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GEOTECHNICAL DESIGN REPORT
WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO. 1564
MAINEDOT WIN 018915.20
BREWER, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Maine Department of Transportation
Augusta, Maine

File No. 132076-005
May 2020

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Maine Department of Transportation
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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Final Geotechnical Design Report
Wilson Street (Route 1A) over Interstate 395, Bridge No. 1564
Interstate 395/Route 9 Connector Roadway Project
MaineDOT WIN 018915.20
Brewer, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Geotechnical Design Report, Wilson Street (Route 1A) over Interstate 395, Bridge No. 1564, MaineDOT WIN 018915.20, Brewer, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our mutually agreed upon work scope and in accordance with the provisions of our project-specific, stand-alone project contract authorized by your William A. Pulver, P.E. on 11 May 2018.

Introduction

This GDR presents the results of preliminary (Phase I) and final design phase (Phase II) subsurface and laboratory testing programs, technical evaluations, and geotechnical design recommendations conducted by Haley & Aldrich, Inc. (Haley & Aldrich) on behalf of the Maine Department of Transportation (MaineDOT) for the proposed replacement of Wilson Street (Route 1A) Bridge No. 1564 over Interstate 395 (I-395) in Brewer, Maine (see Figure 1, Project Locus). Please note that the subject project is part of the greater I-395/Route 9 Connector Roadway Project.

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM AND BASELINE STATIONING

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

PROJECT LOCATION AND EXISTING BRIDGE STRUCTURE

The existing Wilson Street Bridge (bridge) carries Route 1A traffic over I-395 in Brewer, Maine. Based on our review of historic bridge drawings (Appendix D) provided by MaineDOT, it is our understanding that the existing bridge was originally constructed in the early 1980s and consists of a 125-ft long, 60 to 65 ft wide, single-span structure supported on two full-height, cast-in-place (CIP) concrete abutments. The existing abutments are supported on spread footings bearing on fill soils. The geotechnical report for the existing bridge, dated September 1982 (see Reference 1) recommended that footings be designed based on an allowable bearing pressure ranging from approximately 3 to 6 tons per square foot (tsf) for footing widths ranging between approximately 7 and 15 ft. The historic bridge drawings indicate that the "maximum calculated footing pressures" are 5 tsf for abutments and 4 tsf for wingwalls.

Approach embankments required placement of up to approximately 35 ft of fill to meet proposed finish grades. As shown on Sheet Nos. 103 and 104 in Appendix D and described in the geotechnical report for the existing approach roadways, dated November 1982 (see Reference 2), the existing bridge embankments were constructed using resisting toe berms to maintain acceptable global factors of safety throughout embankment construction. These berms were constructed north of the existing east embankment and south of the ramp (I-395 to Wilson Street eastbound) located east of the existing bridge. In a limited area immediately east of the east abutment and north of the embankment, two levels of berms were constructed. We anticipate that the embankment in this area was constructed in stages to achieve the necessary strength gain in the underlying marine clay in order to maintain minimum global stability factors of safety. It is not known if sand or prefabricated vertical drains were installed prior to embankment construction in this area to accelerate pore pressure dissipation, consolidation, and strength gain in the underlying compressible marine clay.

PROPOSED IMPROVEMENTS

The replacement bridge and approach roadway alignment will be offset to the south of the existing bridge alignment as shown on Figure 2, Site and Subsurface Exploration Location Plan (1 of 2). Our current understanding of the proposed bridge structure and approach roadway/embankments is based on the 60 percent draft plan set provided by MaineDOT on 9 March 2020. The total length of the proposed project alignment is approximately 2,250 ft, which consists of a 312-ft long, two-span bridge structure, and approximately 760 ft of approach roadway/embankment. Proposed finished roadway grades along the bridge centerline will range from approximately El. 144 at Abutment No. 1 (west abutment) to approximately El. 135 at Abutment No. 2 (east abutment). To achieve this proposed roadway vertical profile, raise-in-grades of up to approximately 12 to 16 ft will be required. In addition, up to approximately 14 ft of existing embankment fill soils will need to be excavated between the proposed Pier and Abutment No. 2 to achieve the proposed vertical profile for the new connector roadway.

Existing embankment fill soils will be excavated to construct the proposed bridge and connector roadway. It is our understanding that the excavated soil is proposed to be temporarily stockpiled on the embankment north of the approach roadway east of the existing bridge. The stockpile is proposed to be placed over an approximate 120 by 300 ft area (herein referred to as "stockpile area"). The maximum fill height is proposed to be approximately 26 ft (El. 131). The stockpile area is planned to be located in the vicinity of test boring BB-BWS-301 (see Figure 3).

Site Geology

Based on Maine Geological Survey's Surficial Geology of the Veazie Quadrangle, Maine (2011) and deposits observed in recent explorations, surficial deposits mapped at the site consist of artificial fill, alluvial deposits, glacial-marine deposits, and glacial till.

Artificial fill was encountered in recent explorations within the limits of the existing Wilson Street bridge, ramps, and approach embankments. Fill typically consists of clayey silt to sandy silt with varying amounts of gravel. Granular fill was also encountered consisting of minor thicknesses of gravel with varying amounts of sand, and sand with varying amounts of silt and gravel.

Alluvial deposits are mapped within floodplains of low-lying streams within the Veazie Quadrangle, Maine (2011). At the eastern end of the site a stream passes beneath U.S. Route 1 through a large diameter culvert. Alluvial deposits are mapped along the stream corridor throughout this portion of the site. Although these deposits were not encountered in the recent test borings, mapped deposits consist of sand, gravel, silt, and organic sediment.

Glacial-marine deposits of the Presumpscot Formation are mapped in topographically low-lying areas consisting primarily of silt and clay deposits with minor sand and gravel. This deposit was encountered in the central and eastern part of the site but was absent in the westernmost part of the site where ground surface elevations are higher and glacial till deposits are mapped near the surface.

Glacial till deposits were encountered beneath the man-placed fill in recent explorations in the western portion of the site and beneath glacial marine deposits in the central and eastern areas of the site. This glacial till unit primarily consisted of hard silt with varying amounts of sand and gravel with minor deposits of well graded sand with varying amounts of silt and gravel.

According to Bedrock Geology of the Veazie Quadrangle (2011), bedrock within the site is primarily mapped as siltstone and claystone slate of the Brewer Formation. Mapped subordinate rock types consist of fine-grained calcareous quartz-rich meta-arenite and noncalcareous feldspathic metawacke. Thin beds of dark gray to gray-black metalimestone may also be present. The Brewer Formation is Silurian to Ordovician in age. Rock core samples collected in recent explorations at the site consisted of siltstone and schist.

Subsurface Explorations

HISTORIC EXPLORATIONS

Multiple phases of explorations were conducted at the site by the Maine State Highway Commission in association with the original design and construction of the bridge. A total of nine "wash borings" were drilled in the vicinity of each existing bridge abutment as summarized below.

- October 1978 – one "wash boring", designated GP-56-78
- April 1980 – one "wash boring", designated GP-14-1980
- May and August 1982 – five "wash borings", designated GP-26-82, GP-27-82, GP-28-82, GP-45-82 and CB-12-82
- May 1984 – two "wash borings", designated GP-43-84 and GP-44-84

Test borings were drilled to depths ranging from approximately 27 to 50 ft below ground surface (BGS) using 4-inch (in.) and/or 2.5-in. steel casing. Soil samples were generally collected at standard, 5-ft intervals by driving a split-spoon sampler with a 140 lb. hammer dropped from a height of 30 in. Test boring CB-12-82 was advanced approximately 5 ft into bedrock.

The locations of the explorations as well as logs providing information on subsurface conditions encountered in the "wash borings" are provided on historic drawings included in Appendix D.

PRELIMINARY DESIGN PHASE EXPLORATIONS (JULY 2018)

Haley & Aldrich completed a preliminary design phase (Phase I) subsurface exploration program in association with the subject project, which consisted of five test borings, designated BB-BWS-101 through BB-BWS-105, that were drilled at the site in July 2018. The primary purpose of the Phase I subsurface exploration program was to characterize the subsurface conditions along the proposed bridge/roadway alignment and in the vicinity of proposed bridge substructures.

The test boring locations were laid out in the field by Haley & Aldrich using global positioning system (GPS) survey equipment prior the start of drilling. "As-drilled" test boring locations and ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling. The "as-drilled" coordinates and station and offset distance/direction relative to the proposed baseline was provided by MaineDOT. The plan location data of the explorations are summarized in Table I and the locations are shown on Figure 2.

The test borings were drilled by Northern Test Boring, Inc. (NTB) of Gorham, Maine using a Diedrich D50 track-mounted drill rig. Test borings were advanced to depths ranging from approximately 37 to 51 ft BGS using cased-washed drilling methods and a combination of solid-stem augers and 4 in. (HW-size) outside diameter (OD) steel casing. Soil samples were generally collected continuously and/or at standard, 5-ft intervals, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval.

The drill rig was equipped with a calibrated automatic hammer. Based on the calibration information provided by NTB, a theoretical hammer efficiency factor of 0.907 was used for this automatic hammer. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.907) divided by 0.6 (i.e., 60 percent calculated hammer efficiency). Both the raw blow count data (uncorrected N-values) and the corrected N-values are shown on the boring logs in Appendix A.

Test borings drilled at proposed substructure locations (i.e., BB-BWS-102(OW) through BB-BWS-104(OW)), were advanced approximately 11 to 12 ft into bedrock using a 2.0-in. (NQ-size) ID, diamond-tipped core barrel.

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

An in-situ vane shear test was conducted within the marine clay deposit in test boring BB-BWS-105. The vane shear test was conducted with a 65-mm by 130-mm rectangular Geonor vane attached to a 2-ft long, 12-mm diameter rod extension, attached to a string of 5/8-in. OD hollow chrome-moly rods. At the 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 are provided on the test boring logs in Appendix A.

Observation wells were installed in completed boreholes BB-BWS-102(OW) and BB-BWS-104(OW) to provide information on the static groundwater levels at the site. The observation wells consisted of 2-in. ID, machine-slotted PVC pipe and solid PVC riser pipe extending approximately 3 ft above existing ground surface. Each observation well was outfitted with a steel guard pipe and steel lock/cap assembly. Observation well installation and groundwater monitoring reports are provided in Appendix B.

All drilling, sampling, and in-situ testing were performed in accordance with MaineDOT specifications.

FINAL DESIGN PHASE EXPLORATIONS (DECEMBER 2019 TO JANUARY 2020)

Haley & Aldrich completed a final design phase (Phase II) subsurface exploration program in association with the subject project, which consisted of seven test borings, designated BB-BWS-201 through BB-BWS-206 and BB-BWS-301, that were drilled at the site in December 2019 (test borings BB-BWS-201 through BB-BWS-206) and January 2020 (test boring BB-BWS-301). The primary purpose of the Phase II borings was to further characterize the subsurface conditions along the proposed bridge/roadway alignment and in the vicinity of proposed bridge substructures so that design-level technical evaluations could be completed.

Test boring BB-BWS-301 was drilled to characterize subsurface conditions in the proposed stockpile area. The technical evaluations related to the stockpile area will be submitted in a separate memorandum.

The test boring locations were laid out in the field by Haley & Aldrich using global positioning system (GPS) survey equipment prior the start of drilling. "As-drilled" test boring locations and ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling. The "as-drilled" coordinates and station and offset distance/ direction relative to the proposed baseline was provided by MaineDOT. The plan location data of the explorations are summarized in Table I and the locations are shown on Figures 2 and 3.

Test borings BB-BWS-201 through BB-BWS-206 were drilled by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile B53 track-mounted drill rig. Test boring BB-BWS-301 was drilled by S.W. Cole Engineering, Inc. (SWC) of Bangor, Maine using a Mobile B53 track-mounted drill rig. The Phase II test borings were advanced to depths ranging from approximately 37 to 79 ft BGS using cased-washed drilling methods and a combination of solid-stem augers and 4 in. (HW-size) outside diameter (OD) steel casing. Soil samples were generally collected continuously or at standard, 5-ft intervals, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval.

Both drill rigs were equipped with a calibrated automatic hammer. Based on the calibration information provided by NEBC and SWC, a theoretical hammer efficiency factor of 0.842 was used for the NEBC automatic hammer and a theoretical hammer efficiency factor of 0.977 was used for the SWC automatic hammer. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.842 and 0.977) divided by 0.6 (i.e., 60 percent calculated hammer efficiency). Both the raw blow count data (uncorrected N-values) and the corrected N-values are shown on the boring logs.

Test borings drilled at proposed substructure locations (BB-BWS-202, BB-BWS-203, and BB-BWS-206) were advanced approximately 10 to 16 ft into bedrock using a 2.0-in. (NQ-size) ID, diamond-tipped core barrel.

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

All drilling, sampling, and in-situ testing were performed in accordance with MaineDOT specifications.

Generalized Subsurface Conditions

The subsurface conditions encountered at the site during the Phase I and Phase II subsurface exploration programs completed by Haley & Aldrich generally consist of the following geologic units presented in order of increasing depth below ground surface: man-placed fill soils (embankment and in-situ), marine deposits, glacial till, and bedrock. Refer to Table II for a summary of the soil units and encountered thicknesses at each test boring location. The top of the in-situ fill elevation in the final design phase test borings was estimated based on the subsurface profile included in the historical 1984 geotechnical report (Sheet 27). A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the test boring logs included Appendix A. Refer to Figure 4 for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Please note that the information presented below is limited to the conditions encountered in the Phase I and Phase II test borings completed under the direction of Haley & Aldrich. Also note that soil descriptions provided on the test boring logs do not represent actual field conditions other than at the specific test boring locations. The actual conditions encountered between test boring locations will likely vary from those described below.

Subsurface Material		**Approximate Encountered Thickness (ft)**	**Approximate Average Thickness (ft)**	**Generalized Description**
Embankment Fill	Silt, Sand, Gravel	0 to 22.9	21.9	Brown to grey to brown-grey to grey, dry to wet, medium dense to very dense, gravelly fine to coarse SAND (SP, SW, SW-SM, SP-SM), fine to medium Silty SAND (SM), Sandy fine to coarse GRAVEL (GW-GM), hard, SILT (ML), trace to some silt, trace to some fine to coarse sand, trace to some fine to coarse gravel.
In-Situ Fill	Clay, Silt, Sand, Gravel	3.4 to 27.6	13.5	Brown to grey, moist to wet, medium dense to very dense, fine to coarse SAND (SP, SW, SP-SM), fine to medium Silty SAND (SM), fine to coarse GRAVEL (GM, GP), Sandy GRAVEL (GW, GW-GM), stiff to hard, Silty CLAY (CL), Clayey SILT (ML), Sandy SILT (ML), SILT (ML),trace to some clay, trace silt, trace to some fine to coarse sand, trace to some fine to coarse gravel.
Marine Deposit	Clay and Silt	0 to 27.5	16.1	Grey to brown, moist to wet, very stiff to hard, SILT (ML), Silty CLAY (CL), Clayey SILT (ML), trace to little fine to coarse sand, trace fine gravel.
Glacial Till	Silt and Sand	5.3 to >32.8	14.6	Dark grey to grey to brown, moist to wet, very dense, fine to coarse SAND (SP), hard, SILT (ML), Silty CLAY (CL), Clayey SILT (ML), Sandy SILT (ML), well-bonded, trace clay, trace to little silt, trace to little fine to coarse sand, trace to some fine gravel, cobbles, weathered bedrock fragments above top of bedrock.

Bedrock	Top of bedrock near proposed substructures encountered from approximately El. 71.3 to El. 55.6. The top of bedrock is relatively flat and generally slopes down from Abutment No. 1 to Abutment No. 2 (west to east). *(encountered in test borings BB-BWS-102(OW), -103, -104(OW), -202, -203, -206, and -301)*

BEDROCK CONDITIONS

As stated previously, approximately 10 to 16 ft of bedrock was sampled in test borings BB-BWS-102(OW) through BB-BWS-104(OW), BB-BWS-202, BB-BWS-203, and BB-BWS-206, which were drilled in the vicinity of proposed substructure locations. The sampled and recovered bedrock generally consisted of the following:

- Moderately hard to hard, moderately weathered to fresh, aphanitic, SILTSTONE, SCHIST of the Brewer Formation. Primary joints dip at moderate to steep angles and are very close to closely spaced, tight to open, planar, smooth with slight oxidation on a few joint surfaces. Secondary joints dip at low to moderate angles and are spaced moderately close, tight to open, planar to undulating, smooth.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the core run. A summary of pertinent bedrock information encountered at the proposed substructures is presented below.

Substructure	Test Boring No.	Approximate Top of Bedrock Elevation (ft, NAVD 88)	Rock Quality Designation (%)		Rock Quality[1]
			Range	Approximate Average	
Abutment No. 1	BB-BWS-102(OW)	72.4	0 to 10	3	Very Poor
	BB-BWS-202	71.3	82 to 88	85	Good
Pier	BB-BWS-103	72.9	0	0	Very Poor
	BB-BWS-203	66.7	72 to 92	84	Good
Abutment No. 2	BB-BWS-104(OW)	64.1	0 to 76	49	Poor
	BB-BWS-206	55.6	97 to 100	99	Excellent

Note:
[1] Rock quality description based on average RQD value.

The top of bedrock elevation at each test boring location is shown on Figures 2 and 3.

Please note that it is our opinion that the RQD values summarized above and shown on the boring logs included in Appendix A were influenced by the high angle jointing and foliation of the rock, which repeatedly caused the core barrel to block during sampling. Therefore, RQD measurements may not be indicative of the actual rock mass quality.

Photographs of the sampled bedrock are provided for reference in Appendix A.

GROUNDWATER CONDITIONS

As discussed previously, two observation wells were installed in completed boreholes BB-BWS-102(OW) and BB-BWS-104(OW). The observation wells were installed to provide information on the static groundwater levels at the site. The measured water levels during the period 13 July 2018 to 4 April 2020 ranged from approximately 1.5 to 6.0 ft BGS (El. 110.7 to El. 106.2) at BB-BWS-102(OW) to +1.3 (above ground surface) to 1.4 ft BGS (El. 102.2 to El. 99.6) at BB-BWS-104(OW).

In general, water levels may fluctuate with season, precipitation, local soil/bedrock conditions, and excavation means and methods. Therefore, water levels may vary from those summarized above, provided on the testing boring logs included in Appendix A and shown on the groundwater monitoring reports included in Appendix B.

Laboratory Test Results

Preliminary and final design geotechnical laboratory testing programs were undertaken by Haley & Aldrich on representative soil and rock samples to aid in soil classification and determination of engineering soil and rock properties. All laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures by GeoTesting Express, Inc. (GTX) of Acton, Massachusetts. A summary of the preliminary laboratory testing results is shown below.

Laboratory Test	ASTM Test Designation	Soil Unit	No. of Test(s)	Range in Test Results[1]
Moisture Content	ASTM D2216	Marine Deposit	9	WC = 12.2% to 33.0%
Grain Size	ASTM D422	Existing Fill	13	AASHTO Classification: A-1-a, A-1-b, A-4, A-2-4 USCS Classification: GW-GM, SW-SM, SW, SM, ML
		Marine Deposit	7	AASHTO Classification: A-4 USCS Classification: CL-ML
Atterberg Limits	ASTM D4318	Marine Deposit	11	19 < LL < 36 14 < PL < 21
One-Dimensional Consolidation[2, 3]	ASTM D2435 (Method B)	Marine Deposit	1	Preconsolidation Pressure ($\sigma_{p'}$): 2,800 psf Overconsolidation Ratio (OCR): 1.39

				Recompression Ratio (RR): 0.06 Virgin Compress. Ratio (CR): 0.16 Virgin Secondary Compression Ratio ($C_{\alpha\varepsilon}$): 0.0003 Recompression Coefficient of Consolidation (c_{vr}): 0.06 ft^2/day Virgin Coefficient of Consolidation (c_v): 0.08 ft^2/day
Consolidated Undrained Triaxial[3]	ASTM D4767	Marine Deposit	1	Undrained Shear Strength (S_u): 843 psf Strain at Failure: 9.2%
Compressive Strength and Elastic Moduli of Rock	ASTM D7012	Bedrock	4	Peak Compressive Stress (q_p): 2,490 psi to 14,729 psi Young's Modulus (E): 986,000 to 10,300,000 psi Poisson's Ratio (v): 0.04 to 0.41

Notes:
[1] LL = Liquid Limit; PL = Plastic Limit; WC = Moisture Content; psi = pounds per square in.
[2] Consolidation test results are interpretations from laboratory test data.
[3] Test on marine deposit sample taken from boring BB-BWS-301 in temporary stockpile area.

All laboratory test results are shown on the test boring logs included in Appendix A with complete results provided in Appendix C.

Geotechnical Evaluations and Design Recommendations

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, Eighth Edition, 2017, referred to herein as AASHTO LRFD.
- MaineDOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as MaineDOT BDG.

