	9	45		Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
	9D	24/18	17.0 - 19.0	1/2/2/2	4	4	40	-9.2	17.0 Gray, moist to wet, medium stiff, silty CLAY (CL) -MARINE DEPOSIT-(CL)	
							38			
	10D	24/24	19.0 - 21.0	WOH/WOH/WOH/ WOH			40		Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
20	V1A		19.6 - 20.0	Su=855/20 psf			51		55x110 mm vane raw torque readings: V1A: 220/6 in-lbs	
	U1		21.0 - 23.0				45			C#IP-1 WC=41.9% LL=41 PL=19, PI=22
							39			
	V2A		23.6 - 24.0	Su=700/95 psf			40		55x110 mm vane raw torque readings: V2A: 178/26 in-lbs	
	V2B		24.6 - 25.0	Su=640/115 psf			34		V2B: 165/30 in-lbs	
25	11D	24/24	25.0 - 27.0	WOR/WOR/WOR/ WOR			44		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							40			
							27			
	V3A		28.6 - 29.0	Su=580/80 psf			19		55x110 mm vane raw torque readings: V3A: 150/21 in-lbs	
30	V3B		29.6 - 30.0	Su=580/80 psf			13			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

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: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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T_v = Pocket Torvane Shear Strength (psf)
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	U2		30.0 - 32.0				27		V3B: 150/20 in-lbs Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-3 WC=47.7% LL=38 PL=19, PI=19
							19			
	V4A		32.6 - 33.0	Su=600/40 psf			19		55x110 mm vane raw torque readings: V4A: 154/12 in-lbs V4B: 170/32 in-lbs	
	V4B		33.6 - 34.0	Su=660/115 psf			21			
							24			
35	12D	24/24	35.0 - 37.0	WOR/WOR/WOR/ WOR			23		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							25			
							22			
	V5A		38.6 - 39.0	Su=505/135 psf			25		55x110 mm vane raw torque readings: V5A: 130/34 in-lbs V5B: 127/11 in-lbs	
	V5B		39.6 - 40.0	Su=485/40 psf			27			
40							30			
	U3		41.0 - 43.0				30		Gray, wet, soft to medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-2 WC=40.6% LL=37 PL=19, PI=18
							25			
	V6A		43.6 - 44.0	Su=525/115 psf			30		55x110 mm vane raw torque readings: V6A: 137/31 in-lbs V6B: 150/33 in-lbs	
	V6B		44.6 - 45.0	Su=580/135 psf			35			
45							41			
							41			
							51	-39.0	46.8	
							70		Note: Sand and gravel observed in wash water at approximately 46.8 ft. -PROBABLE GLACIAL TILL-	
	13D	0/0	49.0 - 49.0	50(0")				-41.2	49.0	
50	R1	60/60	49.0 - 54.0	RQD = 52%					Top of Bedrock at El.-41.2 Gray, black and white, coarse to medium-grained pegmatitic SCHIST. Moderately hard, fresh to slightly weathered, joints low angle to moderately dipping, very close to close, tight to open, occasional weathered joint surfaces and silt coatings. Rock Mass Quality=Fair Recovery=100% R1 Core Times (min:sec): 49.0-50.0' (2:25); 50.0-51.0' (3:25); 51.0-52.0' (1:58); 52.0-53.0' (2:00); 53.0-54.0' (2:08)	
								-46.2		
55									54.0 **Bottom of Exploration at 54.0 feet below ground surface.**	
60										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

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.

Existing Foundation Evaluations

EVALUATION OF EXISTING FOOTINGS AND PILES

PROVIDED BY FIGG
NO ABUT. COMP. LOADS PROVIDED

Bath Approaches, Bath Maine
Factored Compressive Loads at Piers

4/12/2013

CALCULATED USING FACTORED COMP. LOADS AND NO. OF EXISTING PILES SHOWN BELOW, SEE ALSO TABLE II AND HISTORIC BRIDGE DWGS.

CALCULATED USING FACTORED COMP. LOADS AND EXISTING FOOTING DIMENSIONS SHOWN BELOW, SEE ALSO TABLE I AND HISTORIC BRIDGE DWGS.

DOESN'T INCLUDE COMBINED (LAT./BEN EFFEC

Pier	Contributing Span Length (FT)	Factored Compressive Loads (KIPS)	EXISTING SUBSTR. TYPE	CONTACT PRESSURE (KSF)	LOAD PER PILE (KIP)
1	60.00	890	FOOTINGS	6.85	
2	60.50	897		6.90	
3	61.00	904		6.95	
4	60.50	897		6.90	
5	60.50	897	PILES		56
6	62.50	925			58
7	63.50	939			59
8	63.50	939	FOOTINGS	7.22	
9	64.00	946		7.28	
10	63.50	939	PILES		59
11	63.50	939			59
12	64.38	951			59
13	64.75	956	FOOTINGS	7.35	
14	70.38	1035		7.96	
15	75.50	1106	PILES		69
16	72.00	1057			66
17	69.00	1016			64
18	64.50	953			60
19	59.50	883			55

EXISTING (SEE HISTORIC BRIDGE DWGS)
PIER: MAX. LOAD ON CORNER PILE = 92.4
ABUT: MAX. PILE LOAD = 67.6 K

EXISTING FOOTING DIMENSIONS: SEE TABLE I & HISTORIC BRIDGE DWGS.