Engineering calculations that support the geotechnical recommendations outlined herein are provided for reference in Appendix E.

APPROACH EMBANKMENTS

Subsurface soil conditions along the proposed bridge alignment will affect the planning and design of the construction of the new, widened embankment. As previously stated, maximum raises in grade outside of the existing embankment (beyond the toe of the embankment) are anticipated to be up to approximately 27 ft (at the southern end of Abutment No. 1). Global embankment stability analyses were conducted in order to assess the feasibility of constructing the bridge embankments using normal weight earthfill over the silty to clayey soils present at the site.

Global Embankment Stability

Embankment construction using normal weight earthfill can cause excessive vertical and lateral strains, potentially resulting in a shear failure of the foundation soil and subsequent failure of the embankments. A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 6.0, developed by Rocscience, Inc., to evaluate the likelihood of global stability failures at the site.

Static and seismic stability evaluations were conducted transverse and south of the proposed bridge centerline at proposed Abutment Nos. 1 and 2, where, based on our review of proposed grading plans, maximum raises in grade are proposed. We did not conduct evaluations transverse and north of the centerline since no raises in grade are planned in these areas. We conducted longitudinal evaluations through Abutment No. 1. We did not conduct longitudinal evaluations at Abutment No. 2 since the grading behind Abutment No. 2 slopes away from the abutment and the proposed fill heights are not as high as at Abutment No. 1.

A typical soil profile was developed based on the subsurface conditions encountered in the Phase I and Phase II test borings at each abutment. In addition, for transverse and longitudinal orientation evaluations, a 250 psf live load surcharge was assumed to act over the embankment width (transverse) and along the roadway length (longitudinal) to account for traffic loading. Multiple global stability evaluations were conducted to assess the sensitivity of calculated safety factors to changes in varying:

- Proposed embankment fill friction angles.
- Circular failure surfaces at different depths/strata.
- Undrained shear strengths of glacial till (we considered 2,000 to 4,000 psf) and marine deposits (we considered 500 to 1,500 psf). Values selected for the final evaluations are shown below.

The following physical and strength properties for soil materials were used to complete global stability evaluations.

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Proposed Embankment Fill	125	32	0
In-Situ Fill	120	32 (Abutment No. 1) 30 (Abutment No. 2)	0
Granular Borrow (longitudinal only)	120	32	0
Riprap (longitudinal only)	140	45	0
Marine Deposit	110	0	1,500
Marine Deposit and Glacial Till (Silt)	115	0	2,000
Glacial Till	115	0	2,000

The calculated global stability factors of safety values for varying conditions at Abutment Nos. 1 and 2 are summarized below and calculations are included in Appendix E.

Type of Stability Analyses	Proposed Embankment Fill Friction Angle (degrees)	Lowest Factor of Safety (per Spencer Method)		
		Circular Failure Surface Through Proposed Fill	Circular Failure Surface Through Existing Fill[1]	Circular Failure Surface Through Marine Deposits and/or Glacial Till
West Approach Embankment behind Abutment No. 1 – transverse and longitudinal and south of centerline				
Static (transverse)	32	1.25	1.38	1.96
Seismic (transverse)	32	1.13	1.26	1.77
Static (longitudinal)	32	1.44		2.07
Seismic (longitudinal)	32	1.33		1.80
East Approach Embankment behind Abutment No. 2 – transverse and south of centerline				
Static	32	1.27	1.39	1.60
Seismic	32	1.15	1.26	1.45

Note:
[1] Failure searches included the existing embankment fill and proposed fill. Lowest factors of safety were determined to be located at the approximate interface between the existing and proposed fill layers (refer to Appendix E).

The factor of safety for pseudo-static seismic load cases were calculated using a horizontal acceleration coefficient, k_h, of 0.04g (i.e., one half of the acceleration coefficient, A_s). An acceleration coefficient value of $A_s/2$ was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2; the reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a very short period of time.

The minimum required factor of safety as specified by both AASHTO LRFD and the MaineDOT BDG for embankments under static conditions which do not support structures is 1.3. The minimum required factor of safety for embankments subjected to pseudo-static seismic loading is 1.0 (MaineDOT BDG).

The results of transverse global embankment stability analyses summarized above indicate that the critical failure surface is a shallow failure surface through the slope of the proposed embankment fill result in static factors of safety just below 1.3. We modeled the proposed embankment slope at an inclination of 2H:1V. These surfaces are representative of surficial, sloughing failures, and not a deep-seated global instability. Furthermore, the results indicate that the proposed embankment has a minimum static factor of safety of 1.38 for failure surfaces penetrating through the proposed fill into the underlying in-situ fill, which is greater than the minimum acceptable factor of safety of 1.3 (MaineDOT BDG and AASHTO LRFD). In addition, the minimum calculated factor of safety for the proposed embankment subjected to seismic loading is 1.13, greater than the minimum acceptable factor of safety of 1.0 (MaineDOT BDG).

The results of longitudinal global embankment stability analyses summarized above indicate that the critical failure surface is a shallow failure surface through the riprap slope in front of Abutment No. 1, which results in static factor of safety of 1.44, which is greater than the minimum acceptable factor of safety of 1.3 (MaineDOT BDG and AASHTO LRFD). Furthermore, the results indicate that the proposed embankment has a minimum static factor of safety of 2.07 for failure surfaces penetrating through the proposed fill into the underlying marine deposits. In addition, the minimum calculated factor of safety for the proposed embankment subjected to seismic loading is 1.33, greater than the minimum acceptable factor of safety of 1.0 (MaineDOT BDG).

We therefore conclude that proposed embankment construction using normal-weight earthfill will have acceptable global stability factors of safety provided that the fill is compacted to provide an in-place friction angle of at least 32 degrees.

As noted earlier in this report, the existing bridge embankments were constructed using resisting toe berms constructed adjacent to the existing east embankment to maintain acceptable global stability factors of safety throughout construction. These berms were constructed north of the existing east embankment and south of the ramp (I-395 to Wilson Street eastbound) east of the existing bridge where soft, compressible deposits of marine clay are present. Fortunately, these soft, low-strength compressible soils are not present south of the existing embankments where the proposed new embankments will be constructed. Therefore, resisting toe berms will not be needed to maintain acceptable factors of safety during construction of the new embankments.

Embankment Settlement

As described above, the proposed embankments will be widened to the south to construct the new approach roadways. Based on the test borings, compressible marine deposits are not present beneath the proposed fill slopes to the south of the existing approach embankment at Abutment No. 1. However, stiff, compressible marine deposits are present beneath the proposed fill slopes to the south of the approach embankment at Abutment No. 2. Accordingly, we have evaluated settlements due to the proposed fill that will be placed to construct the proposed approach embankments south of Abutment No. 2 only. We modeled the stress history of the marine clay deposit with two layers (referred to as upper and lower) of equivalent thickness: upper layer OCR of 10.2 (based on shear strength measurements from in-situ vane shear tests and SHANSEP correlations; see Appendix E) and a lower layer OCR of 2.3 (based on an average preconsolidation pressure value of 3.2 ksf from Reference 3 and an assumed existing vertical effective stress of 1.4 ksf).

The computer analyses software, Settle3D Version 4, developed by Rocscience, Inc., was used to evaluate settlements due to the proposed embankment widening at Abutment No. 2. The following marine deposit compressibility parameters and assumptions were used in the settlement analyses:

- Undrained Shear Strength (S_u) = 1,980 psf (from in-situ vane shear test data)
- OCR = 10.2 (upper layer) and 2.3 (lower layer)
- Virgin Compression Ratio (CR) = 0.116 (average value from Reference 3)
- Recompression Ratio (RR) = 0.010 (average value from Reference 3)
- Secondary Compression Ratio ($C_{\alpha\varepsilon}$) = 0.0003 (based the secondary compression ratio on consolidation test data from test boring BB-BWS-301 since historical information for this parameter was not provided in Reference 3).
- Coefficient of Consolidation (c_v) = 0.2739 ft^2/day (Reference 3)
- Subsurface conditions were based on test borings BB-BWS-104 and BB-BWS-105 (which were drilled near the fill slope at Abutment No. 2 and encountered the thickest marine deposits in the area).
- Settlement was evaluated for a design life of 75 years after the roadway embankment widening.
- The following stages and durations were assumed in Settle3D:
  - Early 1980s: Existing embankment constructed (35 ft high with embankment fill unit weight of 125 pcf)
  - Year 2020: Estimate total settlement beneath existing embankment from early 1980s to 2020 (40 years).
  - Year 2021 (reference stage): Widen roadway embankment (dimensions of widening: 37 ft wide, 22 ft high, with embankment fill unit weight of 125 pcf).
  - Year 2096: Estimate total settlement for 75 years following the widening of the embankment.

Based on the above assumptions and the anticipated stress history of the marine deposits beneath the proposed roadway widening, we estimate that approximately 1 to 2 in. of total post-construction settlement will occur between 2021 and 2096, following the widening of the proposed approach embankments south of Abutment No. 2. We anticipate that most of the total settlement amount is due to virgin consolidation of the marine deposits. We estimate that approximately 90 percent of the total settlement will occur approximately one year following the roadway embankment widening.

Available historic information (Reference 3) indicates that the existing Abutment No. 2 was undergoing post-construction settlement in the spring of 1984. At the time, MaineDOT estimated that approximately 3 in. of total settlement had occurred since the end of construction of Abutment No. 2. Based on the existing as-built Wilson Street bridge drawings (Sheet 29, 1984), the maximum calculated footing pressures for the abutments and wingwalls were 5 tsf and 4 tsf, respectively. Based on this historical settlement data and maximum calculated footing pressures of the existing Wilson Street substructures, we believe that our estimated total settlement of 1 in. to 2 in. matches fairly well with the historical settlement that was measured in the spring of 1984.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 using Method C for a mixed soil profile containing both cohesionless and cohesive soils. In instances where SPT N-values were equal to 0 or were in excess of 100 blows per foot (bpf), default values of 1 and 100 bpf were used, respectively.

Based on the nature and thickness of the overburden soils and depth to bedrock encountered in the Phase I and Phase II test borings, we recommend that the site be considered as "Site Class C." Spectral accelerations were determined based on the geographic site location and the recommended "Site Class C" designation using the United States Geological Survey (USGS) software application Seismic Design Parameters v. 2.0 provided the recommended AASHTO response spectra for a 7 percent probability of exceedance in 75 years (approx. 1,000-year return period). The recommended values are summarized below.

Design Parameter	Design Value
Site factor for short-period range of acceleration response spectrum, F_a =	1.200
Site factor for long-period range of acceleration response spectrum, F_v =	1.700
Site factor at zero-period on acceleration response spectrum, F_{pga} =	1.200
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, S_S (g) =	0.143
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, S_1 (g) =	0.043
Peak seismic ground acceleration coeff. on rock, PGA (g) =	0.066
Horizontal response spectral acceleration coeff. at 0.2-s period modified by F_a, S_{DS} (g) =	0.172
Horizontal response spectral acceleration coeff. at 1.0-s period modified by F_v, S_{D1} (g) =	0.074
Peak seismic ground acceleration coefficient modified by F_{pga}, A_s (g) =	0.079

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

BRIDGE FOUNDATION SUPPORT

As shown on the interpretive subsurface profile (Figure 4), in-situ fill and marine deposits are present below the desired substructure bearing levels at the pier and abutment locations. Based on the loading demands provided by MaineDOT, it is our opinion that glacial till and bedrock are suitable to support of the bridge substructures. Due to the depth to the top of the glacial till and bedrock at the of the proposed substructure locations (i.e., between 50 and 60 ft below existing site grades), we recommend that the bridge substructures be supported on deep foundations consisting of either driven or drilled in piles advanced into till or rock. Based on recent discussions with MaineDOT, it is our understanding that they plan on supporting the bridge substructures on steel HP14x117 piles driven to/into bedrock.

We based our analyses on the information presented in the 60 percent draft plan set provided by MaineDOT on 9 March 2020. Further, MaineDOT provided the following types of lateral pile evaluation cases, pile bending axis orientation, Strength and Service pile axial loads lateral loads, applied bending moments at top of pile, and prescribed top of pile lateral displacements, via email to aid our analyses and develop the geotechnical design recommendations presented herein.

<table>
<tr><th>Substructure</th><th>Limit State</th><th>Lateral Pile Evaluation Case</th><th>Pile Bending Axis Orientation</th><th>Pile Axial Load (kips)</th><th>Pile Lateral Load (kips)</th><th>Applied Bending Moment at Top of Pile (kip*ft)</th><th>Top of Pile Lateral Displacement (in.)</th></tr>
<tr><td rowspan="2">Abutment No. 1</td><td>Strength</td><td>Expansion</td><td rowspan="2">Strong</td><td rowspan="2">396</td><td rowspan="2">N/A</td><td>-286</td><td>0.89</td></tr>
<tr><td>Strength</td><td>Contraction</td><td>-282</td><td>1.34</td></tr>
<tr><td rowspan="4">Pier</td><td>Service</td><td>1</td><td rowspan="2">Strong</td><td>326</td><td rowspan="2">13.8</td><td>61.3</td><td rowspan="4">N/A</td></tr>
<tr><td>Strength</td><td>2</td><td>411</td><td>61.5</td></tr>
<tr><td>Service</td><td>3</td><td rowspan="2">Weak</td><td>326</td><td rowspan="2">10.4</td><td>42.0</td></tr>
<tr><td>Strength</td><td>4</td><td>411</td><td>42.3</td></tr>
<tr><td rowspan="2">Abutment No. 2</td><td>Strength</td><td>Expansion</td><td rowspan="2">Strong</td><td rowspan="2">396</td><td rowspan="2">N/A</td><td>-286</td><td>0.89</td></tr>
<tr><td>Strength</td><td>Contraction</td><td>-282</td><td>1.34</td></tr>
</table>

Corrosion Loss

The geotechnical engineering design of the proposed piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual review of the soil samples and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. Therefore, the net factored pile resistance values recommended in the following sections of this GDR do not include a reduction in pile cross-sectional area due to corrosion.

Axial Uplift Pile Resistance

Per our discussions with MaineDOT, it is our understanding that piles will not be subjected to uplift.

Axial Compression Pile Resistance

As discussed previously, it is our opinion that based on the subsurface soil conditions present at the site and the magnitude of the maximum factored axial compression pile demand values provided by MaineDOT, the abutment and pier piles will likely be driven to/into bedrock and resistance provided through end bearing.

Structural Resistance

In accordance with AASHTO LRFD, a distinction is made between piles driven to “soft” bedrock and piles driven to “hard” bedrock. Based on our experience on similar projects with similar bedrock conditions, we consider the bedrock at the site to be “hard”. The structural resistance factor (Article 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving conditions (“hard” rock bearing stratum) is 0.5. In addition, resistance factors for Service and Extreme Limit State loading are 1.0. Therefore, the nominal and factored structural resistance of the steel HP14x117 sections (with F_y = 50 ksi) at the Service, Strength, and Extreme Event Limit States for the proposed substructures is summarized below.

Nominal Structural Resistance (kips)	**Factored Structural Resistance (kips)**		
	Service Limit State (ϕ=1.0)	**Strength Limit State (ϕ=0.5)**	**Extreme Limit State (ϕ=1.0)**
1,722	1,722	861	1,722

Geotechnical Resistance

The nominal axial compressive geotechnical resistance at the Strength Limit State was calculated using guidance from the proposed Intact Rock Method (IRM) that is presented in a report authored by Thomas Sandford, Ph.D., P.E. of the University of Maine with a resistance factor of 0.45 [in accordance with LRFD Table 10.5.5.2.3-1 for piles end bearing in bedrock (Canadian Geotechnical Society, 1985)]. The IRM is equivalent to the Rowe and Armitage (1987) equation (cited by Turner, 2006) that relates the ultimate end bearing geotechnical resistance of intact bedrock to the compressive strength of the bedrock. To determine the "design" unconfined compressive strength of the bedrock mass, we considered the four unconfined compressive strength laboratory test results retrieved near the proposed substructures and published typical unconfined compressive strength values of SCHIST (Richard Goodman, 1980, Table 3.1 and LRFD Table 4.4.8.1.2B). Three out of the four unconfined compressive strength test results experienced discontinuity failures and the unconfined compressive strengths of the samples ranged from approximately 2.1 to 4.4 ksi. The fourth unconfined compressive strength test sample experienced an intact material failure and had an unconfined compressive strength of approximately 14.7 ksi. In our experience, intact material failures, instead of discontinuity failures, are more representative of the realistic failure mechanisms and overall strength of the bedrock mass. Therefore, we excluded the laboratory test samples that experienced discontinuity failures and selected a "design" unconfined compressive strength of 11.3 ksi, which was based on laboratory test samples that experienced intact material failures and published typical unconfined compressive strength values of SCHIST.

The estimated factored geotechnical resistances for steel HP14x117 piles at the Service, Strength, and Extreme Event Limit States for the substructures are summarized below.

Nominal Geotechnical Resistance (kips)	**Factored Geotechnical Resistance (kips)**		
	Service Limit State (ϕ=1.0)	**Strength Limit State (ϕ=0.45)**	**Extreme Limit State (ϕ=1.0)**
973	973	438	973

Drivability Resistance

The engineering design of the proposed abutment and pier piles also included consideration of drivability resistance in accordance with AASHTO LRFD Section 10.7.8. The drivability evaluations were conducted using the computer program GRLWEAP 2010 developed by GRL Engineers, Inc. and were conducted to determine 1) if the piles could be impact driven through the stiff to hard/very dense glacial till to bedrock and 2) if the required nominal loads (design loads) were achievable within the limits of MaineDOT Section 501.

The drivability evaluations were conducted assuming that a Delmag D36-32 open-ended diesel hammer with a maximum rated energy equal to approximately 90,000 ft-lbs will be used to install the abutment and pier piles. Drivability evaluations were completed at each proposed substructure location at the Strength Limit State based on the subsurface conditions encountered in the test borings. The drivability resistance for the steel HP14x117 piles for the proposed substructures at the Service, Strength, and Extreme Event Limit States is summarized below.

Substructure	Nominal Drivability Resistance (kips)	Factored Drivability Resistance (kips)		
		Service Limit State (ϕ=1.0)	Strength Limit State[1] (ϕ=0.65)	Extreme Limit State (ϕ=1.0)
Abutments	609	609	396	609
Pier	632	632	411	632

Note:
[1] Factored resistance is based on the factored (strength) design loads from MaineDOT and do not represent the maximum achievable driven pile resistance.

Our evaluations show that the calculated driving stresses do not exceed the maximum permissible values provided in AASHTO LRFD Section 10.7.8 (approximately 43.6 ksi (87 percent of F_y) for 50 ksi steel piles). Please keep in mind that the drivability resistances summarized above are based on an assumed pile hammer size and the assumption that the piles penetrate through the soil overburden and end bear in/on bedrock. If greater axial compressive pile resistance is needed or if actual pile hammers used to install the piles have different maximum rated energies than those assumed herein, additional evaluations will be required to determine the nominal drivability resistance that can be achieved at a reasonable penetration resistance without overstressing the piles. GRLWEAP results are included in Appendix E.

Summary and Recommended Resistances of Abutment and Pier Piles

The factored resistances of the abutment and pier piles are controlled by the lesser of the Strength Limit State structural, geotechnical, and drivability resistances and are summarized below.

Substructure	Factored Structural Resistance (kips)	Factored Geotechnical Resistance (kips)	Factored Drivability Resistance[1] (kips)	Governing Factored Resistance (kips)
Abutment No. 1	861	438	396	**396**
Pier	861	438	411	**411**
Abutment No. 2	861	438	396	**396**

Note:
[1] Factored resistance is based on the factored (strength) design loads from MaineDOT and do not represent the maximum achievable driven pile resistance.

Lateral Pile Evaluations

Using the displacements, axial loads, and bending moments provided by MaineDOT, Haley & Aldrich conducted lateral pile evaluations at the abutments and pier using the software program LPile by Ensoft, Inc. The shear forces and moments caused by the imposed displacements were provided to MaineDOT to perform the structural design of the piles. MaineDOT provided maximum lateral deflections of the abutments caused by contraction and expansion of the bridge due to thermal deformations.

We modeled a single, vertical HP14x117 pile, with a fixed head condition at the top of the pile for lateral pile evaluations at the abutments and pier. Lateral group effects (pile-soil interaction) were not significant given the pile spacing to pile size ratio. For contraction lateral pile evaluations, we reduced p-multipliers based on the sloped ground surface adjacent to the piles.

We have intermittently submitted our iterative lateral pile evaluations to MaineDOT via email since 11 February 2020.

Abutments

A summary of the maximum shear force and maximum moments as determined by the lateral pile evaluations for Abutment Nos. 1 and 2 is provided below. A pile axial load of 396 kips (Strength Limit State) was used for all lateral pile evaluations at the abutments. The load case resulting in the highest shear force and bending moment is reported. The table below presents results following the application of plastic bending moments, provided by MaineDOT, during the iterative evaluations. The first iteration (plastic moment not applied) of the LPile results of deflection, bending moment, and shear with depth are included in Appendix E.

Substructure	Case	Top of Pile Lateral Displacement (in.)	Applied Bending Moment at Top of Pile (kip*ft)	Maximum Pile Shear (kips)	Maximum Pile Bending Moment (kip*ft)	Estimated Combined Stress (ksi)
Abutment No. 1	Expansion	0.89	-286	238	286	66
	Contraction	1.34	-282	59	282	65
Abutment No. 2	Expansion	0.89	-286	156	286	66
	Contraction	1.34	-282	84	282	65

Pier

A summary of the maximum shear force and maximum bending moments as determined by the lateral pile evaluations for the Pier is provided below. The load case resulting in the highest shear force and moment is reported. LPile results of deflection, bending moment, and shear with depth are included in Appendix E.

Case	Limit State	Bending Axis	Pile Axial Load (kips)	Lateral Load (kips)	Maximum Pile Bending Moment (kip*ft)	Top of Pile Lateral Displacement (in.)	Estimated Maximum Stress (ksi)
1	Service	Strong	326	13.8	61.3	0.06	13.8
2	Strength		411		61.5		16.2
3	Service	Weak	326	10.4	42.0	0.10	18.0
4	Strength		411		42.3		20.5

Pile Settlement and Elastic Pile Compression

Pile settlement due to elastic shortening of the steel HP14x117 piles was evaluated based on the maximum factored Service Limit State loads for the abutment and pier piles of 284 kips and 326 kips, respectively (provided by MaineDOT). Estimates of elastic pile compression for individual piles at each substructure are summarized below.

Substructure	Approximate Elastic Pile Compression (in.)
Abutment No. 1	0.20
Pier	0.17
Abutment No. 2	0.23

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

Estimated Pile Lengths

Each steel HP14x117 pile will be driven to/into bedrock. Based on the site subsurface conditions and the 60 percent draft plan set, we estimate the following pile lengths, as summarized below.

Substructure	Bottom Elevation of Proposed Substructure (ft)	Interpolated Top of Bedrock Elevation at Proposed Centerline (ft)	Estimated Pile Lengths (ft)
Abutment No. 1	131.0	71	65
Pier	111.0	66	50
Abutment No. 2	121.6	55	72

The estimated pile lengths do not account for embedment in the substructure, penetration into bedrock, locations where bedrock may deeper or shallower than that encountered in the final design phase test borings, damaged pile, the additional pile required for dynamic testing instrumentation (per ASTM D4945), or the additional pile length needed to accommodate the Contractor's leads and driving equipment. For the purposes of developing pile quantities for bid, the estimated pile lengths in the table above incorporate an additional 5 ft.

Pile Embedment, Spacing, Clearance, and Pile Material

We recommend that minimum pile spacing, clearance and embedment (into the pile cap) meet the requirements of AASHTO LRFD Section 10.7.1.2. We also recommend that the piles be equipped with a pile tip in accordance with MaineDOT Standard Specification 501.048. Based on the potential for hard driving and sloping bedrock, we recommend the pile tip consist of a HP-77750-B manufactured by Associated Pipe & Fitting, or equivalent, fabricated from cast steel conforming to ASTM A148 Grade 90/60.

ABUTMENT AND WINGWALL DESIGN

We recommend that the abutments and wingwalls be designed for all relevant Service, Strength, and Extreme Limit States and load combinations specified in AASHTO LRFD Sections 3.4.1 and 11.5.5. Additional recommendations are provided below.

The abutment design shall include a drainage system behind the abutment to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutment and wingwall spread footings. Drainage behind the structure shall be in accordance with BDG Section 5.4.2.13.

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

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

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

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

If determined applicable by MaineDOT, the walls and abutments should be designed for a uniform lateral load to account for seismic soil loading in accordance with LRFD Article A.11.3.1 (Mononobe-Okabe Method). Based on the seismic site class (Site Class "C"), we recommend a seismic active earth pressure coefficient, K_{AE}, of 0.30 be used for design for both abutments. Note that this soil pressure includes both the static and seismic lateral earth loads. We neglected the slightly sloping backfill surface behind Abutment No. 1, since the sloping backfill percentage did not significantly alter the K_{AE}. Therefore, the K_{AE} value of 0.30 is applicable for both abutments.

Frost Protection

The minimum depth of embedment/cover for footings or other below grade structures was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG. Based on a design freezing index equal to 1,650 freezing degree-days, we recommend that the abutments and pier bear a minimum of 6 ft below the lowest adjacent ground surface exposed to freezing. However, based on discussions with the MaineDOT and in reference to MaineDOT BDG Figures 5-2 and 5-3, we understand that MaineDOT plans to embed the abutments 4 ft. Refer to Appendix E, Calculations, for supporting documentation.

Construction Considerations

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

TEMPORARY EARTH SUPPORT

Based on the proposed elevation of the bottom of the foundations and depending on the construction sequence of the project, temporary earth support systems may be needed to construct the substructures (if conventional temporary sloping is not feasible). Based on the subsurface soil and groundwater conditions at the site, we anticipate that the most cost-effective excavation support systems for construction of the substructures will consist of steel sheeting.

Design of temporary works, including temporary earth support system(s) are typically the responsibility of the Contractor and should be conducted by a Licensed Professional Engineer in the State of Maine hired by the Contractor. We recommend that temporary earth support system(s) be designed to support all appropriate combinations of earth, water, and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The Contractor is responsible for choosing an applicable factor of safety for the earth support system(s). The Contractor's design should also consider the means and methods and construction sequencing proposed. We recommend that design calculations and shop drawings be prepared by the Contractor, stamped by a Licensed Professional Engineer in the State of Maine, and submitted to MaineDOT for review prior to construction.

DYNAMIC PILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the minimum required nominal pile resistances in the field using dynamic testing methods. The piles should be driven to a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 in accordance with AASHTO LRFD Table 10.5.5.2.3-1 (nominal resistances are 609 and 632 kips at the abutments and pier, respectively). We recommend that Contractor perform two dynamic pile load tests with 24-hour (minimum) restrike tests at each substructure location (six total) to evaluate hammer system efficiencies, driving stresses in the pile, and the nominal resistance of the piles. We recommend that the dynamic testing be completed prior to production pile driving and that CAPWAP analysis be performed on each pile installed during the dynamic test program (six total). The CAPWAP results will be used to finalize driving criteria for the production piles to assure that the piles achieve the necessary resistance without being overstressed.

SUBMITTAL REVIEWS

The Contract Drawings and Special Provisions (if needed) should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. Standard Specifications require that the Contractor and the Contractor's engineer perform analyses and submit results to MaineDOT for review. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions, and specifications, and to facilitate design changes if subsurface conditions differ from those anticipated prior to the start of construction.

CONSTRUCTION MONITORING

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

- Dynamic testing of driven test piles.
- Installation of production driven piles.
- Placement and compaction of compacted fills.

Limitations and Closure

This report is prepared for the exclusive use of MaineDOT relative to the Wilson Street Bridge Replacement in Brewer, Maine. There are no intended beneficiaries other than MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and from Haley & Aldrich indicating that the Report is adequate for such other use. Use of this report by such other person or entity without the written authorization of MaineDOT and Haley & Aldrich shall be at such other person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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

We understand that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans and specifications) and that the information contained in the CDs takes precedence over the information provided in this report.

We appreciate the opportunity to provide engineering services on this project. Please do not hesitate to call if you have any questions or comments.

Sincerely yours,
HALEY & ALDRICH, INC.

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Enclosures:

- Table I – Preliminary and Final Design Phase Exploration Location Data
- Table II – Preliminary and Final Design Phase Exploration Subsurface Data
- Figure 1 – Project Locus
- Figure 2 – Site and Subsurface Exploration Location Plan 1
- Figure 3 – Site and Subsurface Exploration Location Plan 2
- Figure 4 – Interpretive Subsurface Profile
- Appendix A – Test Boring Logs and Rock Core Photographs
- Appendix B – Observation Well Installation and Groundwater Monitoring Reports
- Appendix C – Laboratory Test Results
- Appendix D – Historic Bridge Drawings and Subsurface Information
- Appendix E – Final Design Calculations

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References

1. *Subsurface Investigation for the Proposed Construction of a Structure to Carry Route 1A over Interstate 395 in the City of Brewer*, Maine Department of Transportation, Materials & Research Division, Soils Section, dated September 1982.

2. *Subsurface Investigation for the Proposed Construction of a Portion of Interstate 395 (Station 300+50 to 330+00) and Ramps at the Route 1A Interchange and the Reconstruction of a Portion of Wilson Street (Route 1A) in the City of Brewer*, Maine Department of Transportation, Materials & Research Division, Soils Section, dated November 1982.

3. *Brewer-Wilson Street Over I-395 Memorandum*, Maine Department of Transportation, Materials & Research Division, Soils Section, dated 12 June 1984.

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TABLE I

Preliminary and Final Design Phase Exploration Location Data
Wilson Street (Route 1A) over Interstate 95, Bridge No. 1564
MaineDOT WIN No. 018915.00
Brewer, Maine

Haley & Aldrich, Inc. File No.: 132076-005

Test Boring No.[1]	Ground Surface Elevation (ft)[3,4]	Station[5]	Offset Distance (ft) & Direction[5]	Coordinates[2]	
				Northing	Easting
BB-BWS-101	119.9	511+33.6	48.1 RT	463,778	1,744,851
BB-BWS-102(OW)	112.2	513+81.8	48.7 RT	463,647	1,745,067
BB-BWS-103	107.1	515+26.0	54.9 RT	463,571	1,745,190
BB-BWS-104(OW)	100.9	516+58.8	62.9 RT	463,500	1,745,302
BB-BWS-105	106.9	518+14.0	52.4 RT	463,437	1,745,447
BB-BWS-201	118.3	512+34.0	81.0 RT	463,694	1,744,920
BB-BWS-202	143.8	513+64.0	45.0 LT	463,737	1,745,097
BB-BWS-203	111.2	515+19.0	14.0 LT	463,610	1,745,203
BB-BWS-204	136.3	516+07.0	39.0 LT	463,614	1,745,306
BB-BWS-205	135.7	516+30.0	76.0 LT	463,635	1,745,344
BB-BWS-206	134.6	516+96.0	37.0 LT	463,570	1,745,383
BB-BWS-301	100.5	525+62.1	84.9 LT	463,233	1,746,181

Notes:

1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.

2 As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference NAD83, Maine 2000 Central Zone coordinate system.

3 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.

4 Elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

5 Station and offset information relative to the Wilson Street baseline determined by MaineDOT and provided to Haley & Aldrich.

	Individual	Date
Prepared By:	NAS	3/10/2020
Checked By:	EMS	4/10/2020
Reviewed By:	WAC	4/10/2020

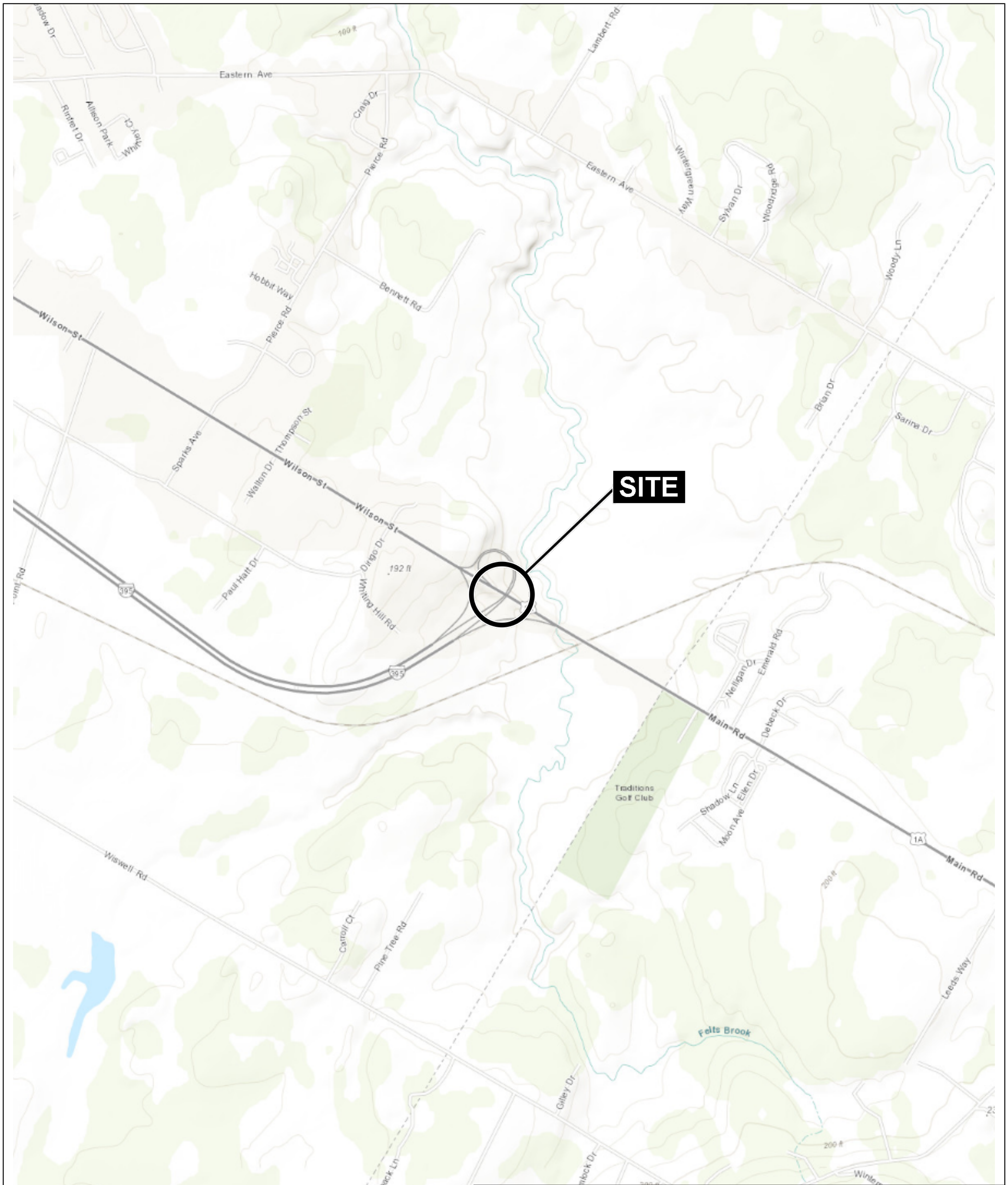
TABLE II
Preliminary and Final Design Phase Exploration Subsurface Data
Wilson Street (Route 1A) over Interstate 95, Bridge No. 1564
MaineDOT WIN No. 018915.00
Brewer, Maine

Haley & Aldrich, Inc. File No.: 132076-005

Test Boring No.[1]	Ground Surface Elevation (ft)[2,3]	Total Exploration Depth (ft)	El. Bottom of Exploration	Embankment Fill[4,5,6]			In-Situ Fill[4,5,6]			Marine Deposit[4,5,6]			Glacial Till/Weathered Bedrock[4,5,6]			Bedrock[4,5,6]	
				Depth to Top (ft)	El. of Top (ft)	Thickness (ft)	Depth to Top (ft)	El. of Top (ft)	Thickness (ft)	Depth to Top (ft)	El. of Top (ft)	Thickness (ft)	Depth to Top (ft)	El. of Top (ft)	Thickness (ft)	Depth to Top (ft)	El. of Top (ft)
PRELIMINARY PHASE TEST BORINGS																	
BB-BWS-101	119.9	45.8	74.1	NE	NE	NE	0.0	119.9	13.0	NE	NE	NE	13.0	106.9	>32.8	--	--
BB-BWS-102(OW)	112.2	51.3	60.9	NE	NE	NE	0.0	112.2	10.0	NE	NE	NE	10.0	102.2	29.8	39.8	72.4
BB-BWS-103	107.1	45.0	62.1	NE	NE	NE	0.0	107.1	3.6	3.6	103.5	16.0	19.6	87.5	14.6	34.2	72.9
BB-BWS-104(OW)	100.9	48.0	52.9	NE	NE	NE	0.0	100.9	6.0	6.0	94.9	19.0	25.0	75.9	11.8	36.8	64.1
BB-BWS-105	106.9	37.0	69.9	NE	NE	NE	0.0	106.9	19.5	19.5	87.4	10.5	30.0	76.9	>1.0	--	--
FINAL PHASE TEST BORINGS																	
BB-BWS-201	118.3	42.0	76.3	NE	NE	NE	0.0	118.3	10.2	NE	NE	NE	10.2	108.1	>31.8	--	--
BB-BWS-202	143.8	81.7	62.1	0.7	143.1	21.5	22.2	121.6	21.8	44.0	99.8	21.0	65.0	78.8	7.5	72.5	71.3
BB-BWS-203	111.2	60.0	51.2	NE	NE	NE	0.0	111.2	9.1	9.1	102.1	27.5	36.6	74.6	7.9	44.5	66.7
BB-BWS-204	136.3	37.0	99.3	0.4	135.9	22.9	23.3	113.0	3.4	NE	NE	NE	26.7	109.6	>10.3	--	--
BB-BWS-205	135.7	56.0	79.7	0.5	135.2	22.2	22.7	113.0	23.7	46.4	89.3	7.6	54.0	81.7	>2.0	--	--
BB-BWS-206	134.6	89.0	45.6	0.8	133.8	20.8	21.6	113.0	27.6	49.2	85.4	4.8	54.0	80.6	25.0	79.0	55.6
BB-BWS-301	100.5	42.1	58.4	NE	NE	NE	0.0	100.5	13.7	13.7	86.8	22.1	35.8	64.7	5.3	41.1	59.4
AVERAGE VALUE[7]	119.0	52.9	66.0	0.6	137.0	21.9	7.5	111.5	13.5	23.9	93.7	16.1	31.7	87.3	14.6	49.7	66.1

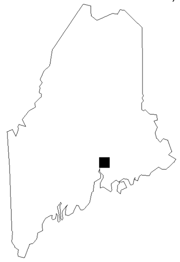
Notes:
[1] Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
[3] Elevations are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[4] "NE" indicates stratum was not encountered in test boring. Estimated in-situ fill thickness based on historical geotechnical report.
[5] "--" indicates test boring was not drilled deep enough to determine presence of stratum.
[6] "> 32.8" indicates test boring was not drilled deep enough to determine entire stratum thickness. Actual total stratum thickness greater than value shown.
[7] Average does not include "NE", ">", or "--" values.