L = 10 FT
W = 13 FT
T = 3 FT

AREA = (10 FT)(13 FT) = 130 FT²

CONTACT PRESSURE: 6.85 KSF → 7.96 KSF
FACTORED BEARING RESISTANCE: 20 KSF

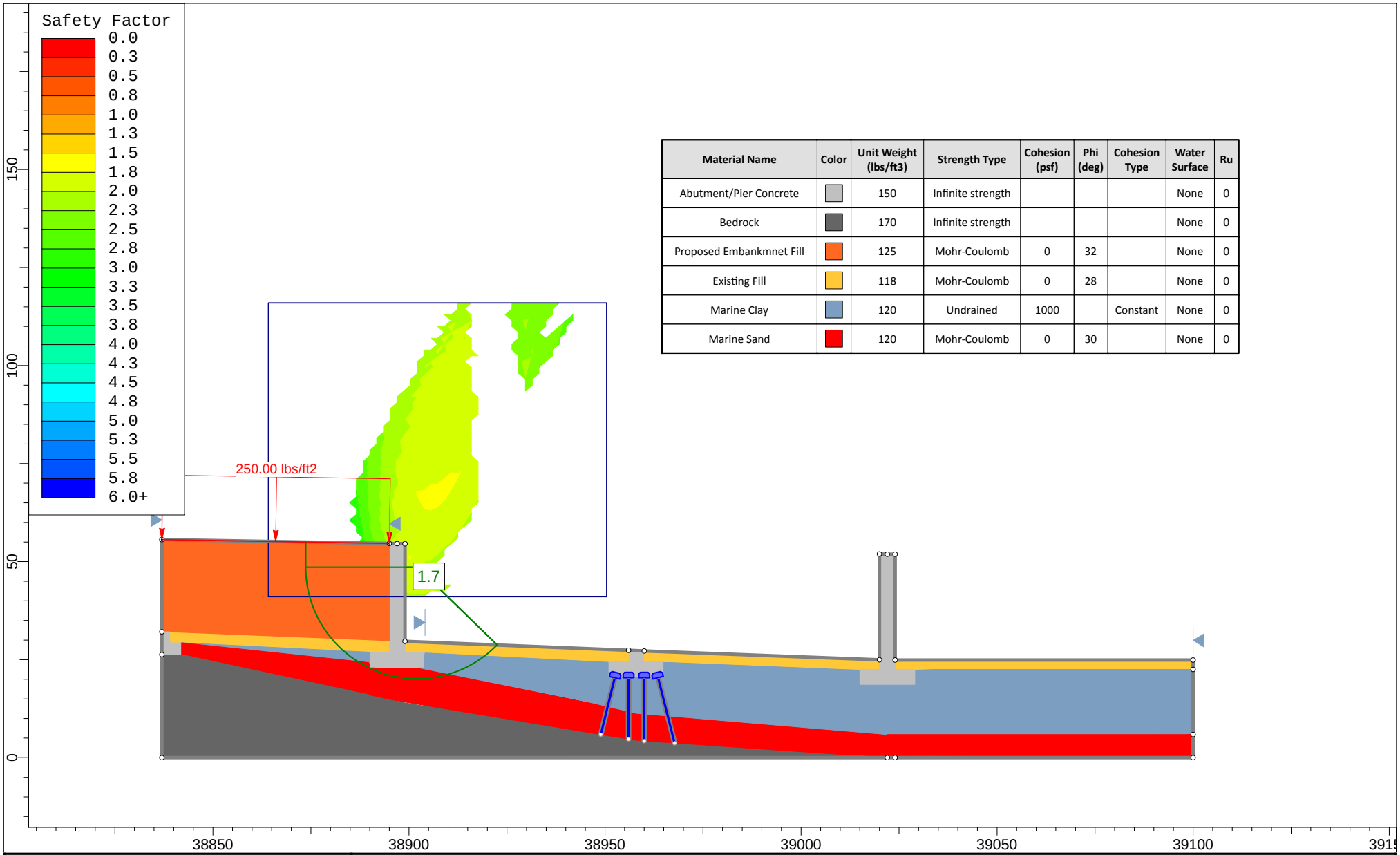
EXISTING NO. OF PILES: SEE TABLE II & HISTORIC BRIDGE DWGS

PIER = 16 PILES

ABUTMENT = 19 PILES

COMP. LOAD PER PILE: 55 KIPS → 69 KIPS
MAX. PILE LOAD (PIER): 92.4 KIPS

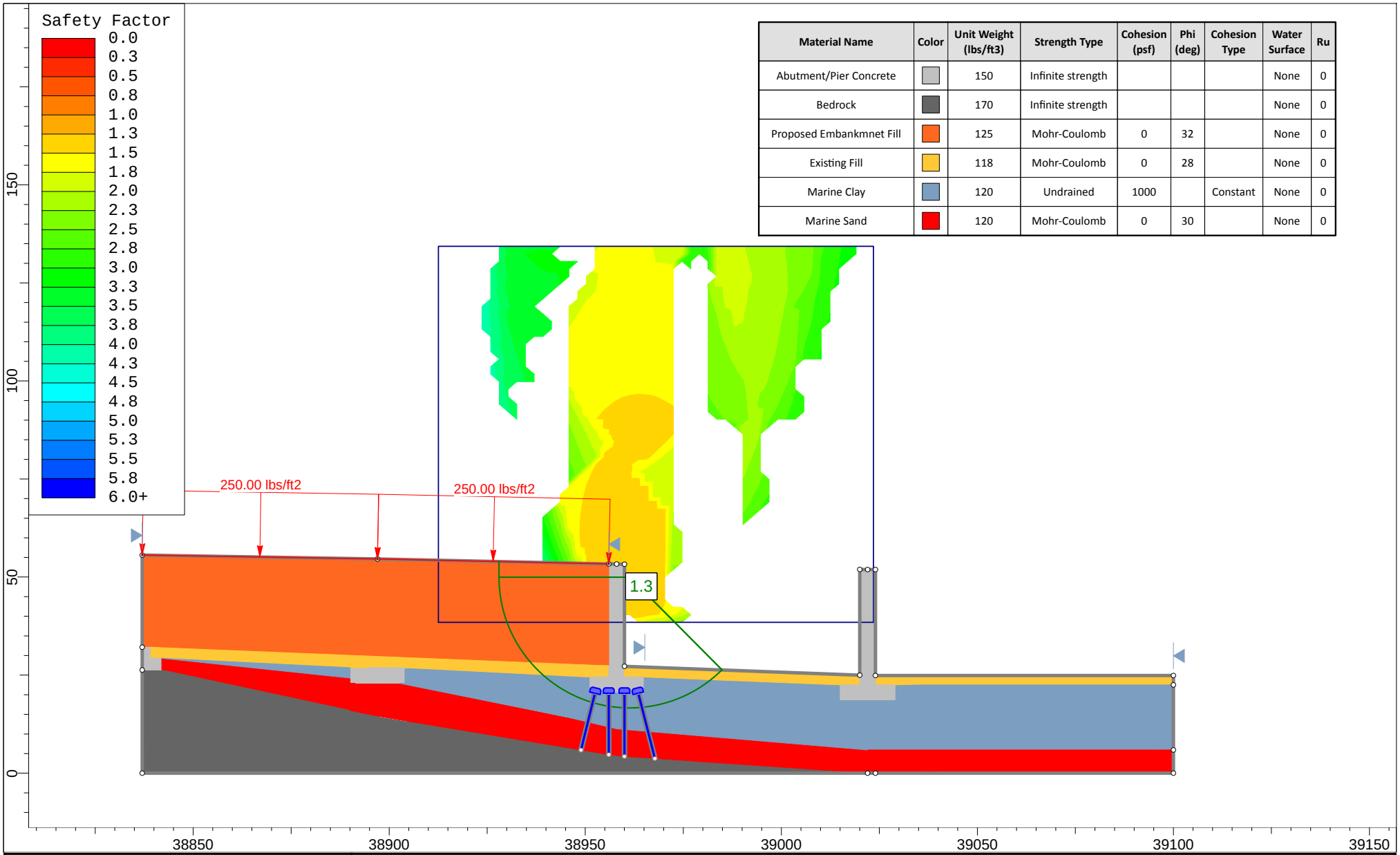
Settlement and Stability Evaluations



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Abutment/Pier Concrete		150	Infinite strength				None	0
Bedrock		170	Infinite strength				None	0
Proposed Embankmnet Fill		125	Mohr-Coulomb	0	32		None	0
Existing Fill		118	Mohr-Coulomb	0	28		None	0
Marine Clay		120	Undrained	1000		Constant	None	0
Marine Sand		120	Mohr-Coulomb	0	30		None	0