	Individual	Date
Prepared By:	NAS	3/10/2020
Checked By:	EMS	4/10/2020
Reviewed By:	WAC	4/10/2020



MAP SOURCE: ESRI

SITE COORDINATES: 44°46'17"N, 68°43'18"W



132076-003_1_LOCUS.PDF

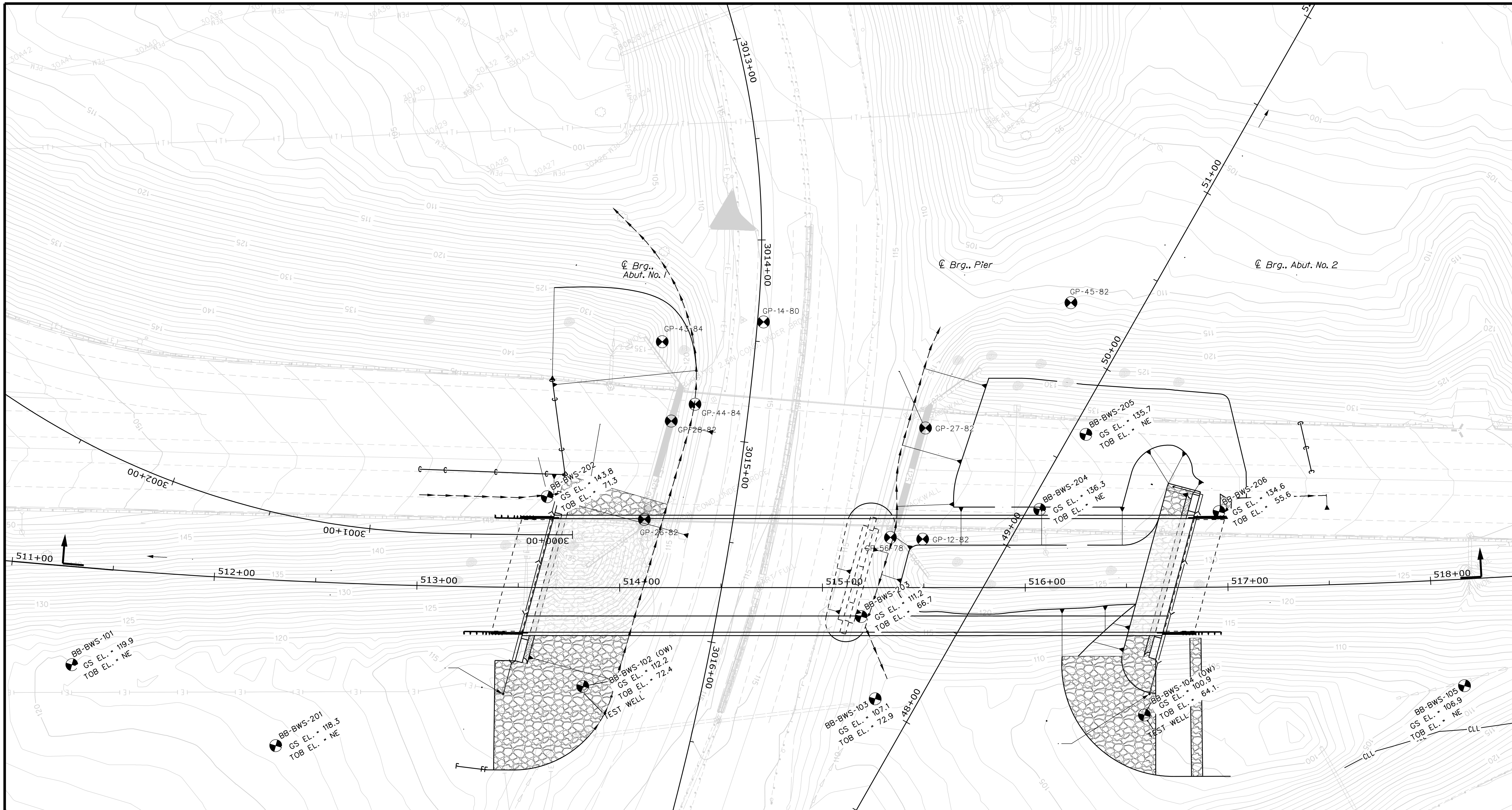
HALEY ALDRICH

WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO.1564
MAINEDOT WIN 018915.20
BREWER, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
MAY 2020

FIGURE 1



LEGEND

BB-BWS-203 GS EL. = 111.2 TOB EL. = 66.7	DESIGNATION, AS-DRILLED GROUND SURFACE ELEVATION (GS), APPROXIMATE TOP OF BEDROCK (TOB) ELEVATION, AND AS-DRILLED LOCATION OF FINAL DESIGN PHASE TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS AND MONITORED BY HALEY & ALDRICH, INC. IN JANUARY 2020.
BB-BWS-103 GS EL. = 107.91 TOB EL. = 72.9	DESIGNATION, AS-DRILLED GROUND SURFACE ELEVATION (GS), APPROXIMATE TOP OF BEDROCK (TOB) ELEVATION, AND AS-DRILLED LOCATION OF PRELIMINARY DESIGN PHASE TEST BORING DRILLED BY NORTHERN TEST BORING, INC. AND MONITORED BY HALEY & ALDRICH, INC. IN JULY 2018.
GP-45-82	DESIGNATION AND APPROXIMATE LOCATION OF HISTORIC TEST BORING.
	DESIGNATION, APPROXIMATE LOCATION, AND ORIENTATION OF INTERPRETIVE SUBSURFACE PROFILE (SEE FIGURE 4).

KEY

BB = Bridge Boring
NE = Not Encountered
OW = Observation well installed in completed borehole.

PLAN

25 0 25 50
Scale of Feet

HALEY ALDRICH

WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO. 1564
MAINEDOT WIN 018915.20
BREWER, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN (1 OF 2)

SCALE: AS SHOWN
MAY 2020

FIGURE 2

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(500)

BRIDGE NO.

WIN 018915.00

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	N. Sherwood	3-17-20
CHECKED-REVIEWED	K. Post	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

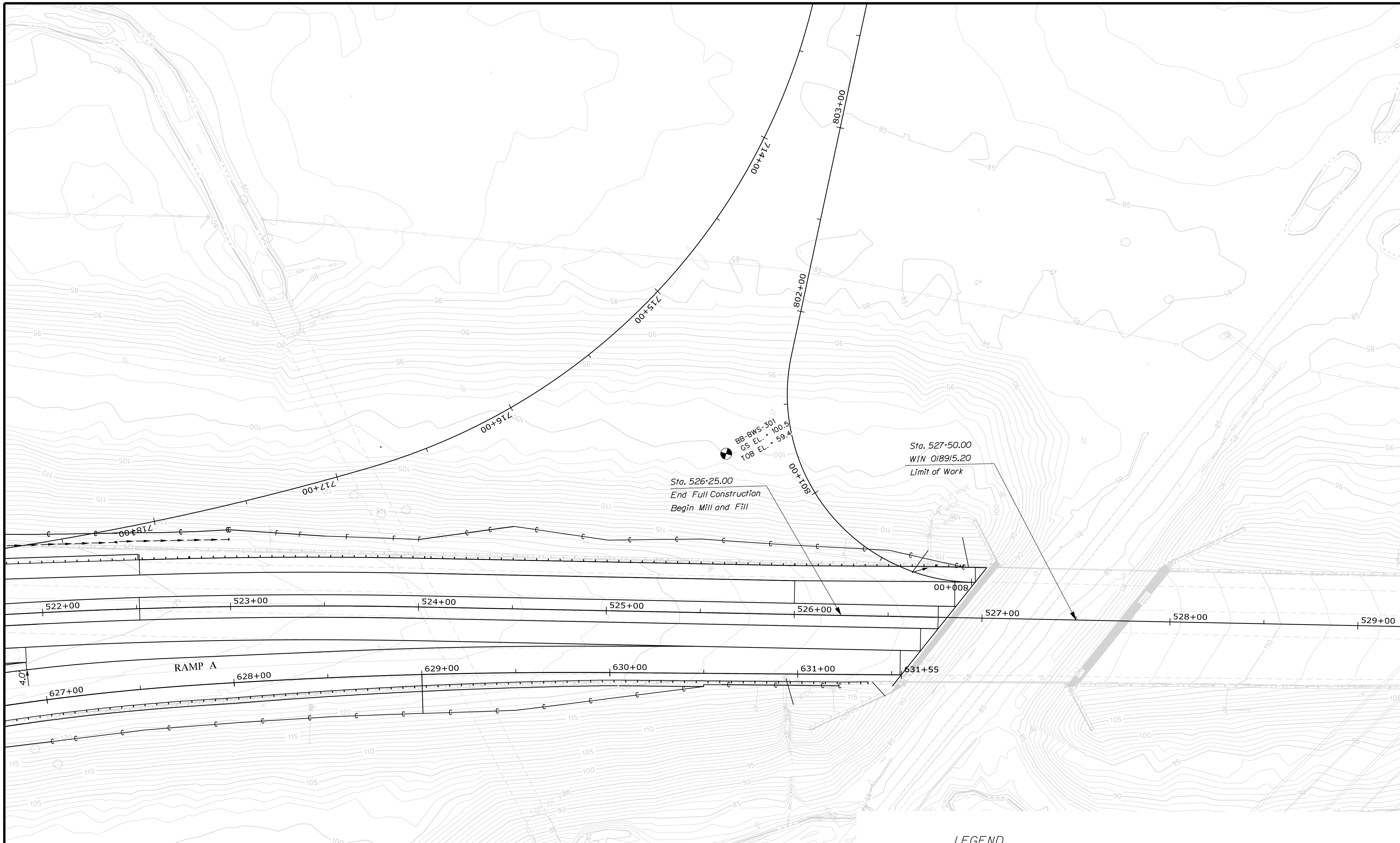
I-395 - ROUTE 9 CONNECTOR
INTERSTATE 95
BREWER-EDDINGTON PENOBSCOT COUNTY

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SHEET NUMBER

Filename: ...\002_GenPlan Wilson Street .dgn
Division:
Username:
Date:5/5/2020

Filename: ...\005_GenPlan Wilson Street Ramp .dgnDivision:
Username:
Date:5/5/2020



LEGEND

BB-BWS-301 GS EL. = 100.5 TOB EL. = 59.4	DESIGNATION, AS-DRILLED GROUND SURFACE ELEVATION (GS), APPROXIMATE TOP OF BEDROCK (TOB) ELEVATION, AND AS-DRILLED LOCATION OF FINAL DESIGN PHASE TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS AND MONITORED BY HALEY & ALDRICH, INC. IN JANUARY 2020.

PLAN

25 0 25 50
Scale of Feet

HALEY ALDRICH

WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO. 1564
MAINEDOT WIN 018915.20
BREWER, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN (2 OF 2)

SCALE: AS SHOWN
MAY 2020

FIGURE 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(500)

BRIDGE PLANS
WIN 018915.00
BRIDGE NO.

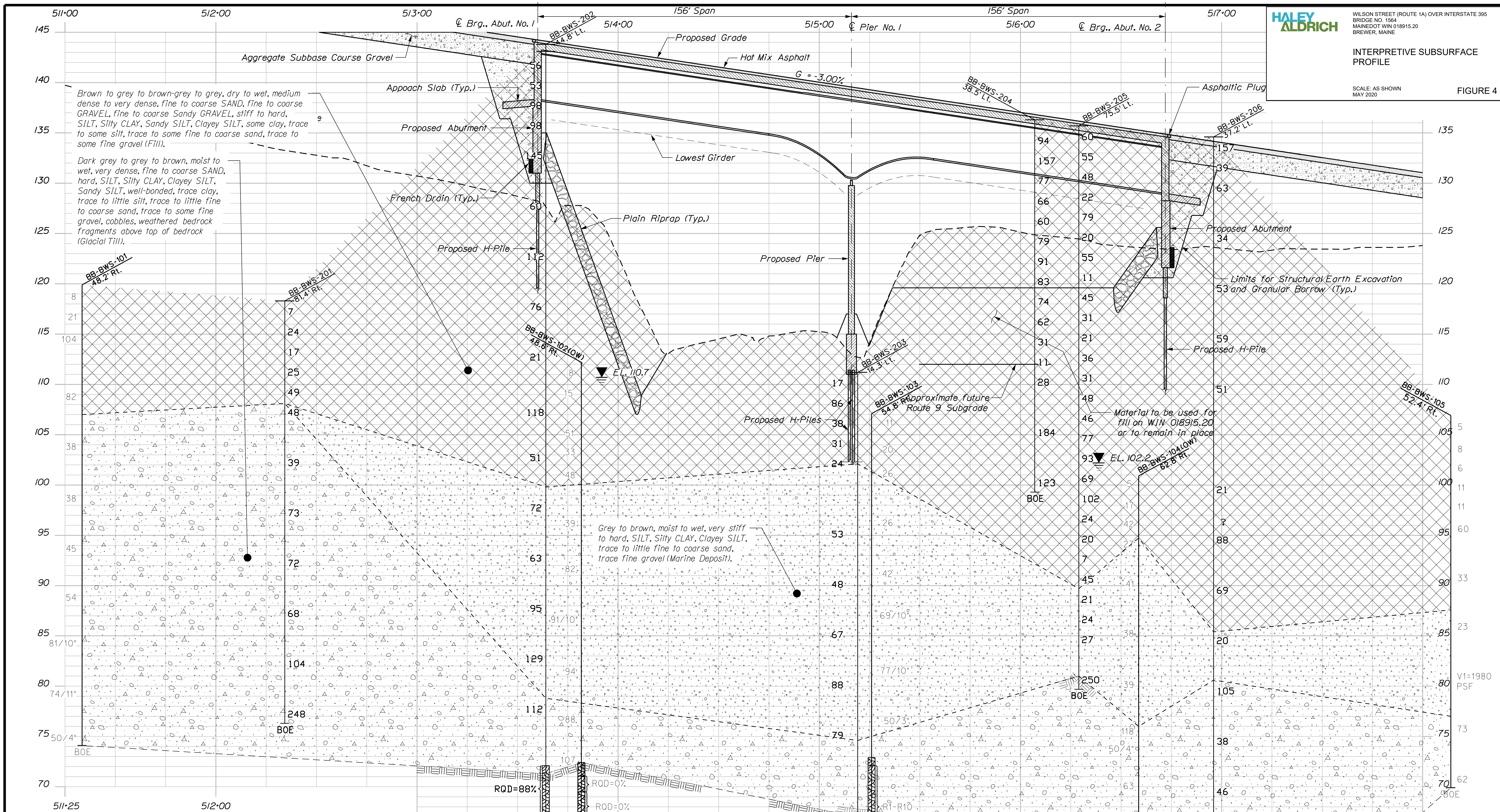
	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	N. Sherwood	K. Post	3-10-20
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

I-395 - ROUTE 9 CONNECTOR
INTERSTATE 95
BREWER-EDDINGTON PENOBSCOT COUNTY

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SHEET NUMBER
20
OF 138



LEGEND

- Boring No. / Offset
- Pavement Thickness, if applicable
- 10 — Energy Corrected Standard Penetration Test (SPT) N60-Value (Blows Per Foot)
- V1=855 psf — Denotes in-situ Vane Shear Test Performed at depth shown with Peak Undrained Shear Strength Provided
- Weathered Bedrock, if applicable
- Approximate Top of Bedrock
- RQD= Rock Quality Designation of Bedrock Core Sample
- BOE= Bottom of Exploration
- BOE — No Refusal
- R — Refusal
- Approximate elevation of stabilized groundwater
- WOH = Weight of Hammer
- WOR= Weight of Rods

Notes:
1. This generalized interpretive subsurface profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

HALEY ALDRICH

WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO. 1564
MAINEDOT WIN 018915.20
BREWER, MAINE

INTERPRETIVE SUBSURFACE PROFILE

SCALE: AS SHOWN
MAY 2020

FIGURE 4

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(500)

WIN 018915.00

BRIDGE PLANS

BRIDGE NO.

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	N. Sherwood	
CHECKED-REVIEWED	K. Post	3-17-20
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

I-395 - ROUTE 9 CONNECTOR
INTERSTATE 95
BREWER-EDDINGTON PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

21

OF 138

Filename: ...\CAD\004_Wilson St.Profile.dgn
Division:
Username:
Date:5/5/2020

APPENDIX A

Test Boring Logs and Rock Core Photographs

HALEY ALDRICH

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Route 9/I-395 Connector **Location:** Brewer and Eddington, Maine	**Boring No.:** BB-BWS-101 **WIN:** 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	119.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-13-18/07-13-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	--
Boring Location:	Sta. 511+33.6, 48.1 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	2/2/3/4	5	8	HSA			Grey-brown, damp, medium stiff, SILT, trace fine sand, trace organics -FILL-(ML)	
	2D	24/19	2.00 - 4.00	4/6/8/12	14	21				Grey-brown, moist, very stiff, Clayey SILT, trace organics -FILL-(ML)	
	3D	16/18	4.00 - 5.33	16/19/50(4")	69	104				Brown, moist, hard, fine to coarse Sandy SILT, trace fine to coarse gravel -FILL-(ML)	
5								114.9		5.0 Note: Cobble encountered at approximately 5.0 ft.	
10	4D	24/12	10.00 - 12.00	44/37/17/18	54	82	HW			Brown, wet, very dense, Silty SAND, some fine to coarse gravel -FILL-(SM)	
								106.9		Note: Drill action and wash water contents indicate stratum change at approximately 13.0 ft. 13.0	
15	5D	24/17	15.00 - 17.00	10/11/14/17	25	38				Grey, moist, hard, Clayey SILT, trace medium to coarse sand, trace fine gravel -GLACIAL TILL-(ML)	
20	6D	24/20	20.00 - 22.00	10/11/14/17	25	38				Grey, moist, hard, Clayey SILT, trace fine gravel -GLACIAL TILL-(ML)	
25										Note: Drill action indicates greater resistance at approximately 24.0 ft.	

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 10 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.

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: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine
Boring No.: BB-BWS-101
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	119.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-13-18/07-13-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	--
Boring Location:	Sta. 511+33.6, 48.1 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/16	25.00 - 27.00	10/11/19/30	30	45	HW			Grey, moist, hard, Clayey SILT, trace fine to coarse sand, trace fine gravel -GLACIAL TILL-(ML)	
30	8D	24/16	30.00 - 32.00	12/15/21/31	36	54				Grey, moist, hard, Clayey SILT, trace coarse sand, trace coarse gravel -GLACIAL TILL-(ML)	
35	9D	16/10	35.00 - 36.33	17/31/50(4")						Grey, moist, hard, Clayey SILT, trace coarse sand, trace coarse gravel -GLACIAL TILL-(ML) Note: Drill action indicates coarse material at approximately 37.0 ft.	
40	10D	16.92/23	40.00 - 41.41	21/24/50(5")						Grey, moist, hard, Clayey SILT, little fine to coarse sand, little fine to coarse gravel -GLACIAL TILL-(ML) Note: Drill action indicates coarse material from approximately 42.0 to 45.0 ft.	
45	11D	10/13	45.00 - 45.83	50/50(4")				74.1		Grey, moist, hard, Sandy SILT, little fine to coarse gravel -GLACIAL TILL-(ML) 45.8 **Bottom of Exploration at 45.8 feet below ground surface.**	
50											

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 10 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.

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-BWS-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine	Boring No.: BB-BWS-102(OW) WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	112.2	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-12-18/07-13-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+81.8, 48.7 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks
Hammer Efficiency Factor: 0.907		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	2/2/3/3	5	8	SSA			Brown, moist, medium stiff, SILT, little fine to coarse sand, trace fine gravel, trace organics -FILL-(ML)	
								111.2		1.0	
	2D	24/18	2.00 - 4.00	3/4/6/5	10	15				Grey-brown, moist, stiff, Clayey SILT, trace fine sand, trace organics -FILL-(ML)	
	3D	24/19	4.00 - 6.00	4/17/17/12	34	51				Grey-brown, moist, hard, Clayey SILT, trace fine to coarse sand, trace fine to coarse gravel, trace organics -FILL-(ML)	
5											
	4D	24/24	6.00 - 8.00	7/11/11/9	22	33				Brown, wet, hard, SILT, trace clay, little fine to coarse sand, trace fine to coarse gravel, trace organics -FILL-(ML) Brown, moist, hard, SILT, some fine to coarse sand, trace fine to coarse gravel -FILL-(ML)	
								104.2		8.0	
	5D	24/3	8.00 - 10.00	9/11/21/30	32	48				Brown, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt -FILL-(GW) Note: Low recovery due to probable cobble.	
10								102.2		10.0	
	6D	24/19	10.00 - 12.00	12/12/14/16	26	39	HW			Brown, moist, hard, SILT, trace fine sand, trace fine gravel -GLACIAL TILL-(ML)	
15	7D	24/23	15.00 - 17.00	20/20/34/39	54	82				Grey, damp, hard, SILT, trace fine to coarse sand, trace fine gravel -GLACIAL TILL-(ML)	
20	8D	16/18	20.00 - 21.33	20/41/50(4")						Grey, damp, hard, SILT, little fine to coarse sand, little fine to coarse gravel -GLACIAL TILL-(ML)	
25											

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 10 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.
2. Observation well installed in completed borehole. See observation well installation and groundwater monitoring reports for details.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-102(OW)
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	112.2	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-12-18/07-13-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+81.8, 48.7 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	9D	22/19	25.00 - 26.83	17/27/35/50(3")	62	94	HW			Grey, damp, hard, SILT, trace coarse sand, trace fine gravel -GLACIAL TILL-(ML)	
30	10D	22/21	30.00 - 31.83	14/31/27/50(4")	58	88				Grey, moist, hard, SILT, little fine to coarse sand, little fine to coarse gravel -GLACIAL TILL-(ML)	
35	11D	14/14	35.00 - 36.17	17/21/50(2")	71	107				Grey, moist, hard, SILT, little fine to coarse sand, little fine to coarse gravel -GLACIAL TILL-(ML)	
										Note: Drill action and wash indicating potential rock.	
40							RC	72.4		39.8 Top of Bedrock at El. 72.4	
	R1	15.6/15	41.00 - 42.30	RQD = 0%			NQ CORE			R1: Grey, aphanitic SILTSTONE with occasional calcite veins. Moderately hard, fresh to slightly weathered. Few discernible joints, moderate to steeply dipping, slight staining on few joint surfaces, highly fractured Rock Quality=Very Poor Recovery=94% -BREWER FORMATION- R1 Core Times (min:sec): 41.0-42.0'(2:45); 42.0-42.3' (1:00)	
	R2	38.4/38	42.30 - 45.50	RQD = 0%						R2: Grey, aphanitic SILTSTONE with occasional calcite veins. Moderately hard, fresh to slightly weathered. Joints dipping at moderate to steep angles, very close spacing, tight to open, smooth to rough, slight staining on few joint surfaces. Rock Quality=Very Poor Recovery=100% -BREWER FORMATION- R2 Core Times (min:sec): 42.3-43.3' (5:30); 43.3-44.3' (5:45); 44.3-45.5' (3:45)	
45	R3	51.6/42	45.50 - 49.80	RQD = 10%						R3: Grey, aphanitic SILTSTONE with occasional calcite veins. Moderately hard, fresh. Joints dipping at steep angles, very close to	
50	R4	18/18	49.80 - 51.30	RQD = 0%							

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-102(OW)
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	112.2	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-12-18/07-13-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+81.8, 48.7 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							NQ CORE	60.9		close spacing, tight to open, smooth, slight staining on few joint surfaces. Rock Quality=Very Poor Recovery=81% -BREWER FORMATION- R3 Core Times (min:sec): 45.5-46.5' (2:15); 46.5-47.5' (2:30); 47.5-48.5' (3:45); 48.5-49.8' (2:30) R4: Grey, aphanitic SILTSTONE with occasional calcite veins. Moderately hard, fresh to slightly weathered. Joints dipping at steep angles, very close to close spacing, tight to open, smooth, slight staining on few joint surfaces. Rock Quality=Very Poor Recovery=100% -BREWER FORMATION- R4 Core Times (min:sec): 49.8-50.8' (1:45); 50.8-51.3' (0:45) 51.3 **Bottom of Exploration at 51.3 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-103
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	107.1	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-9-18/07-9-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 515+26, 54.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	3.8 ft
Hammer Efficiency Factor: 0.907		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.00 - 2.00	1/1/6/7	7	11	HSA			Brown, moist, medium dense, fine to medium SAND, little silt, trace organics -FILL-(SM)	
										Note: Weathered rock within spoon tip. Cobble zone from 2.2 to 2.8 ft.	
	2D/2A	24/18	3.00 - 5.00	5/6/7/9	13	20		103.5		Dark grey, wet, medium dense, fine to coarse SAND, little silt, trace fine to coarse gravel, well graded -FILL-(SM)	
										3.6	
5	3D	24/12	5.00 - 7.00	7/8/9/11	17	26	51			Olive-brown, damp to moist, very stiff, Clayey SILT, slightly mottled -MARINE DEPOSIT-(ML)	
							73			Olive-brown, damp, very stiff, SILT, little clay, fine sand and coarse gravel, trace medium to coarse sand and fine gravel, slightly mottled -MARINE DEPOSIT-(CL-ML)	G#474545 A-4(0) (CL-ML) LL=22 PL=16 PI=6 WC=13.1
							75				
							81				
							68				
10	4D	24/22	10.00 - 12.00	7/7/10/12	17	26	32			Brown, damp, very stiff, SILT, little clay, trace fine to coarse sand and fine gravel -MARINE DEPOSIT-(CL-ML)	G#474546 A-4(0) (CL-ML) LL=19 PL=15 PI=4 WC=16.0
							51				
							79				
							183				
							221				
15	5D	24/21	15.00 - 17.00	12/14/14/15	28	42	86			Brown, damp, hard, SILT, little clay, trace fine to coarse sand and fine gravel -MARINE DEPOSIT-(CL-ML)	G#474547 A-4(4) (CL-ML) LL=23 PL=16 PI=7 WC=15.4
							75				
							96				
							118				
							104	87.5		Note: Color change to gray at approximately 19.6 ft. 19.6	
20	6D	16/16	20.00 - 21.33	15/19/50(4")			HW			Grey, damp, hard, SILT, trace fine gravel -GLACIAL TILL-(ML)	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector				**Boring No.:**	BB-BWS-103
Location: Brewer and Eddington, Maine				**WIN:**	18915.00
Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	107.1	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-9-18/07-9-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 515+26, 54.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	3.8 ft
Hammer Efficiency Factor: 0.907		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	16/16	25.00 - 26.33	19/27/50(4")			HW			Grey, damp, hard, SILT, trace fine to coarse gravel -GLACIAL TILL-(ML)	
30	8D	9/10	30.00 - 30.75	22/50(3")						Grey, damp, hard, SILT, trace fine to coarse sand, trace fine to coarse gravel -GLACIAL TILL-(ML)	
							RC	72.9		34.2 Top of Bedrock at El. 72.9	
35	R1	14.4/9	35.00 - 36.20	RQD = 0%			NQ CORE			R1: Grey, aphanitic SILTSTONE with occasional thin calcite veins. Hard, fresh to slightly weathered. Primary joints dipping at moderate to steep angles, spaced very close, tight to open, planar, smooth. Slight oxidation on few joint surfaces. Note: Core barrel frequently jamming due to steep angle joints. Rock Quality=Very Poor Recovery=64% -BREWER FORMATION-	
	R2	13.2/9	36.20 - 37.30	RQD = 0%						R2: Similar to R1 above Recovery=69% R2 Core Times (min:sec): 36.2-37.3' (1:15)	
	R3	8.4/6	37.30 - 38.00	RQD = 0%						R3: Similar to R1 above Recovery=75% R3 Core Times (min:sec): 36.8-38.0' (2:45)	
	R4	12/12	38.00 - 39.00	RQD = 0%						R4: Similar to R1 above Recovery=100% R4 Core Times (min:sec): 38.0-39.0' (2:00)	
	R5	13.2/11	39.00 - 40.10	RQD = 0%						R5: Similar to R1 above Recovery=85% R5 Core Times (min:sec): 39.0-40.0' (3:00); 40.0-40.1' (0:45)	
40	R6	10.8/7	40.10 - 41.00	RQD = 0%						R6: Similar to R1 above Recovery 89% R6 Core Times (min:sec): 40.1-41.0' (2:00)	
	R7	14.4/7	41.00 - 42.20	RQD = 0%						R7: Similar to R1 above, except slightly to moderately weathered Recovery=50% R7 Core Times (min:sec): 41.0-42.0' (2:75); 42.0-42.1' (0.45)	
	R8	13.2/2	42.20 - 43.30	RQD = 0%						R8: Similar to R1 above Recovery=15% R8 Core Times (min:sec): 42.2-43.2' (3:00); 43.2'-43.3' (0:15)	
	R9	13.2/11	43.30 - 44.40	RQD = 0%						R9: Similar to R1 above Recovery=85% R9 Core Times (min:sec): 43.3-44.3' (2:45); 44.3-44.4' (0:15)	
	R10	7.2/5	44.40 - 45.00	RQD = 0%						R10: Similar to R1 above	
45								62.1			
50											

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 20 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 9/I-395 Connector			**Boring No.:**	BB-BWS-103
Location:	Brewer and Eddington, Maine			**WIN:**	18915.00
Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	107.1	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-9-18/07-9-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 515+26, 54.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	3.8 ft
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										Recovery=71% R10 Core Times (min:sec): 44.5-45.0' (2:45)	
										45.0	
										Bottom of Exploration at 45.0 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-104(OW)
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	100.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-10-18/07-10-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+58.8, 62.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks
Hammer Efficiency Factor: 0.907		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/21	0.00 - 2.00	1/1/2/3	3	5	SSA			Grey grading to brown, moist, medium stiff, Clayey SILT, trace fine sand, trace organics, slightly mottled, naturally-deposited reworked soil -FILL-(ML)	
	2D	24/15	2.00 - 4.00	3/4/7/9	11	17				Grey-brown, moist, very stiff, Clayey SILT -FILL-(ML)	
								96.9		4.0	
	3D	24/13	4.00 - 6.00	12/14/14/7	28	42				Brown, wet, dense, Silty fine to coarse GRAVEL, some fine to coarse sand -FILL-(GM)	
5											
								94.9		6.0	
	4D	24/17	6.00 - 8.00	7/8/10/11	18	27	30			Brown, moist, very stiff, SILT, little clay, trace fine to coarse sand -MARINE DEPOSIT-(CL-ML)	G#474548 A-4(2) (CL-ML) LL=21 PL=16 PI=5 WC=17.4
							46				
							51				
							63			Note: Gradual color change at approximately 9.8 ft.	
								91.1		9.8	
10	5D	24/18	10.00 - 12.00	8/11/16/17	27	41	OPEN			Grey, moist, hard, SILT, little clay, trace fine to coarse sand, trace fine gravel -MARINE DEPSOIT-(CL-ML)	G#474549 A-4(4) (CL-ML) LL=24 PL=17 PI=7 WC=15.4
										Note: Drill action indicates probable cobble from approximately 14.5 to 15.0 ft.	
15	6D	24/20	15.00 - 17.00	9/11/14/16	25	38				Grey, moist, hard, SILT, little clay, trace fine to coarse sand and fine gravel -MARINE DEPOSIT-(CL-ML)	G#474550 A-4(3) (CL-ML) LL=21 PL=14 PI=7 WC=12.2
20	7D	24/21	20.00 - 22.00	12/12/14/14	26	39				Grey, moist, hard, SILT, some clay, trace fine to coarse sand -MARINE DEPOSIT-(CL-ML)	G#474551 A-4(2) (CL-ML) LL=21 PL=16 PI=5 WC=14.7
25											

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 10 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.
2. Observation well installed in completed borehole. See observation well installation and groundwater monitoring reports for details.

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: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-104(OW)
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	100.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-10-18/07-10-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+58.8, 62.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks

Hammer Efficiency Factor: 0.907 **Hammer Type:** Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8MD	16/0	25.00 - 26.33	16/28/50(4")	78	118	HW	75.9		25.0 No recovery, possibly pushed on cobble.	
	9D	10/10	27.00 - 27.83	22/50(4")						Grey, moist, hard, SILT, little fine to coarse sand, trace fine to coarse gravel -GLACIAL TILL-(ML)	
30	10D	24/20	30.00 - 32.00	10/11/31/31	42	63				Grey, dry, hard, SILT, some fine to coarse sand, trace fine to coarse gravel -GLACIAL TILL-(ML)	
35	11D	9.96/11	35.00 - 35.83	30/50(4")						Grey, dry, hard, SILT, some fine to coarse sand, trace fine to coarse gravel -GLACIAL TILL-(ML)	
							RC	64.1		36.8 Top of Bedrock at El. 64.1	
	R1	60/60	38.00 - 43.00	RQD = 70%			NQ CORE			R1: Grey, fine-grained to aphanitic SILTSTONE with frequent thin calcite or quartz veins, hard, fresh. Primary joints dipping at steep angles, spaced very close to close, tight to open, planar to undulating, smooth. Secondary joints dipping at low to moderate angles, spaced moderately close, tight to open, planar to undulating, smooth. Few joint faces healed with calcite. Fracturing frequency decreases with depth.Slight pitting observed. Rock Quality=Fair Recovery=100% -BREWER FORMATION- R1 Core Times (min:sec): 38.0-39.0' (10:60); 39.0-40.0' (8:00); 40.0-41.0' (7:15); 41.0-42.0' (3:30); 42.0-43.0' (4:15)	qp=2,490 psi (41.3-41.9')
40											
	R2	35/35	43.00 - 45.92	RQD = 0%						R2: Core barrel became plugged at 45.9 ft. Similar to R1 with 1 to 2-in. thick quartz intrusions. Highly fractured due to drilling process. Rock Quality=Very Poor Recovery=100% -BREWER FORMATION- R2 Core Times (min:sec): 45.0-45.9' (1:00)	
45	R3	25/25	45.92 - 48.00	RQD = 76%						R3: White aphanitic to medium grained quartz intrusion. Hard, fresh. Moderate angle contact with parent rock. Rock Quality=Very Good Recovery=76% -BREWER FORMATION R3 Core Times (min:sec): 45.9-46.9' (4:00); 46.9-47.9' (7:00); 47.9-48.0' (2:30)	
								52.9		48.0	
50											

Remarks:

1. Washed ahead of casing in approximate 5-ft intervals below 10 ft. Casing driven (advanced) after washing ahead, casing blows not recorded.
2. Observation well installed in completed borehole. See observation well installation and groundwater monitoring reports for details.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-104(OW)
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	100.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-10-18/07-10-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+58.8, 62.9 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										**Bottom of Exploration at 48.0 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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Boring No.: BB-BWS-104(OW)

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine
Boring No.: BB-BWS-105
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	106.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-11-18/07-11-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 518+14, 52.4 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	1/1/2/4	3	5	SSA			Grey-brown, moist, medium stiff, Silty CLAY, trace organics, trace fine gravel -FILL-(CL)	
	2D	24/13	2.00 - 4.00	2/2/3/4	5	8				Grey, moist, medium stiff, Silty CLAY, 1-in. peat layer -FILL-(CL)	
	3D	24/22	4.00 - 6.00	2/2/2/2	4	6				Grey, moist, medium stiff, Silty CLAY -FILL-(CL)	
5											
	4D	24/19	6.00 - 8.00	3/3/4/3	7	11				Grey, moist, stiff, Silty CLAY -FILL-(CL)	
								99.4		7.5	
	5D	24/13	8.00 - 10.00	3/3/4/3	7	11		98.9		Grey-brown, moist, stiff, Clayey SILT, trace fine to coarse sand, trace fine to coarse gravel -FILL-(ML)	
										8.0	
								97.4		Grey, moist, stiff, Silty CLAY -FILL-(CL)	
10	6D	24/19	10.00 - 12.00	8/17/23/29	40	60	HW			9.5	
										Grey-brown, moist, hard, Clayey SILT, trace fine to coarse gravel -FILL-(ML)	
										Brown, moist, hard, SILT, trace fine to coarse gravel, trace fine to coarse sand -FILL-(ML)	
15								91.9		15.0	
	7D	24/20	15.00 - 17.00	7/13/9/19	22	33				Grey, moist, dense, fine to coarse SAND, little fine to coarse gravel, little silt, top 10 in. of sample -FILL-(SP)	
								90.9		16.0	
										Brown, moist, hard, SILT, trace organics -FORMER TOPSOIL/FILL-(ML)	
								87.4		19.5	
20	8D	24/24	20.00 - 22.00	7/7/8/9	15	23				Olive-brown, moist, very stiff, Silty CLAY, mottled -MARINE DEPOSIT-(CL)	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: BB-BWS-105
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	106.9	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	N. Klausmeyer	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	07-11-18/07-11-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 518+14, 52.4 Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	9D V1	24/24	25.00 - 27.00 25.64 - 26.00	3/3/3/3 Su=1,980/390 psf	6	9	HW			Olive-brown, moist, stiff, Silty CLAY, mottled -MARINE DEPOSIT-(CL) 55x110 mm raw torque reading: V1: 510/10 in-lbs	
30	10D	24/7	30.00 - 32.00	21/22/26/23	48	73		76.9		30.0 Grey, moist, very dense, fine to coarse SAND, some silt, little fine to coarse gravel -GLACIAL TILL-(SM)	
35	11D	24/15	35.00 - 37.00	23/22/19/17	41	62		70.9 69.9		Grey, moist, very dense, fine to coarse SAND, some silt, little fine to coarse gravel -GLACIAL TILL-(SM) 36.0 Grey, moist, hard, Clayey SILT, some fine sand, trace coarse sand, trace fine gravel -GLACIAL TILL-(ML) 37.0 **Bottom of Exploration at 37.0 feet below ground surface.**	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-201
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	118.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-11-19/12-12-19	**Drilling Method:**	HW Drive to 30.0'	**Core Barrel:**	--
Boring Location:	Sta. 512+34, 81R	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11.0 ft
Hammer Efficiency Factor: 0.842		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.0 - 2.0	1/2/3/2	5	7	HW PUSH			Brown mottled, wet, medium stiff, Silty CLAY, organics -FILL-(CL)	
	2D	24/22	2.0 - 4.0	3/7/10/15	17	24				Similar to 1D, except very stiff -FILL-(CL)	
	3D	24/24	4.0 - 6.0	3/6/6/9	12	17				Similar to 1D, except very stiff -FILL-(CL)	
5											
	4D	24/24	6.0 - 8.0	13/9/9/39	18	25				Similar to 1D, except very stiff, trace gravel -FILL-(CL)	
							44	110.7		7.6	
	5D	24/19	8.0 - 10.0	13/18/17/17	35	49	122			Note: Gravel layer. Brown, dry, hard, SILT, little medium sand, little gravel -FILL-(ML)	
							130				
10	6D	24/19	10.0 - 12.0	24/20/14/12	34	48	HW	108.1		10.2 Brown, moist, dense, fine to medium SAND, some silt, little gravel -GLACIAL TILL-(SM)	
15	7D	24/5	15.0 - 17.0	10/13/15/18	28	39		103.3		15.0 Brown, moist, hard, Silty CLAY, trace medium sand -GLACIAL TILL-(CL)	
20	8D	24/20	20.0 - 22.0	14/31/21/37	52	73				Brown, moist, hard, Silty CLAY, trace coarse sand, trace fine gravel -GLACIAL TILL-(CL)	
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine
Boring No.: BB-BWS-201
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	118.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-11-19/12-12-19	**Drilling Method:**	HW Drive to 30.0'	**Core Barrel:**	--
Boring Location:	Sta. 512+34, 81R	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	11.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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T_V = Pocket Torvane Shear Strength (psf)
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	9D	24/13	25.0 - 27.0	22/27/24/18	51	72	HW				
								92.2		26.1 Grey, moist, very dense, fine to medium SAND, some silt, trace gravel -GLACIAL TILL-(SM)	
30	10D	24/24	30.0 - 32.0	27/37/24/89	61	86	OPEN	88.3		30.0 Grey, wet, very dense, fine Sandy SILT, trace gravel -GLACIAL TILL-(ML)	
35	11D	24/24	35.0 - 37.0	22/33/41/63	74	104				Grey, moist, very dense, fine Sandy SILT -GLACIAL TILL-(ML)	
40	12D	24/24	40.0 - 42.0	63/85/92/100	177	248		78.3		40.0 Grey, moist, very dense, fine to medium SAND, some silt, trace gravel -GLACIAL TILL-(SM)	
								76.3		42.0 **Bottom of Exploration at 42.0 feet below ground surface.**	
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-202
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.8	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53; NEBC #23 hammer	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-02-19/12-05-19	**Drilling Method:**	SSA 0-10'; NW/HW Drive to 72.5'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+64, 45L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	35.8 ft
Hammer Efficiency Factor: 0.842		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	143.1		-BITUMINOUS CONCRETE- 0.7	
	1D	24/18	1.0 - 3.0	22/18/22/32	40	56				Brown, dry, very dense, fine to medium SAND, some silt, little fine to coarse gravel -FILL-(SM)	
								140.8		3.0	
	2D	24/18	3.0 - 5.0	18/19/19/50(4")	38	53				Brown, dry, very dense, fine to medium SAND, some fine to coarse gravel, little silt -FILL-(SP-SM)	
5								138.8		5.0	
	3D	24/15	5.0 - 7.0	10/30/40/26	70	98				Brown-grey, dry, very dense, fine to coarse SAND, some fine to coarse gravel, little silt -FILL-(SW-SM)	
	4D	24/15	7.0 - 9.0	19/37/33/40	70	98				Brown to tan, dry, very dense, fine to coarse SAND, little silt, little fine to coarse gravel -FILL-(SW-SM)	
10								133.8		10.0	
	5D	24/12	10.0 - 12.0	33/41/62/52	103	145	HW			Brown to grey, moist to wet, very dense, fine to coarse SAND, some gravel, trace silt -FILL-(SW)	
15								128.8		15.0	
	6D	24/15	15.0 - 17.0	25/24/19/59	43	60	45			Brown, moist, very dense, fine to medium SAND, little gravel, trace silt, trace coarse sand -FILL-(SP)	
							28				
							65				
							150				
							94				
20								123.8		20.0	
	7D	24/13	20.0 - 22.0	28/39/41/54	80	112	60			Brown, moist, very dense, fine to coarse SAND, trace fine gravel -FILL-(SW)	
							62				
							225				
							172				
25							203				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Wilson Street Bridge Replacement			**Boring No.:**	BB-BWS-202
Location:	Brewer, Maine			**WIN:**	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	143.8	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53; NEBC #23 hammer	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-02-19/12-05-19	**Drilling Method:**	SSA 0-10'; NW/HW Drive to 72.5'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+64, 45L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	35.8 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	24/15	25.0 - 27.0	21/26/28/32	54	76	51	118.8		25.0 Brown, moist, very dense, fine to coarse SAND, some gravel -FILL-(SW)	
							47				
							52				
							90				
							137				
30	9D	24/6	30.0 - 32.0	6/8/7/12	15	21	67	113.8		30.0 Grey, moist, medium dense, fine to coarse GRAVEL, some clay, some fine to coarse sand -FILL-(GM)	
							82				
							77				
							139				
							121				
35	10D 10A	0/0 24/15	35.0 - 35.0 35.5 - 37.5	50/0" 63/43/41/23	84	118	71	108.8		35.0 Note: Split-spoon refusal at 35.0 ft; advanced roller bit to 35.5 ft and re-sample. Grey to dark gray, moist, very dense, fine to coarse SAND, some gravel, trace silt -FILL-(SW)	
							87				
							75	106.6		37.2	
							120				
							135				
40	11D	24/3	40.0 - 42.0	25/22/14/24	36	51	83			Grey, moist, dense, coarse SAND, some fine gravel, trace fine to medium sand -FILL-(SP)	
							114				
							62				
							81				
							132	99.8		44.0	
45	12D	24/17	45.0 - 47.0	9/24/27/24	51	72	HW			Grey, moist, hard, Clayey SILT, trace coarse sand, trace fine gravel -MARINE DEPOSIT-(ML)	
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-202
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	143.8	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53; NEBC #23 hammer	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-02-19/12-05-19	**Drilling Method:**	SSA 0-10'; NW/HW Drive to 72.5'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 513+64, 45L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	35.8 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	13D	24/23	50.0 - 52.0	18/19/26/27	45	63	HW			Grey, moist, hard, Silty CLAY to Clayey SILT, little medium sand, trace gravel -MARINE DEPOSIT-(CL/ML)	
55	14D	24/24	55.0 - 57.0	33/33/35/42	68	95				Grey, wet, hard, Silty CLAY to Clayey SILT, trace medium sand -MARINE DEPOSIT-(CL/ML)	
60	15D	24/24	60.0 - 62.0	22/37/55/68	92	129	27	83.8		60.0 Grey, wet, hard, fine Sandy SILT to Clayey SILT, little fine sand, trace coarse sand -MARINE DEPOSIT-(ML/CL)	
							44				
							52				
							49				
							62				
65	16D	24/21	65.0 - 67.0	25/37/43/50(1")	80	112	84	78.8		65.0 Dark grey, wet, hard, SILT, little fine to medium sand, trace fine gravel -GLACIAL TILL-(ML)	
	R1	60/52	66.7 - 71.7				NQ CORE	77.2		66.6 Grey, very dense, wet, fine to coarse SAND, little silt, little gravel -GLACIAL TILL-(SW-SM) R1: Recovered cobble. R1 Core Times (min:sec): 66.7-67.7' (1:58); 67.7-68.7' (1:11); 68.7-69.7' (1:24); 69.7-70.7' (1:09); 70.7-71.7 (1:54)	
70											
	17D	10/4	71.7 - 72.5	9/135(4.0")			100	72.1		71.7 Grey, very dense, wet, fine to coarse SAND, some gravel, trace silt, weathered rock fragments -GLACIAL TILL-(SW)	
	R2	49/51	72.5 - 76.6	RQD = 88%			NQ CORE	71.3		72.5 Top of Bedrock at El. 71.3 Hard, fresh, light grey, aphanitic to fine-grained SCHIST. Joints dipping moderate to high angles, moderately spaced, planar, rough,	
75											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Wilson Street Bridge Replacement Location: Brewer, Maine	Boring No.: BB-BWS-202 WIN: 18915.00