SLIDEINTERPRET 6.024

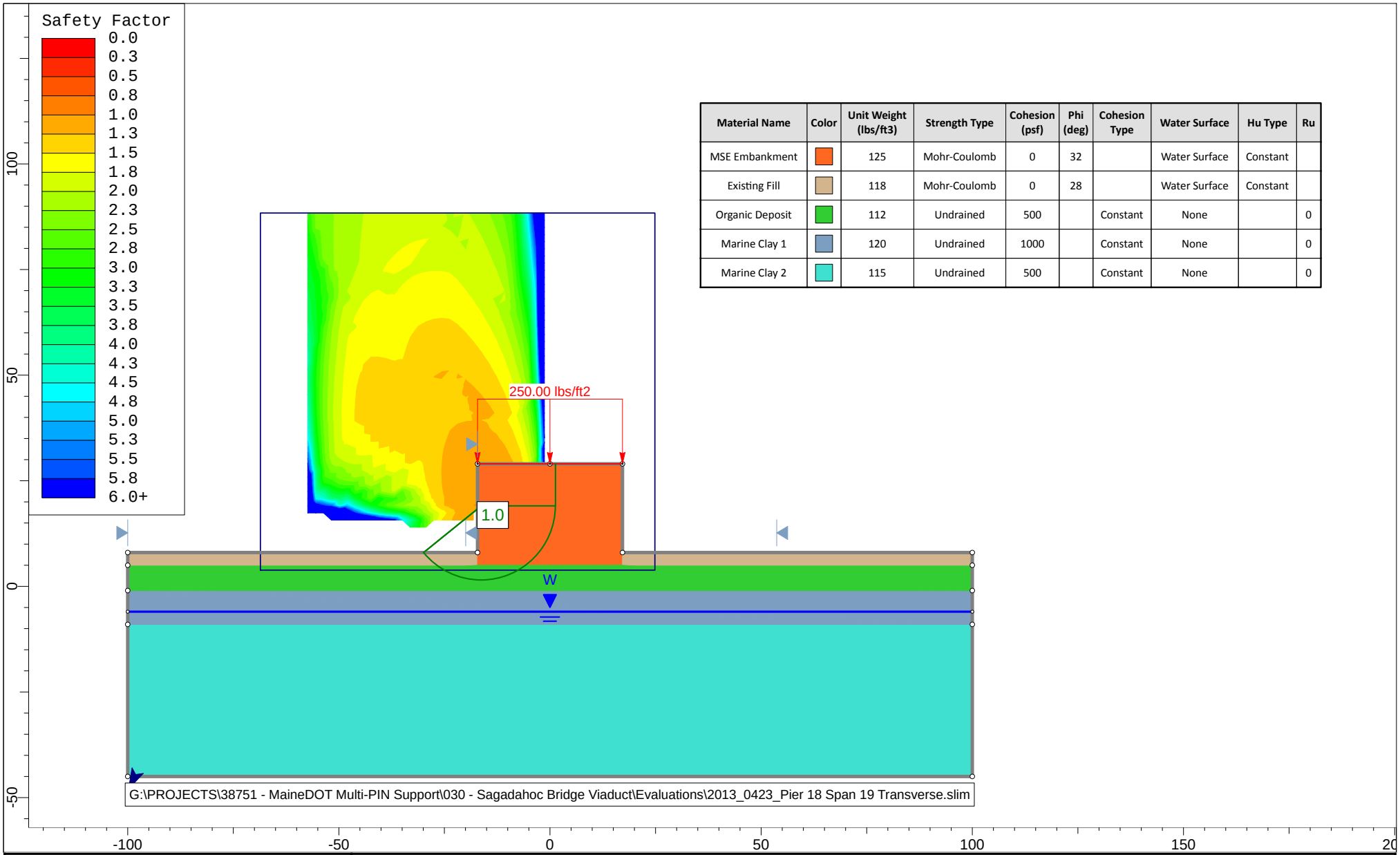
Project	Route 1 Viaduct Bridge Replacement		
Analysis Description	Span 1-5 Infill Parallel to Baseline		
Drawn By Bryan C. Steinert, P.E.	*Scale* 1:406	*Company* Haley & Aldrich, Inc.	
Date 4/23/2013, 9:33:14 AM		*File Name* 2013_0423_HAI_Abutment 1 Longitudinal_Pier 5.slim	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Ru
Abutment/Pier Concrete		150	Infinite strength				None	0
Bedrock		170	Infinite strength				None	0
Proposed Embankmnet Fill		125	Mohr-Coulomb	0	32		None	0
Existing Fill		118	Mohr-Coulomb	0	28		None	0
Marine Clay		120	Undrained	1000		Constant	None	0
Marine Sand		120	Mohr-Coulomb	0	30		None	0

SLIDEINTERPRET 6.024

Project	Route 1 Viaduct Bridge Replacement		
Analysis Description	Span 1-5 Infill Parallel to Baseline		
Drawn By Bryan C. Steinert, P.E.	*Scale* 1:406	*Company* Haley & Aldrich, Inc.	
Date 4/23/2013, 9:33:14 AM		*File Name* 2013_0423_HAI_Abutment 1 Longitudinal_Pier 6.slim	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Water Surface	Hu Type	Ru
MSE Embankment		125	Mohr-Coulomb	0	32		Water Surface	Constant	
Existing Fill		118	Mohr-Coulomb	0	28		Water Surface	Constant	
Organic Deposit		112	Undrained	500		Constant	None		0
Marine Clay 1		120	Undrained	1000		Constant	None		0
Marine Clay 2		115	Undrained	500		Constant	None		0