Driller:	New England Boring Contractors	Elevation (ft.)	143.8	Auger ID/OD:	--
Operator:	B. Enos	Datum:	NAVD 88	Sampler:	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	Rig Type:	Mobile B53; NEBC #23 hammer	Hammer Wt./Fall:	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-02-19/12-05-19	Drilling Method:	SSA 0-10'; NW/HW Drive to 72.5'	Core Barrel:	NQ-2.0 in. ID
Boring Location:	Sta. 513+64, 45L	Casing ID/OD:	HW-4.0 in. ID	Water Level*:	35.8 ft
Hammer Efficiency Factor:	0.842	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							NQ			tight to open, frequent quartz/calcite veins. Rock Quality=Good Recovery=104% -BREWER FORMATION- R2 Core Times (min:sec): 72.5-73.5' (1:10); 73.5-74.5' (3:42); 74.5-75.5' (3:22); 75.5-76.5' (6:21)	
	R3	61/61	76.6 - 81.7	RQD = 82%			CORE			Similar to R2 Rock Quality=Good -BREWER FORMATION- Recovery=100% R3 Core Times (min:sec): 76.6-77.7' (1:31); 77.7-78.7' (2:32); 78.7-79.7' (2:52); 79.7-80.7' (3:21); 80.7-81.7' (3:45)	
80											qp=2077 psi (79.7-80.2')
								62.1		81.7 **Bottom of Exploration at 81.7 feet below ground surface.**	
85											
90											
95											
100											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-203
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	111.2	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-11-2019; 12-11-2019	**Drilling Method:**	HW to 20.0'; NW to 44.5'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 515+19, 14L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	3/4/8/12	12	17	HW PUSH			Grey to brown, moist, medium dense, fine to coarse GRAVEL, some fine sand, trace silt, well graded -FILL-(GW)	
	2D	24/17	2.0 - 4.0	26/30/31/38	61	86				Similar to 1D, except very dense -FILL-(GW) Note: Granite gravel layer at 3.5 ft.	
								107.7		3.5 Brown, dry, hard, SILT, some gravel -FILL-(ML)	
	3D	24/23	4.0 - 6.0	12/12/15/17	27	38	44			Similar to above -FILL-(ML)	
5							-				
	4D	24/20	6.0 - 8.0	11/11/11/10	22	31	37			Brown, dry, hard, SILT -FILL-(ML)	
							-				
	5D	24/24	8.0 - 10.0	7/8/9/9	17	24	32	103.2		8.0 Grey, moist, very stiff, Silty CLAY, organics (roots) -FILL-(CL)	
							49	102.1		9.1 Grey, moist, very stiff, Silty CLAY -MARINE DEPOSIT-(CL)	
10							85				
							HW				
							69				
							62				
15	6D	24/19	15.0 - 17.0	14/17/21/26	38	53	74	96.2		15.0 Brown, wet, hard, Clayey SILT, trace medium to coarse sand, trace gravel -MARINE DEPOSIT-(ML)	
							84				
							67				
							59				
							71				
20	7D	24/16	20.0 - 22.0	7/16/18/18	34	48	78			Brown-grey, wet, hard, SILT, trace fine sand -MARINE DEPOSIT-(ML)	
							76				
							HW				
25											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Wilson Street Bridge Replacement Location: Brewer, Maine	Boring No.: BB-BWS-203 WIN: 18915.00

Driller:	New England Boring Contractors	Elevation (ft.):	111.2	Auger ID/OD:	--
Operator:	B. Enos	Datum:	NAVD 88	Sampler:	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	Rig Type:	Mobile B53	Hammer Wt./Fall:	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-11-2019; 12-11-2019	Drilling Method:	HW to 20.0'; NW to 44.5'	Core Barrel:	NQ-2.0 in. ID
Boring Location:	Sta. 515+19, 14L	Casing ID/OD:	HW-4.0 in. ID	Water Level*:	
Hammer Efficiency Factor:	0.842	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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WOR/C = Weight of Rods or Casing
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	8D	24/15	25.0 - 27.0	8/21/27/28	48	67	HW			Grey, wet, hard, Clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
30	9D	24/24	30.0 - 32.0	23/31/32/50	63	88				Grey, wet, hard, SILT, trace fine sand -MARINE DEPOSIT-(ML)	
35	10D	24/8	35.0 - 37.0	8/4/52/73	56	79				Grey, wet, hard, SILT, trace fine sand, trace gravel -MARINE DEPOSIT-(ML)	
								74.6		36.6 Grey, wet, very dense, Silty GRAVEL, trace clay, trace fine sand -GLACIAL TILL-(GM)	
40	11D	24/12	40.0 - 42.0	22/100(4")						Similar to 10D, except trace fine to coarse sand -GLACIAL TILL-(GM)	
								68.5		42.7 -WEATHERED BEDROCK-	
							NQ	66.7		44.5 Top of Bedrock at El. 66.7	
45	R1	60/60	45.0 - 50.0	RQD = 92%			CORE			Hard, fresh, grey, aphanitic to fine-grained SCHIST. Joints dipping at moderate to high angles, very close to moderately close, planar to undulating, rough, open. Rock Quality=Excellent Recovery=100% -BREWER FORMATION- R1 Core Times (min:sec): 45.0-46.0' (2:52); 46.0-47.0' (3:54); 47.0-48.0' (3:36); 48.0-49.0' (2:30); 49.0-50.0' (4:30)	qp=4448 psi (45.3-45.9')
50											

Remarks:

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: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-203
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	111.2	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-11-2019; 12-11-2019	**Drilling Method:**	HW to 20.0'; NW to 44.5'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 515+19, 14L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	R2	60/43	50.0 - 55.0	RQD = 72%			NQ CORE			Similar to R1, except very close to close Rock Quality=Fair Recovery=72% -BREWER FORMATION- R2 Core Times (min:sec): 50.0-51.0' (3:29); 51.0-52.0' (3:41); 52.0-53.0' (4:19); 53.0-54.0' (2:52); 54.0-55.0' (2:43)	
55	R3	60/60	55.0 - 60.0	RQD = 87%						Hard, fresh, grey, aphanitic to fine-grained PHYLLITE/SCHIST. Joints dipping at moderate to high angles, very close to close, undulating, rough, tight to open. Rock Quality=Good Recovery=100% -BREWER FORMATION- R3 Core Times (min:sec): 55.0-56.0' (3:07); 56.0-57.0' (3:22); 57.0-58.0' (4:53); 58.0-59.0' (3:11); 59.0-60.0' (2:58)	
60								51.2		60.0 **Bottom of Exploration at 60.0 feet below ground surface.**	
65											
70											
75											

Remarks:

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: Wilson Street Bridge Replacement Location: Brewer, Maine	Boring No.: BB-BWS-204 WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	136.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-5-2019; 12-6-2019	**Drilling Method:**	SSA to 10.0'; HW to 35.0'	**Core Barrel:**	--
Boring Location:	Sta. 516+07, 39L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	8.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	135.9		-BITUMINOUS CONCRETE- 0.4	
	1D	24/17	1.0 - 3.0	13/27/40/36	67	94				Brown to grey, dry, very dense, fine to medium SAND, some gravel, trace silt, trace coarse sand -FILL-(SP)	
	2D	24/24	3.0 - 5.0	49/65/47/38	112	157		133.3		3.0 Brown to grey, dry, very dense, fine to medium Silty SAND, trace coarse sand, trace fine gravel -FILL-(SM)	
5	3D	24/18	5.0 - 7.0	28/27/28/21	55	77		131.3		5.0 Brown to grey, dry, very dense, Gravelly fine to coarse SAND, trace silt -FILL-(SW-SM)	G#545433 A-1-a(0) SW-SM
	4D	24/13	7.0 - 9.0	39/22/25/32	47	66		129.3		7.0 Brown to grey, dry, very dense, fine to medium SAND, little gravel, trace silt, trace coarse sand -FILL-(SP)	
	5D	24/18	9.0 - 11.0	21/25/18/19	43	60		127.3		9.0 Brown to grey, dry, very dense, fine to medium Silty SAND, trace coarse sand, trace gravel -FILL-(SM)	
10							HW				
	6D	24/13	11.0 - 13.0	18/31/25/23	56	79		125.3		11.0 Brown to grey, dry, very dense, fine to medium SAND, some gravel, trace silt, trace coarse sand -FILL-(SP)	
	7D	24/19	13.0 - 15.0	50/34/31/49	65	91		123.3		13.0 Brown, dry, very dense, fine to coarse SAND, some fine to coarse gravel, trace silt -FILL-(SW-SM)	G#545437 A-1-b(0) SW-SM
15	8D	24/8	15.0 - 17.0	27/32/27/25	59	83	25	121.3		15.0 Brown to dark grey, wet, very dense, fine to coarse SAND, some fine to coarse gravel, trace silt -FILL-(SW)	G#545438 A-1-b(0)
							38				
	9D	24/14	17.0 - 19.0	19/15/38/39	53	74	63	119.3		17.0 Brown to grey, moist, very dense, fine to medium Silty SAND, little coarse sand, little gravel -FILL-(SM)	
							67				
	10D	24/7	19.0 - 21.0	15/23/21/11	44	62	48	117.3		19.0 Brown to dark grey, wet, very dense, Gravelly fine to coarse SAND, trace silt -FILL-(SW)	G#545440 A-1-a(1) SW
20							73				
	11D	24/12	21.0 - 23.0	13/11/11/10	22	31	62	115.3		21.0 Brown to brown-grey, moist, dense, Sandy SILT, little gravel -FILL-(ML)	G#545441 A-4(0) ML
							83				
	12D	24/7	23.0 - 25.0	5/4/4/4	8	11	27			Brown, moist, stiff, SILT, little fine to coarse sand, trace clay -FILL-(ML)	G#545442 A-4(0) ML
25							31				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-204
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	136.3	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-5-2019; 12-6-2019	**Drilling Method:**	SSA to 10.0'; HW to 35.0'	**Core Barrel:**	--
Boring Location:	Sta. 516+07, 39L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	8.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	13D	24/15	25.0 - 27.0	8/11/9/10	20	28	29			Brown, moist, very stiff, SILT, little fine to coarse sand, trace clay, trace gravel -FILL-(ML)	
							38	109.6		26.7	
							HW			Brown, moist, very stiff, SILT, trace to some fine to coarse sand -GLACIAL TILL-(ML)	
30								106.3		30.0	
	14D	24/9	30.0 - 32.0	20/73/58/31	131	184				Brown, moist, very dense, fine to coarse SAND, some gravel, little silt -GLACIAL TILL-(SW-SM)	
								104.5		31.8	
								104.3		Weathered Bedrock (layer in sample) 32.0	
35	15D	24/9	35.0 - 37.0	37/41/47/43	88	123				Brown, wet, very dense, fine to coarse SAND, some gravel, little silt -GLACIAL TILL-(SW-SM)	
								99.3		37.0	
										Bottom of Exploration at 37.0 feet below ground surface.	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement		**Boring No.:**	BB-BWS-205
Location: Brewer, Maine		**WIN:**	18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	135.7	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-10-2019; 12-10-2019	**Drilling Method:**	HW to 54.0'	**Core Barrel:**	--
Boring Location:	Sta. 516+30, 76L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	--
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.0 - 2.0	25/23/20/18	43	60	HW PUSH	135.2		-BITUMINOUS CONCRETE- 0.5 Brown, dry, very dense, fine to coarse SAND, some silt, trace gravel -FILL-(SM)	
	2D	24/16	2.0 - 4.0	12/10/29/28	39	55		133.7		2.0 Brown, dry, very dense, Sandy fine to coarse GRAVEL, trace silt -FILL-(GW-GM)	G#545432 A-1-a(1) GW-GM
	3D	24/19	4.0 - 6.0	14/16/18/17	34	48		131.7		4.0 Brown, dry, dense, fine to medium SAND, little silt, little gravel -FILL-(SP-SM)	
5											
	4D	24/17	6.0 - 8.0	19/9/7/3	16	22		129.7		6.0 Brown, dry, medium dense, fine to coarse SAND, some fine to coarse gravel, little silt -FILL-(SW-SM)	G#545434 A-1-b(0) SW-SM
	5D	24/10	8.0 - 10.0	8/43/13/18	56	79	36	127.7		8.0 Brown, moist, hard, SILT, little fine to medium sand, trace gravel and brown moist, very dense, fine to medium SAND, trace coarse sand, trace silt, trace gravel -FILL-(SM)	G#545435 A-2-4(0) SM
							49				
10	6D	24/8	10.0 - 12.0	10/7/7/9	14	20	72			Brown, wet, medium dense, fine to coarse SAND, some silt, some fine to coarse gravel -FILL-(SM)	G#545436 A-2-4(0) SM
	7D	24/11	12.0 - 14.0	6/14/25/14	39	55	43	123.7		12.0 Brown, moist, very dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW)	
							32				
	8D	24/0	14.0 - 16.0	4/5/3/5	8	11	34			No Recovery	
15							84				
	9D	24/13	16.0 - 18.0	9/11/21/15	32	45	47	119.7		16.0 Brown to grey, wet, hard, fine to medium Sandy SILT -FILL-(ML)	
							53				
	10D	24/15	18.0 - 20.0	8/14/8/4	22	31	64			Brown to grey, moist, hard, SILT, some fine to medium sand, trace gravel -FILL-(ML)	G#545439 A-4(0) ML
							82				
20	11D	24/15	20.0 - 22.0	5/7/8/15	15	21	HW			Brown to grey, wet, very stiff, SILT, some fine to medium sand, trace gravel -FILL-(ML)	
	12D	24/11	22.0 - 24.0	12/12/14/8	26	36				Brown, moist to wet, hard, SILT, little fine to medium sand -FILL-(ML)	
25	13D	24/0	24.0 - 26.0	5/7/15/9	22	31				No Recovery	

Remarks:

Note: Hole caved to 14.0 ft after removing casing, dry.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Wilson Street Bridge Replacement Location: Brewer, Maine	Boring No.: BB-BWS-205 WIN: 18915.00

Driller:	New England Boring Contractors	Elevation (ft.)	135.7	Auger ID/OD:	--
Operator:	B. Enos	Datum:	NAVD 88	Sampler:	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	Rig Type:	Mobile B53	Hammer Wt./Fall:	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-10-2019; 12-10-2019	Drilling Method:	HW to 54.0'	Core Barrel:	--
Boring Location:	Sta. 516+30, 76L	Casing ID/OD:	HW-4.0 in. ID	Water Level*:	--
Hammer Efficiency Factor:	0.842	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							HW				
	14D	24/24	26.0 - 28.0	12/9/25/10	34	48				Brown, wet to moist, hard, SILT, little fine to medium sand, trace gravel -FILL-(ML)	
	15D	24/21	28.0 - 30.0	11/11/22/17	33	46				Brown to rust-brown, moist, hard, SILT, little fine sand, little gravel -FILL-(ML)	
30	16D	24/12	30.0 - 32.0	14/37/18/27	55	77	31			Brown to grey, moist, hard, fine to medium Sandy SILT, trace coarse sand, trace gravel -FILL-(ML)	
							55				
	17D	24/16	32.0 - 34.0	15/25/41/22	66	93	58	103.7		32.0 Brown, moist, very dense, fine to coarse SAND, little silt, trace gravel -FILL-(SP-SM)	
							76				
35	18D	24/6	34.0 - 36.0	16/23/26/16	49	69	51	101.7		34.0 Grey, wet, very dense, fine to medium SAND, some gravel, little silt -FILL-(SP)	
							57				
	19D	24/17	36.0 - 38.0	38/23/50/50	73	102	72	99.7		36.0 Brown, moist, very dense, fine to medium Silty SAND, trace coarse sand, trace gravel -FILL-(SM)	
							59				
	20D	24/6	38.0 - 40.0	11/10/7/3	17	24	77	97.7		38.0 Grey, moist, very stiff, SILT, little fine to coarse sand, trace gravel -FILL-(ML)	
							151				
40	21D	24/0	40.0 - 42.0	5/5/9/8	14	20	101			No Recovery	
							96				
	22D	24/15	42.0 - 44.0	2/3/2/2	5	7	103			Grey, wet, medium stiff, SILT, some fine sand, little gravel -FILL-(ML)	
							52				
45	23D	24/19	44.0 - 46.0	16/17/15/21	32	45	92	91.7		44.0 Grey to dark grey, moist, dense, fine to medium Silty SAND, trace gravel -FILL-(SM)	
							132				
	24D	24/16	46.0 - 48.0	4/8/7/9	15	21	49	89.7 89.3		46.0 Note: Bricks and fine to coarse sand, trace gravel encountered from 46.0 to 46.4 ft. 46.4 Grey, moist, very stiff, Silty CLAY -MARINE DEPOSIT-(CL)	
							57				
	25D/A	24/24	48.0 - 50.0	6/6/11/18	17	24	59			Grey to brown mottled, moist, very stiff, Silty CLAY to Clayey SILT -MARINE DEPOSIT-(CL/ML)	
50							72				

Remarks:

Note: Hole caved to 14.0 ft after removing casing, dry.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Wilson Street Bridge Replacement Location: Brewer, Maine	Boring No.: BB-BWS-205 WIN: 18915.00

Driller:	New England Boring Contractors	Elevation (ft.)	135.7	Auger ID/OD:	--
Operator:	B. Enos	Datum:	NAVD 88	Sampler:	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	Rig Type:	Mobile B53	Hammer Wt./Fall:	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-10-2019; 12-10-2019	Drilling Method:	HW to 54.0'	Core Barrel:	--
Boring Location:	Sta. 516+30, 76L	Casing ID/OD:	HW-4.0 in. ID	Water Level*:	--
Hammer Efficiency Factor:	0.842	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	26D	24/24	50.0 - 52.0	8/11/8/8	19	27	HW			Grey to brown mottled, wet, very stiff, Silty CLAY -MARINE DEPOSIT-(CL)	
								81.7		54.0	
	27D	24/17	54.0 - 56.0	78/85/93/104	178	250				Grey to brown, wet, hard, SILT, some gravel, trace fine sand -GLACIAL TILL-(ML)	
55											
								80.2		55.5	
								79.7		-WEATHERED BEDROCK- 56.0	
										Bottom of Exploration at 56.0 feet below ground surface.	
60											
65											
70											
75											

Remarks:

Note: Hole caved to 14.0 ft after removing casing, dry.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Wilson Street Bridge Replacement			**Boring No.:**	BB-BWS-206
Location:	Brewer, Maine			**WIN:**	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	134.6	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-06-2019; 12-10-2019	**Drilling Method:**	HW drive to 79.0'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+96, 37L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	30.0 ft
Hammer Efficiency Factor: 0.842		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.0 - 2.0	40/85/27/20/18	112	157	HW PUSH	133.8		-BITUMINOUS CONCRETE- 0.8	
								132.6		Brown, dry, hard, fine to medium SAND, some silt, little gravel -FILL-(SM) 2.0	
	2D	24/18	2.0 - 4.0	18/7/21/20	28	39				Grey, dry, hard, SILT, some fine to coarse sand, some gravel -FILL-(ML)	
	3D	24/21	4.0 - 6.0	20/25/20/26	45	63				Grey, dry, hard, SILT, some gravel, little fine to coarse sand -FILL-(ML)	
5											
							62				
							63				
							71				
	4D	24/24	9.0 - 11.0	15/12/12/11	24	34	69			Brown, dry, hard, SILT, little fine to coarse sand, little gravel -FILL-(ML)	
10							67				
							81				
							84				
							91				
	5D	24/16	14.0 - 16.0	18/19/19/20	38	53	HW			Brown, moist, hard, SILT, trace fine sand -FILL-(ML)	
15											
							58				
							62				
							63				
	6D	24/16	19.0 - 21.0	13/19/23/38	42	59	51			Brown to rust-brown, wet, hard, SILT, little fine sand, trace coarse sand -FILL-(ML)	
20							56				
							63				
							68				
							81				
	7D	24/13	24.0 - 26.0	17/19/17/21	36	51	117			Brown, wet, hard, Clayey SILT, little fine sand -FILL-(ML)	
25											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Wilson Street Bridge Replacement **Location:** Brewer, Maine	**Boring No.:** BB-BWS-206 **WIN:** 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	134.6	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-06-2019; 12-10-2019	**Drilling Method:**	HW drive to 79.0'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+96, 37L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	30.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							132				
							303				
							HW				
								105.9		28.7	
	8D	12/5	29.0 - 30.0	63/82						Brown to dark grey, wet, very dense, Sandy GRAVEL, little silt -FILL-(GW-GM) Note: Split-spoon refusal at 30.0 ft.	
30											
							63				
								100.6		34.0	
	9D	24/9	34.0 - 36.0	10/9/6/7	15	21	69			Brown, moist, very stiff, fine Sandy SILT, little coarse sand, trace fine gravel -FILL-(ML)	
35							68				
							74				
							51				
							83	95.6		39.0	
	10D	24/7	39.0 - 41.0	16/16/47/49	63	88	72			Brown to grey, wet, very dense, fine to coarse Sandy GRAVEL, little silt -FILL-(GW-GM)	
40							80				
							83				
							85				
							88				
	11D	24/18	44.0 - 46.0	17/12/37/19	49	69	96			Grey, wet, very dense, fine to coarse Sandy GRAVEL, trace fine sand, trace silt -FILL-(GW-GM)	
45							60				
							61				
							68				
							71	85.4		49.2	
50	12D	24/24	49.0 - 51.0	5/6/8/9	14	20	81			Brown to grey, wet, very stiff, Silty CLAY, trace gravel	

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-206
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	134.6	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-06-2019; 12-10-2019	**Drilling Method:**	HW drive to 79.0'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+96, 37L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	30.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							HW			-MARINE DEPOSIT-(CL)	
								80.6		54.0	
	13D	24/18	54.0 - 56.0	29/38/37/21	75	105	65			Brown, moist, hard, fine Sandy SILT, trace gravel -GLACIAL TILL-(ML)	
55							64				
							74				
							81				
							71				
								75.6		59.0	
	14D	24/18	59.0 - 61.0	9/11/16/11	27	38	70			Grey, moist, hard, Silty CLAY, trace fine sand, trace gravel, well bonded -GLACIAL TILL-(CL)	
60							74				
							122				
							77				
							68				
								70.6		64.0	
	15D	24/10	64.0 - 66.0	18/15/18/21	33	46	71			Grey, moist, hard, SILT, little medium to coarse sand, trace fine sand, trace gravel, well bonded -GLACIAL TILL-(ML)	
65							93				
							97				
							169				
							204				
	16D	24/9	69.0 - 71.0	65/34/53/25	87	122	221			Grey, wet, hard, Sandy SILT, little medium to coarse sand, little gravel, trace clay, well bonded -GLACIAL TILL-(ML)	
70							122				
							144				
							151				
							201				
	17D	24/22	74.0 - 76.0	17/23/43/39	66	93	209			Grey to brown, wet, hard, SILT, little fine to coarse sand, little gravel, well bonded	
75											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Wilson Street Bridge Replacement **Location:** Brewer, Maine	**Boring No.:** BB-BWS-206 **WIN:** 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	134.6	**Auger ID/OD:**	--
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	J. Fletcher	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	12-06-2019; 12-10-2019	**Drilling Method:**	HW drive to 79.0'	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 516+96, 37L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	30.0 ft
Hammer Efficiency Factor:	0.842	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							HW			-GLACIAL TILL-(ML)	
	R1	60/60	79.0 - 84.0	RQD = 100%			NQ CORE	55.6		79.0 Top of Bedrock at El. 55.6 Hard, fresh, grey, aphanitic to fine-grained SCHIST. Joints dipping at moderate to high angles, moderately close spacing, planar, rough, slickenside texture, quartz vein at 81.2 ft. Rock Quality=Excellent Recovery=100% -BREWER FORMATION- R1 Core Times (min:sec): 79.0-80.0' (2:46); 80.0-81.0' (3:26); 81.0-82.0' (2:59); 82.0-83.0' (3:00); 83.0-84.0' (2:31)	qp=14729 psi (79.8-80.5')
80											
	R2	60/60	84.0 - 89.0	RQD = 97%						Similar to R1. Rock Quality=Excellent Recovery=100% -BREWER FORMATION- R2 Core Times (min:sec): 84.0-85.0' (2:24); 85.0-86.0' (2:59); 86.0-87.0' (2:05); 87.0-88.0' (2:36); 88.0-89.0' (2:48)	
85											
								45.6		89.0 **Bottom of Exploration at 89.0 feet below ground surface.**	
90											
95											
100											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement		**Boring No.:**	BB-BWS-301
Location: Brewer, Maine		**WIN:**	18915.00

Driller:	S.W. Cole Engineering, Inc.	**Elevation (ft.)**	100.5	**Auger ID/OD:**	--
Operator:	K. Hascom	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	1-30-2020/1-30-2020	**Drilling Method:**	SSA to 10'; HW 10 to 25'	**Core Barrel:**	--
Boring Location:	Sta. 525+62.1, 84.9L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	12.0 ft
Hammer Efficiency Factor:	0.977	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.0 - 2.0	2/3/6/4	9	15	SSA			Note: Frozen at ground surface. Brown, dry, stiff, SILT, some fine sand, trace gravel, roots -FILL-(ML)	
5	2D	24/18	5.0 - 7.0	6/6/6/6	12	20				Brown, moist, very stiff, SILT, little fine sand to coarse sand, trace gravel -FILL-(ML)	G#545430 A-4(0) ML
10	3D	24/14	10.0 - 12.0	1/3/5/4	8	13	32			Brown, moist, stiff, SILT, some fine to coarse sand, little fine to coarse gravel -FILL-(ML)	G#545431 A-4(0) ML
							55	88.8 88.5		11.7 Brown, fine to coarse SAND -FILL-(SM)	
	4D	24/22	12.0 - 14.0	3/3/3/3	6	10	46			12.0	
							48	86.8		Note: Water encountered at 12.0 ft. Brown to grey, wet, medium stiff, SILT, some fine to coarse sand, little gravel -FILL-(ML)	
	1U	24/24	14.0 - 16.0				53			13.7 Note: 1U - Little fine to medium sand within CLAY matrix observed in top and bottom of tube sample.	
15							50			Note: 2U - Grey, wet, Silty CLAY, trace fine to medium sand in top of tube sample. Fine to coarse SAND, little gravel in bottom of tube sample. 0.2 ft void at bottom inside tube. -MARINE DEPOSIT-(CL)	
	2U	24/19	16.0 - 18.0				66				
							57				
							87				
	5D	24/21	19.0 - 21.0	2/3/7/10	10	16	69	81.5		19.0 Brown-grey mottled, moist, stiff, Silty CLAY to Clayey SILT, trace organics -MARINE DEPOSIT-(CL/ML) Note: Attempt 55x110 mm vane shear test at 19.0 to 20.0 ft. Vane refusal at 19.0 ft.	G#545427 LL=36, PL=21 PI=15, WC=24
20							137				
							183				
							209				
							193				
	6D	24/24	24.0 - 26.0	2/3/4/4	7	11	OPEN			Similar to 5D above	G#545428 LL=35, PL=20
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wilson Street Bridge Replacement
Location: Brewer, Maine

Boring No.: BB-BWS-301
WIN: 18915.00

Driller:	S.W. Cole Engineering, Inc.	**Elevation (ft.)**	100.5	**Auger ID/OD:**	--
Operator:	K. Hascom	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Diedrich D50	**Hammer Wt./Fall:**	HW-140#/30 in.; SS-140#/30
Date Start/Finish:	1-30-2020/1-30-2020	**Drilling Method:**	SSA to 10'; HW 10 to 25'	**Core Barrel:**	--
Boring Location:	Sta. 525+62.1, 84.9L	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	12.0 ft
Hammer Efficiency Factor:	0.977	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							OPEN				PI=15, WC=29
								71.5		Note: Grey, Silty CLAY in wash water at 29.0 ft. 29.0	
	3U	24/24	29.0 - 31.0							Note: 3U - Grey, wet, medium stiff, Silty CLAY.	G#545426 LL=35, PL=19 PI=16, WC=32
30											
	V1		31.0 - 32.0	Su=840/185 psf						55x110 mm vane raw torque readings: V1: 18/4 ft lbs	
	V2		32.0 - 33.0	Su=980/325 psf						V2: 21/7 ft lbs	
	7D	24/24	34.0 - 36.0	push thru vane						Grey, wet, medium stiff, Silty CLAY (CL) -MARINE DEPOSIT-(CL)	G#545429 LL=31, PL=19 PI=12, WC=33
35	V3		34.0 - 35.0	Su=790/235 psf						55x110 mm vane raw torque readings: V3: 17/5 ft lbs	
	V4		35.0 - 35.8	Su=650/235 psf				64.7		V4: 14/5 ft lbs Note: Vane refusal at 35.8 ft. 35.8	
										Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, loosely to moderately bonded -GLACIAL TILL-(SM)	
	8D	22/12	39.0 - 40.8	13/12/20/50(4")	32	52				Grey, wet, very dense, fine to medium SAND, some gravel, little silt, well bonded -GLACIAL TILL-(SM)	
40								59.7		40.8	
								59.4		Note: Probable weathered bedrock based on drill action. -PROBABLE WEATHERED BEDROCK- 41.1	
								58.4		Note: Dark grey phyllite rock chips in wash water. Top of probable bedrock at El. 59.4. Advanced roller bit to 42.1 ft. 42.1	
										Bottom of Exploration at 42.1 feet below ground surface.	
45											
50											

Remarks:

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

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

ROCK CORE PHOTOGRAPHS
I-395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER/EDDINGTON, MAINE

Top Row: BB-BWS-102, Run No. R1 41.0 (left) to 42.3 (middle), Run No. R2 42.3 (middle) to 45.5 (right)
Top Middle Row: BB-BWS-102, Run No. R3 45.5 (left) to 49.8 (middle), Run No. R4 49.8 (middle) to 51.3 (right)
Bottom Middle Row: BB-BFB-101, Run No. R1 42.0 (left) to 45.0 (middle), Run No. R2 45.0 (middle) to 50.0 (right)
Bottom Row: BB-BFB-101, Run No. R2 continued 45.0 (left) to 50.0 (middle), Run No. R3 50.0 (middle) to 52.0 (right)

ROCK CORE PHOTOGRAPHS
I-395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER/EDDINGTON, MAINE

Top Row: BB-BWS-103, Run No. R1 35.0 (left) to 36.2, Run No. R2 36.2 to 37.3, Run No. R3 37.3 to 38.0, Run No. R4 38.0 to 39.0, Run No. R5 39.0 to 40.1 (right)
Top Middle Row: BB-BWS-103, Run No. R6 40.1 (left) to 41.0, Run No. R7 41.0 to 42.2, Run No. R8 42.2 to 43.3, Run No. R9 43.3 to 44.4, Run No. R10 44.4 to 45.0 (right)

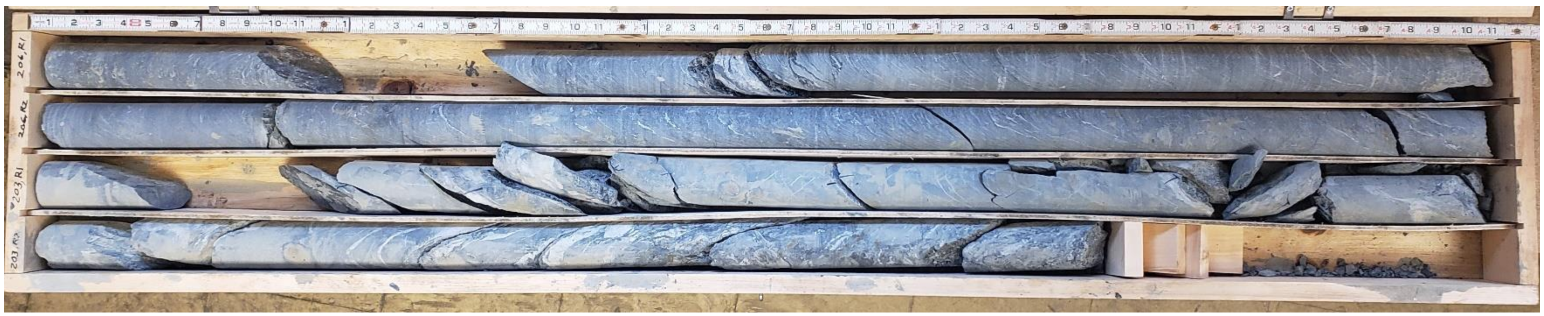
ROCK CORE PHOTOGRAPHS
I-395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER/EDDINGTON, MAINE

Top Row: BB-BWS-104, Run No. R1 38.0 (left) to 43.0 (right)
Top Middle Row: BB-BWS-104, Run No. R2 43.0 (left) to 45.9 (middle), Run No. R3 45.9 (middle) to 48.0 (right)

ROCK CORE PHOTOGRAPHS
I-395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER/EDDINGTON, MAINE

Top Row: BB-BWS-202, Run No. R1 66.7 (left) to 71.7 (right)
Top Middle Row: BB-BWS-202, Run No. R2 72.5 (left) to 76.6 (right)
Bottom Middle Row: BB-BWS-202, Run No. R3 76.6 (left) to 81.7 (right)
Bottom Row: BB-BWS-203, Run No. R3 55.0 (left) to 60.0 (right)

ROCK CORE PHOTOGRAPHS
I-395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER/EDDINGTON, MAINE



Top Row: BB-BWS-206, Run No. R1 79.0 (left) to 84.0 (right)
Top Middle Row: BB-BWS-206, Run No. R2 84.0 (left) to 89.0 (right)
Bottom Middle Row: BB-BWS-203, Run No. R1 45.0 (left) to 50.0 (right)
Bottom Row: BB-BWS-203, Run No. R2 50.0 (left) to 55.0 (right)

APPENDIX B

Observation Well Installation and Groundwater Monitoring Reports

HALEY ALDRICH

OBSERVATION WELL INSTALLATION REPORT

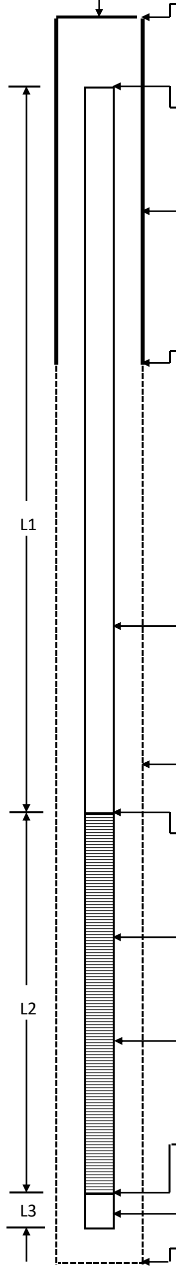
Well No. BB-BWS-102(OW)
Boring No. BB-BWS-102(OW)

PROJECT	Route 9/I-395 Connector	**H&A FILE NO.**	132076-002
LOCATION	Brewer, Maine/Eddington, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	N. Klausmeyer
CONTRACTOR	Northern Test Borings Inc.	**DATE INSTALLED**	7/12/2018
DRILLER	M. Nadeau	**WATER LEVEL**	2.9 ft. (depth below gs)

Ground El. 112.2 ft   **Location** SEE PLAN   ☑ **Guard Pipe**
El. Datum NAVD 88   ☐ **Roadway Box**

SOIL/ROCK CONDITIONS	BOREHOLE BACKFILL
	BENTONITE
	2.0 ft
-FILL-	
10.0 ft	
-GLACIAL TILL-	FILTER SAND
	~30.0 ft
	BOREHOLE CUTTINGS
39.8 ft	39.8 ft
-BEDROCK-	BENTONITE
51.3	51.3

(Bottom of Exploration)
(Numbers refer to depth from ground surface in feet)



Type of protective cover	Locking Cap	
Height of top of guard pipe above ground surface	2.6	ft
Height of top of riser pipe above ground surface	2.3	ft
Type of protective casing:	Steel Guard Pipe	
Length	5.0	ft
Inside Diameter	4.0	in
Depth of bottom of guard pipe	2.4	ft

Type of Seals	Top of Seal (ft)	Thickness (ft)
Bentonite Seal	0.0	2.0

Type of riser pipe:	Sch. 40 PVC	
Inside diameter of riser pipe	2.0	in
Type of backfill around riser	Filter Sand	
Diameter of borehole	4.0	in
Depth to top of well screen	2.7	ft
Type of screen	Sch. 40 PVC	
Screen gauge or size of openings	0.010	in
Diameter of screen	2.0	in
Type of backfill around screen	No. 2 Filter Sand	
Depth of bottom of well screen	17.7	ft
Bottom of Silt trap	18.0	ft
Depth of bottom of borehole	18.0	ft

(Not to Scale)

5.0 ft + 15.0 ft + 0.3 ft = 20.3 ft
Riser Pay Length (L1) + Length of screen (L2) + Length of silt trap (L3) = Pay length

COMMENTS:

GROUNDWATER MONITORING REPORT

HALEY ALDRICH

OW/PZ NUMBER
BB-BWS-102(OW)
Page 1 of 1

PROJECT	Route 9/I-395 Connector	**H&A FILE NO.**	132076-005
LOCATION	Brewer, Maine / Eddington, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	Klausmeyer/Hollauer/Fletcher
CONTRACTOR	Northern Test Borings Inc.	**DATE**	4/9/2020

ELEVATION OF REFERENCE POINT (ft) 112.2 **REFERENCE POINT:** **Ground Surface** ☑ **PVC** ☐ **Other** ☐

Date	Time	Elapsed Time (days)	Depth of Water from Reference Point (ft)	Elevation of Water (ft)	Remarks	Read By
7/13/2018	11:20 AM	0	-2.9	109.3	Initial Reading - 70s & Sunny	NLK
8/10/2018	1:15 PM	28	-4.1	108.2	80s & Sunny	NLK
8/13/2018	11:45 PM	31	-4.5	107.7	70s & Overcast	NLK
9/4/2018	2:49 PM	53	-5.6	106.7	80s & Sunny	NLK
9/10/2018	11:45 AM	59	-6.0	106.2	60s & Sunny	NLK
10/16/2018	7:30 AM	95	-3.4	108.8	40s & Sunny	HH
12/20/2018	11:15 AM	160	-2.4	109.9	20s & Sunny w/ 4" of snow	NLK
10/30/2019	11:23 AM	314	-1.9	110.4	50s & Cloudy	HH
12/12/2019	1:44 PM	517	-2.3	109.9	20s & Sunny	JKF
4/4/2020	8:59 AM	631	-1.5	110.7	40s & Sunny	HH

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OBSERVATION WELL INSTALLATION REPORT

HALEY ALDRICH

Well No. BB-BWS-104(OW)
Boring No. BB-BWS-104(OW)