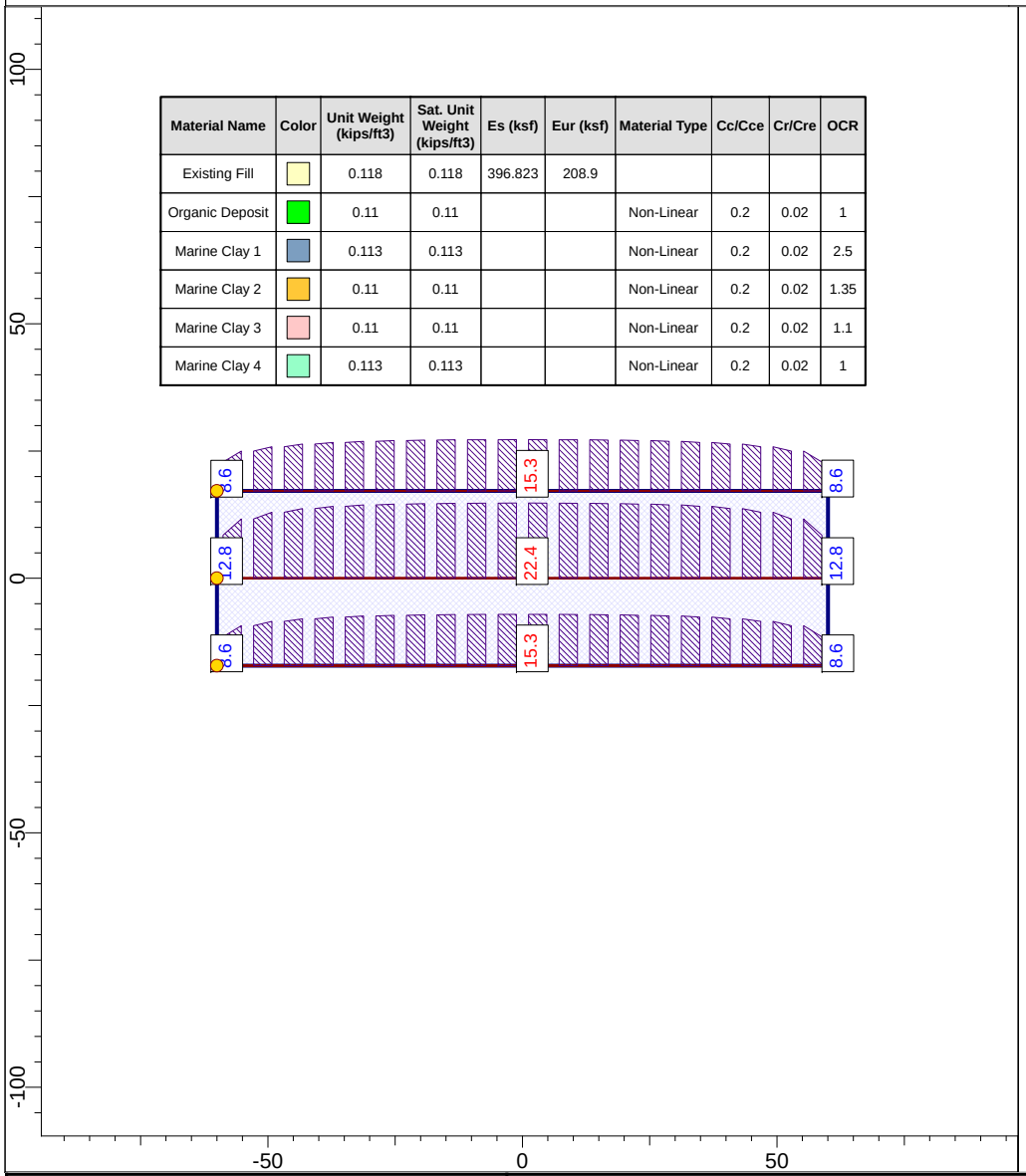
SLIDEINTERPRET 6.024

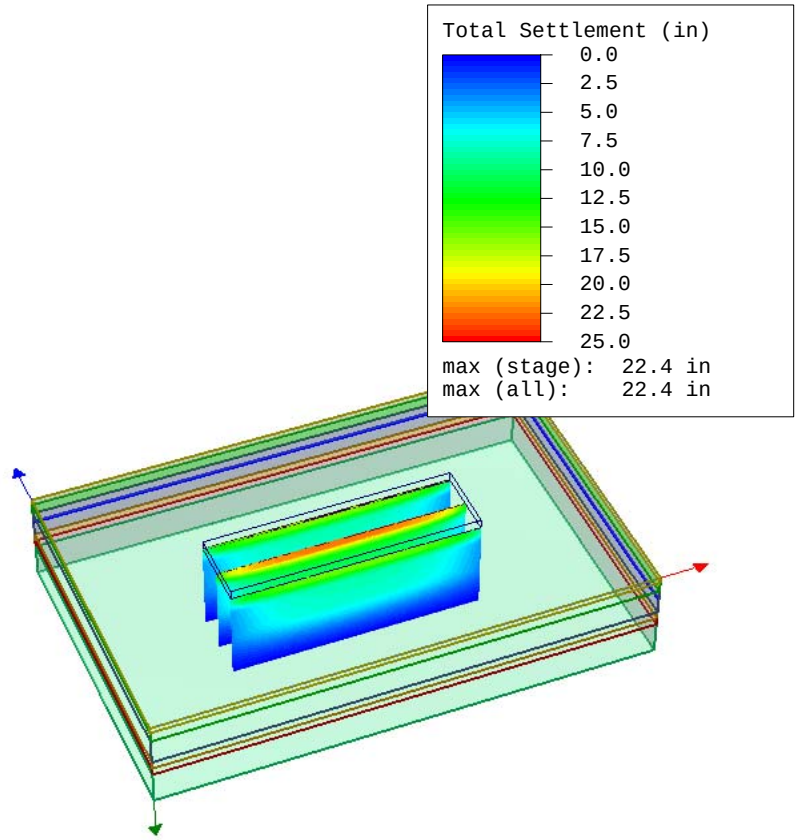
Project	Route 1 Viaduct Bridge Replacement		
Analysis Description	Span 19/20 Infill Transverse to Baseline		
Drawn By Bryan C. Steinert, P.E.	*Scale* 1:377	*Company* Haley & Aldrich, Inc.	
Date 4/23/2013, 12:44:06 PM		*File Name* 2013_0423_Pier 18 Span 19 Transverse.slim	

Stage 1

Data Type: Total Settlement

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	Cc/Cce	Cr/Cre	OCR
Existing Fill		0.118	0.118	396.823	208.9				
Organic Deposit		0.11	0.11			Non-Linear	0.2	0.02	1
Marine Clay 1		0.113	0.113			Non-Linear	0.2	0.02	2.5
Marine Clay 2		0.11	0.11			Non-Linear	0.2	0.02	1.35
Marine Clay 3		0.11	0.11			Non-Linear	0.2	0.02	1.1
Marine Clay 4		0.113	0.113			Non-Linear	0.2	0.02	1





Project	Route 1 Viaduct Bridge Replacement		
Analysis Description	Span 19/20 Infill W/Normal Weight Earthfill		
Drawn By	Bryan C. Steinert, P.E.	Company	Haley & Aldrich, Inc.
Date	4/23/2013, 1:12:04 PM	File Name	2013_0423_HAI_Route 1 Viaduct Bridge Span 19-20 Infill.s3z

SETTLE3D 2.016

HALEY & ALDRICH

Calculations

File No.	38751-030		
Sheet	1 of 4		
Client	MAINEDOT	Date	4/30/2013
Project	ROUTE 1 VIADUCT BRIDGE REPLACEMENT	Computed By	BCS
Subject	SPAN 11 & 12 INFILL (PIER 10-12) SETTLEMENT & STABILITY	Checked By	WAC

- ESTIMATE SETTLEMENT AND FACTOR OF SAFETY AGAINST EMBANKMENT STABILITY RESULTING FROM PLACEMENT OF NORMAL WEIGHT EARTHFILL FROM EXISITING GROUND SURFACE UP TO EXISTING BRIDGE DECK LEVEL. EARTHFILL WILL BE RETAINED BY VERTICAL-SIDED RETAINING WALLS (MSE WALLS).

- SUBSURFACE CONDITIONS, STRENGTH PROPERTIES AND STRESS HISTORY AND COMPRESSIBILITY CHARACTERISTICS TAKEN FROM THE FOLLOWING HISTORIC TEST BORINGS:

  BORING 14 (PIER 10)
  BORING 7 (PIER 12)

SUMMARIZE SUBSURFACE CONDITIONS....

TEST BORING NO.	GS ELEV. (FT)	STRATA THICKNESS (FT) FILL	CLAY	SAND	TOP OF BEDROCK ELEVATION	
14	68.0	0.0	17.5	0.0	EL. 50.5	**ASSUME GW @ 8' BGS
7	64.7	8.0	8.0	8.5	EL. 40.2	

** NOTE: FILL ENCOUNTERED IN TEST BORING 7 DESCRIBED AS "COAL ASHES".

FOR PURPOSES OF FEASIBILITY-LEVEL SETTLEMENT AND STABILITY EVALUATIONS AND CONSIDERING NO RECENT SUBSURFACE INFORMATION IS AVAILABLE, CONSIDER WORSE CASE SOIL PROFILE (GREATEST CLAY THICKNESS) ∴ BORING 14 (PIER 10)

MARINE CLAY LAYER CAN BE SUBDIVIDED INTO TWO SUBLAYERS; SOFT TO MED. CONSISTENCY SILTY CLAY OVERLYING STIFF TO V. STIFF SILTY CLAY. SUMMARIZE THICKNESSES....

SOFT - MED. = 12 FT "LAYER A" ; γ = 112 PCF
STIFF - V. STIFF = 5.5 FT "LAYER B" ; γ = 115 PCF

LIMITED SHEAR STRENGTH DATA FROM HISTORIC TEST BORINGS....

LAYER A: 1 LAB VANE @ 576 PSF SAY 500 PSF

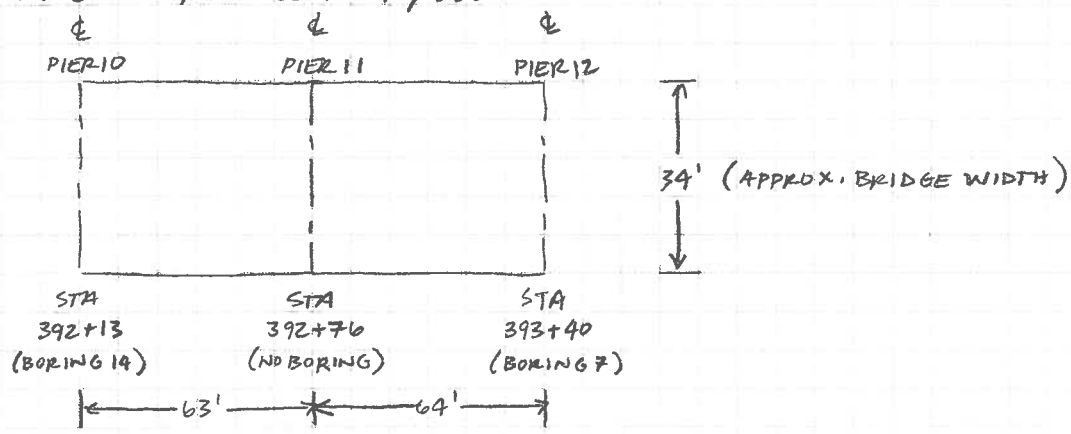
LAYER B: 1 LAB + 1 FIELD VANE @ 2,000 PSF SAY 1,000 PSF (CONSERVATIVE BUT STABILITY LIKELY CONTROLLED BY LAYER A.

HALEY & ALDRICH

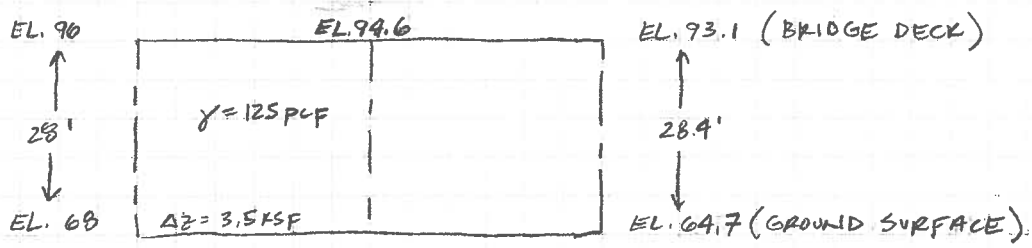
Calculations

File No.	38751-030		
Sheet	2 of 4		
Client	MAINEDOT	Date	4/30/2013
Project	ROUTE 1 VIADUCT BRIDGE REPLACEMENT	Computed By	BCS
Subject	SPAN 11 & 12 INFILL (PIER 10-12) SETTLEMENT & STABILITY	Checked By	WAC

- SUMMARIZE INFILL GEOMETRY....



PLAN
N.T.S.



PROFILE
N.T.S.

- CALCULATE EXISTING VERTICAL EFFECTIVE STRESS @ MIDPOINT OF EACH SUBLAYER....

LAYER A = (6 FT)(112 PCF) = 672 PSF

LAYER B = (8 FT)(112 PCF) + (4 FT)(112 − 62.4 PCF) + (2.75 FT)(115 − 62.4 PCF)
= 1,239 PSF

- CALCULATE STRESS INCREASE @ MIDPOINT OF EACH SUBLAYER.... UNIFORMLY LOADED RECTANGULAR AREA (HOLTZ & KOVACS, 306)

B = 34 FT ; L = 127 FT

LAYER A : z = 6 FT ; LAYER B : z = 15.75 FT

	B/z	L/z	I	4I	Δz
LAYER A	5.7	21.2	0.25	1.0	× 3,500 PSF = 3,500 PSF
LAYER B	2.2	8.1	0.24	0.96	× 3,500 PSF = 3,360 PSF

HALEY & ALDRICH

Calculations

File No.	38751-030		
Sheet	3 of 4		
Client	MAINEDOT	Date	4/30/2013
Project	ROUTE 1 VIADUCT BRIDGE REPLACEMENT	Computed By	BCS
Subject	SPAN 11 & 12 INFILL (PIER 10-12) SETTLEMENT & STABILITY	Checked By	WAC

- ASSUME EACH SUBLAYER IS NORMALLY-CONSOLIDATED (OCR) FOR UPPER BOUND (CONSERV.) SETTLEMENT ESTIMATE

LAYER A:

$$\Delta H = 12'(0.02)(12^{IN}/_{FT}) \log\left(\frac{672+672}{672}\right) = 0.87 \text{ IN.}$$

$$\Delta H = 12'(0.2)(12^{IN}/_{FT}) \log\left(\frac{672+2828}{672}\right) = 20.64 \text{ IN.}$$

LAYER B:

$$\Delta H = 5.5'(0.02)(12^{IN}/_{FT}) \log\left(\frac{1,239+1,239}{1,239}\right) = 0.40 \text{ IN.}$$

$$\Delta H = 5.5'(0.2)(12^{IN}/_{FT}) \log\left(\frac{1,239+2,121}{1,239}\right) = 5.72 \text{ IN.}$$

~ 28 IN.

- ASSUME EACH SUBLAYER HAS OCR = 1.25 .... LAYER A = 840 PSF ; LAYER B = 1,549 PSF

LAYER A:

$$\Delta H = 12'(0.02)(12^{IN}/_{FT}) \log\left(\frac{672+840}{672}\right) = 1.01 \text{ IN.}$$

$$\Delta H = 12'(0.2)(12^{IN}/_{FT}) \log\left(\frac{672+2,660}{672}\right) = 20.03 \text{ IN.}$$

LAYER B:

$$\Delta H = 5.5'(0.02)(12^{IN}/_{FT}) \log\left(\frac{1,239+1,549}{1,239}\right) = 0.46 \text{ IN.}$$

$$\Delta H = 5.5'(0.2)(12^{IN}/_{FT}) \log\left(\frac{1,239+1,811}{1,239}\right) = 5.16 \text{ IN.}$$

~ 27 IN.

- ASSUME EACH SUBLAYER HAS OCR = 1.50 .... LAYER A = 1,008 PSF ; LAYER B = 1,859 PSF

LAYER A:

$$\Delta H = 12'(0.02)(12^{IN}/_{FT}) \log\left(\frac{672+1,008}{672}\right) = 1.15 \text{ IN.}$$

$$\Delta H = 12'(0.2)(12^{IN}/_{FT}) \log\left(\frac{672+2,492}{672}\right) = 19.38 \text{ IN.}$$

~ 26 IN.

LAYER B:

$$\Delta H = 5.5'(0.02)(12^{IN}/_{FT}) \log\left(\frac{1,239+1,859}{1,239}\right) = 0.53 \text{ IN.}$$

$$\Delta H = 5.5'(0.2)(12^{IN}/_{FT}) \log\left(\frac{1,239+1,501}{1,239}\right) = 4.55 \text{ IN.}$$

HALEY & ALDRICH

Calculations

File No.	38751-030		
Sheet	4 of 4		
Client	MAINE DOT	Date	4/30/2013
Project	ROUTE 1 VIADUCT BRIDGE REPLACEMENT	Computed By	BCS
Subject	SPAN 11 & 12 INFILL (PIER 10-12) SETTLEMENT & STABILITY	Checked By	WAC

- ASSUME EACH SUBLAYER HAS OCR = 2.0 .... LAYER A = 1,344 PSF ; LAYER B = 2,478 PSF

LAYER A:

$$\Delta H = 12'(0.02)(12^{IN}/FT)\log\left(\frac{672+1,344}{672}\right) = 1.37 \text{ IN.}$$

$$\Delta H = 12'(0.2)(12^{IN}/FT)\log\left(\frac{672+2156}{672}\right) = 17.97 \text{ IN.}$$

LAYER B:

$$\Delta H = 5.5'(0.02)(12^{IN}/FT)\log\left(\frac{1,239+2,478}{1,239}\right) = 0.63 \text{ IN.}$$

$$\Delta H = 5.5'(0.2)(12^{IN}/FT)\log\left(\frac{1,239+882}{1,239}\right) = 3.08 \text{ IN}$$

~ 23 IN.

- ASSUME EACH SUBLAYER HAS OCR = 4 .... LAYER A = 2,688 PSF ; LAYER B = 4,956 PSF

LAYER A:

$$\Delta H = 12'(0.02)(12^{IN}/FT)\log\left(\frac{672+2,688}{672}\right) = 2.01 \text{ IN.}$$

$$\Delta H = 12'(0.2)(12^{IN}/FT)\log\left(\frac{672+812}{672}\right) = 9.91 \text{ IN.}$$

LAYER B:

$$\Delta H = 5.5'(0.02)(12^{IN}/FT)\log\left(\frac{1,239+3,360}{1,239}\right) = 6.75 \text{ IN.}$$

~ 13 IN.

- SUMMARIZE SETTLEMENT ESTIMATES....

OCR	SETTLEMENT (IN.)
1.0	28
1.25	27
1.50	26
2.0	23
4.0	13