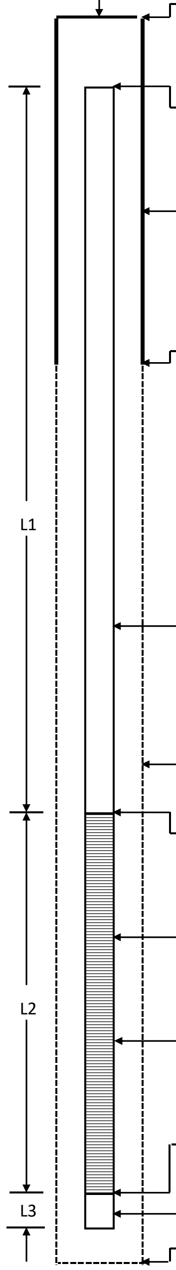
PROJECT	Route 9/I-395 Connector	**H&A FILE NO.**	132076-002
LOCATION	Brewer, Maine/Eddington, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	N. Klausmeyer
CONTRACTOR	Northern Test Borings Inc.	**DATE INSTALLED**	7/11/2018
DRILLER	M. Nadeau	**WATER LEVEL**	0.0 ft. (at ground surface)

Ground El. 100.9 ft
El. Datum NAVD 88
Location SEE PLAN

☑ **Guard Pipe**
☐ **Roadway Box**

SOIL/ROCK CONDITIONS	BOREHOLE BACKFILL
-FILL-	BENTONITE
	2.0 ft
6.0 ft	
-MARINE DEPOSIT-	FILTER SAND
25.0 ft	
	~27.0 ft
-GLACIAL TILL-	BOREHOLE CUTTINGS
36.8 ft	36.8 ft
BEDROCK-	BENTONITE
48.0	48.0

(Bottom of Exploration)
(Numbers refer to depth from ground surface in feet)



(Not to Scale)

Type of protective cover	Locking Cap	
Height of top of guard pipe above ground surface	2.7	ft
Height of top of riser pipe above ground surface	2.3	ft
Type of protective casing:	Steel Guard Pipe	
Length	5.0	ft
Inside Diameter	4.0	in
Depth of bottom of guard pipe	2.3	ft

Type of Seals	Top of Seal (ft)	Thickness (ft)
Bentonite Seal	0.0	2.0

Type of riser pipe:	Sch. 40 PVC	
Inside diameter of riser pipe	2.0	in
Type of backfill around riser	Filter Sand	
Diameter of borehole	4.0	in
Depth to top of well screen	2.7	ft
Type of screen	Sch. 40 PVC	
Screen gauge or size of openings	0.010	in
Diameter of screen	2.0	in
Type of backfill around screen	No. 2 Filter Sand	
Depth of bottom of well screen	17.7	ft
Bottom of Silt trap	18.0	ft
Depth of bottom of borehole	48.0	ft

5.0 ft + 15.0 ft + 0.3 ft = 20.3 ft
Riser Pay Length (L1) + Length of screen (L2) + Length of silt trap (L3) = Pay length

COMMENTS:

GROUNDWATER MONITORING REPORT

OW/PZ NUMBER: BB-BWS-104(OW)

Page 1 of 1

PROJECT	Route 9/I-395 Connector	**H&A FILE NO.**	132076-005
LOCATION	Brewer, Maine / Eddington, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	Klausmeyer/Hollauer/Fletcher
CONTRACTOR	Northern Test Borings Inc.	**DATE**	4/9/2020

ELEVATION OF REFERENCE POINT (ft) 100.9 **REFERENCE POINT:** Ground Surface ☑ PVC ☐ Other ☐

Date	Time	Elapsed Time (days)	Depth of Water from Reference Point (ft)	Elevation of Water (ft)	Remarks	Read By
7/11/2018	9:15 AM	0	0.0	100.9	Initial Reading - 80s & Sunny	NLK
7/13/2018	11:40 AM	2	0.0	100.9	70s & Sunny	NLK
8/10/2018	1:10 PM	30	0.0	100.9	80s & Sunny	NLK
8/13/2018	11:40 PM	33	0.0	100.9	70s & Overcast	NLK
9/4/2018	3:01 PM	55	-0.6	100.3	80s & Sunny	NLK
9/10/2018	11:50 AM	61	-1.1	99.9	60s & Cloudy	NLK
10/16/2018	7:40 AM	97	-0.4	100.5	40s & Sunny	HH
12/20/2018	11:20 AM	162	+0.5	101.4	20s & Sunny w/ 4" of snow - water was frozen above ground surface	NLK
10/30/2019	11:02 AM	314	-1.4	99.6	50s & Cloudy	HH
12/12/2019	1:05 PM	519	+1.0	101.9	20s & Sunny, water was frozen above ground surface	JKF
4/4/2020	8:50 AM	633	+1.3	102.2	40s & Sunny, water was above ground surface	HH

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APPENDIX C

Laboratory Test Results

GeoTesting
EXPRESS

Technologies to manage risk for infrastructure

Boston
Atlanta
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Los Angeles
New York
www.geotesting.com

Transmittal

TO:

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

DATE: 3/31/2020	GTX NO: 311345
RE: Rte 9/I-395 Conn. - Wilson St Bridge	

COPIES	DATE	DESCRIPTION
	3/31/2020	**March 2020 Laboratory Test Report**

REMARKS:

SIGNED: [signature]
Sarah Delaney, Assistant Laboratory Manager

CC:

APPROVED BY: [signature]
Jonathan Campbell, Laboratory Manager

March 31, 2020

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

RE: Rte 9/I-395 Conn. - Wilson St Bridge, Brewer & Eddington, ME (GTX-311345)

Dear Mr. Bryan Steinert:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received 20 samples from you on 2/14/2020. These samples were labeled as follows:

Boring Number	Sample Number	Depth
BB-BWS-202	R3	79.7-8.02 ft
BB-BWS-203	R1	45.3-45.9 ft
BB-BWS-204	10D	19-21 ft
BB-BWS-204	11D	21-23 ft
BB-BWS-204	13D	25-27 ft
BB-BWS-204	3D	5-7 ft
BB-BWS-204	7D	13-15 ft
BB-BWS-204	8D	15-17 ft
BB-BWS-205	10D	18-20 ft
BB-BWS-205	2D	2-4 ft
BB-BWS-205	4D	6-8 ft
BB-BWS-205	5D	8-10 ft
BB-BWS-205	6D	10-12 ft
BB-BWS-206	R1	79.8-80.5 ft
BB-BWS-301	2D	5-7 ft
BB-BWS-301	3D	10-12 ft
BB-BWS-301	3U	29-31 ft
BB-BWS-301	5D	19-21 ft
BB-BWS-301	6D	24-26 ft
BB-BWS-301	7D	34-36 ft

GTX performed the following tests on these samples:

1 ASTM D2435 - Incremental Consolidation
13 ASTM D422 - Grain Size Analysis - Sieve Only
4 ASTM D4318 - Atterberg Limits
1 ASTM D4767 - CU Triaxial Shear Test

3 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,

Sarah Delaney
Assistant Laboratory Manager

GeoTesting EXPRESS

Technologies to manage risk for infrastructure

Boston
Atlanta
Chicago
Los Angeles
New York
www.geotesting.com

Geotechnical Test Report **3/31/2020**

GTX-311345

Rte 9/I-395 Conn. - Wilson St Bridge

Brewer & Eddington, ME

Client Project No.: 132076-005

Prepared for:

Haley & Aldrich, Inc.

GeoTesting EXPRESS

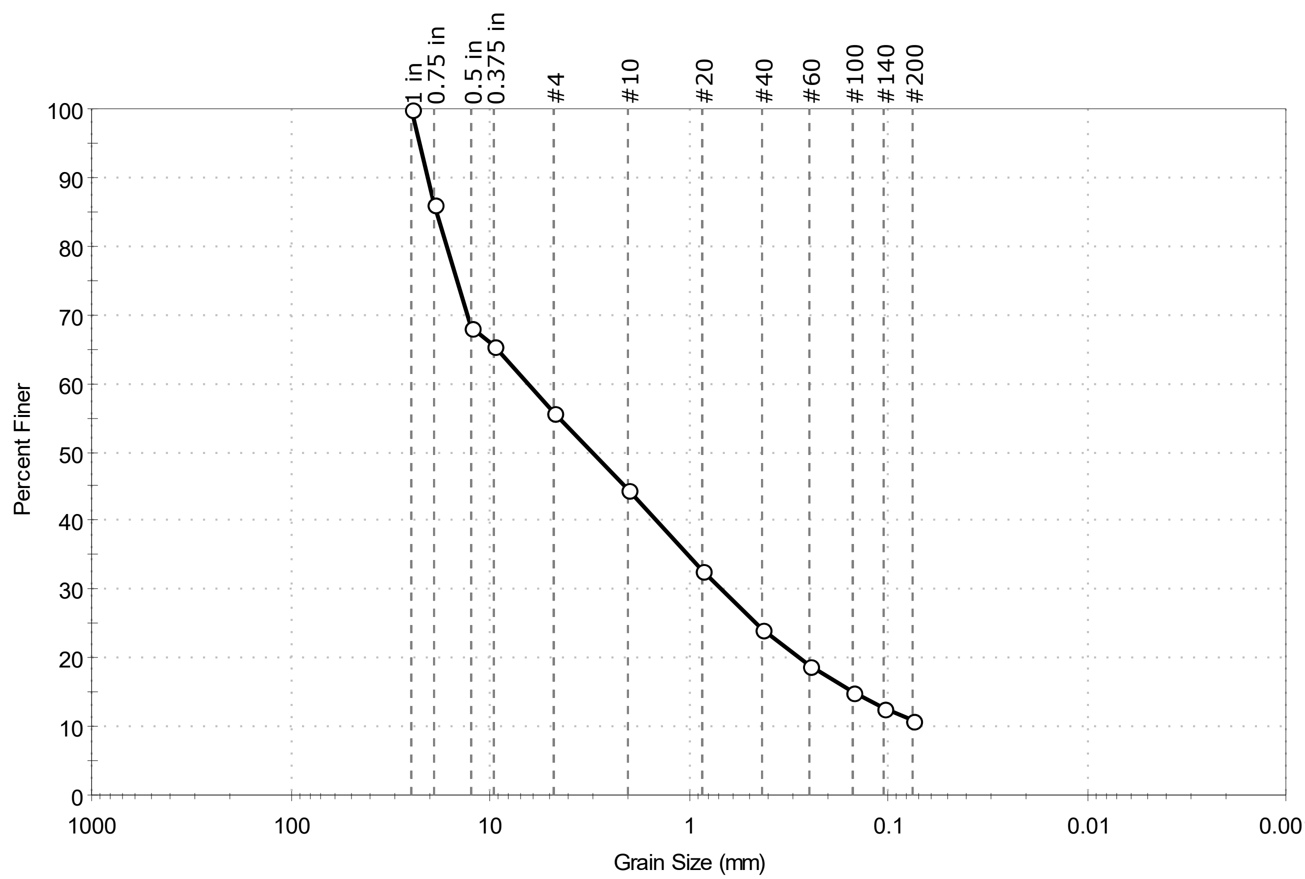
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-204	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 03/02/20	Checked By:	emm
Depth : 5-7 ft	Test Id: 545433		

Test Comment: ---

Visual Description: Mosit, light olive brown sand with silt and gravel

Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	44.2	44.8	11.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	86		
0.5 in	12.50	68		
0.375 in	9.50	65		
#4	4.75	56		
#10	2.00	45		
#20	0.85	33		
#40	0.42	24		
#60	0.25	19		
#100	0.15	15		
#140	0.11	13		
#200	0.075	11		

Coefficients

D_{85} =18.4713 mm | D_{30} =0.6818 mm

D_{60} =6.4237 mm | D_{15} =0.1493 mm

D_{50} =3.0381 mm | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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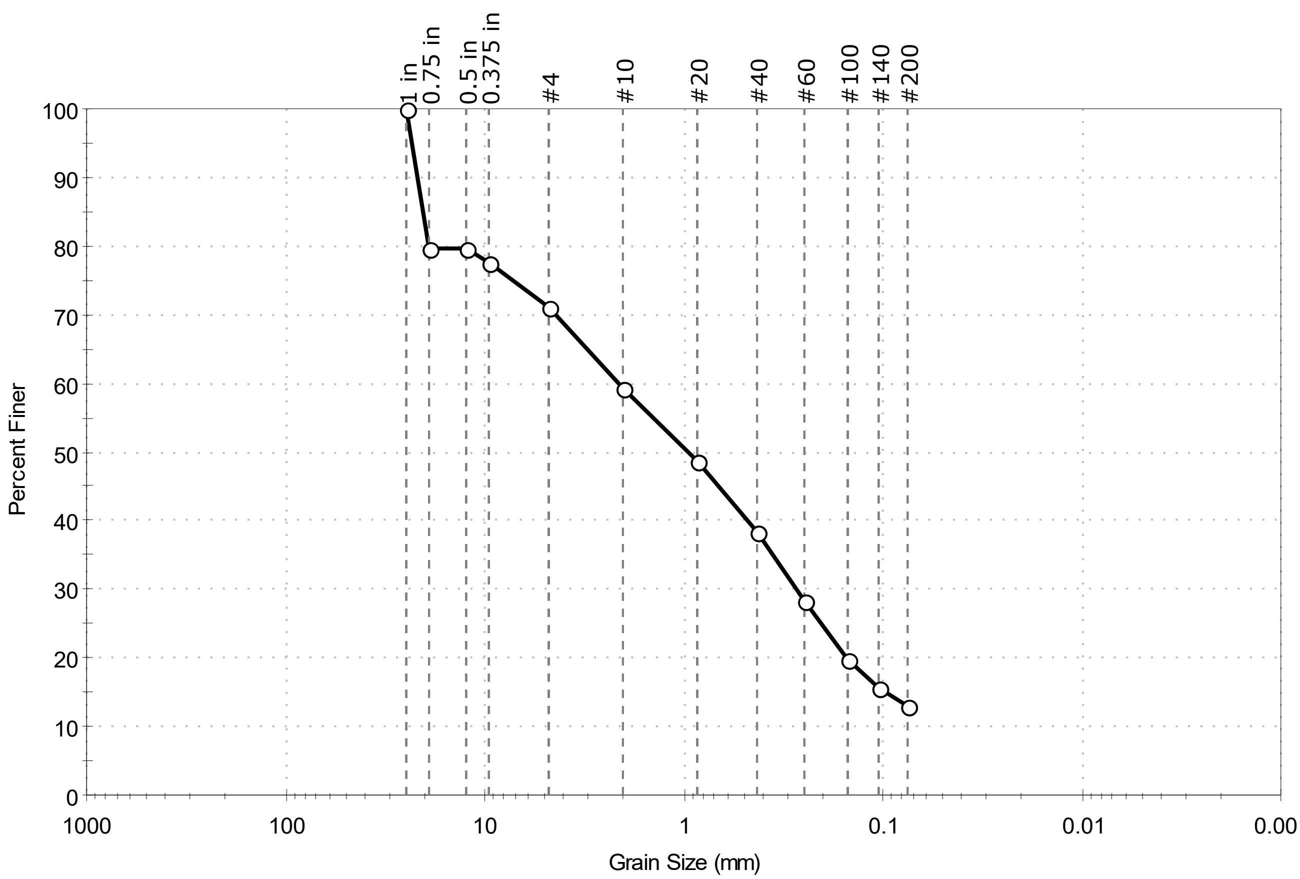
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-204	Sample Type: jar	Tested By:	ckg
Sample ID: 7D	Test Date: 03/02/20	Checked By:	emm
Depth : 13-15 ft	Test Id: 545437		

Test Comment: ---
Visual Description: Moist, light olive brown silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	28.9	58.1	13.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	80		
0.5 in	12.50	80		
0.375 in	9.50	78		
#4	4.75	71		
#10	2.00	59		
#20	0.85	49		
#40	0.42	38		
#60	0.25	28		
#100	0.15	20		
#140	0.11	16		
#200	0.075	13		

Coefficients

D_{85} =20.3980 mm　D_{30} =0.2738 mm
D_{60} =2.1200 mm　D_{15} =0.0970 mm
D_{50} =0.9483 mm　D_{10} =N/A
C_u =N/A　C_c =N/A

Classification

ASTM　N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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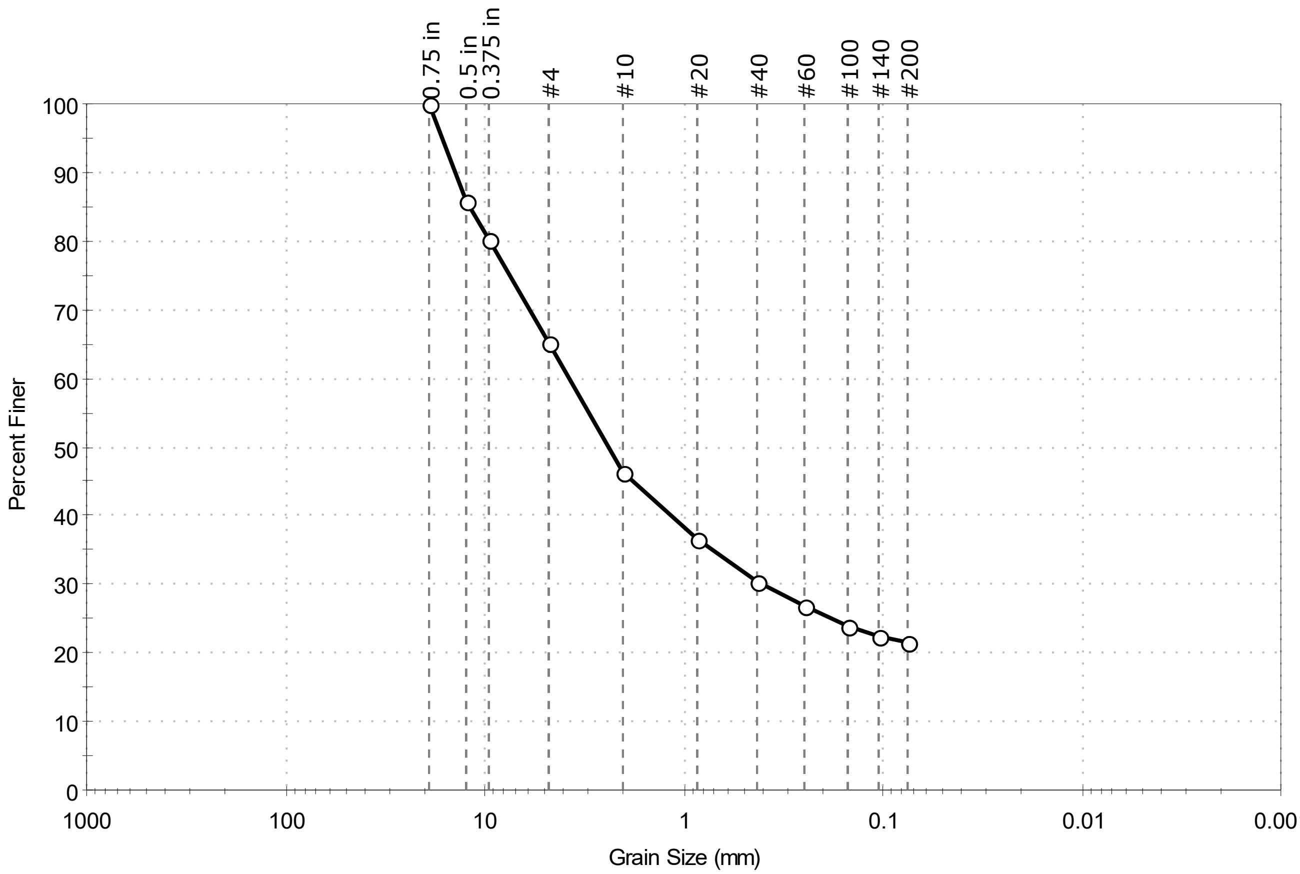
GeoTesting EXPRESS

Client: Haley & Aldrich, Inc.			
Project: Rte 9/I-395 Conn. - Wilson St Bridge			
Location: Brewer & Eddington, ME		Project No:	GTX-311345
Boring ID: BB-BWS-204	Sample Type: jar	Tested By:	ckg
Sample ID: 8D	Test Date: 03/02/20	Checked By:	emm
Depth : 15-17 ft	Test Id: 545438		

Test Comment: ---
Visual Description: Moist, dark grayish brown silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.8	43.6	21.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	86		
0.375 in	9.50	80		
#4	4.75	65		
#10	2.00	46		
#20	0.85	37		
#40	0.42	30		
#60	0.25	27		
#100	0.15	24		
#140	0.11	22		
#200	0.075	22		

Coefficients

D_{85} =11.9526 mm | D_{30} =0.4038 mm
D_{60} =3.7434 mm | D_{15} =N/A
D_{50} =2.3618 mm | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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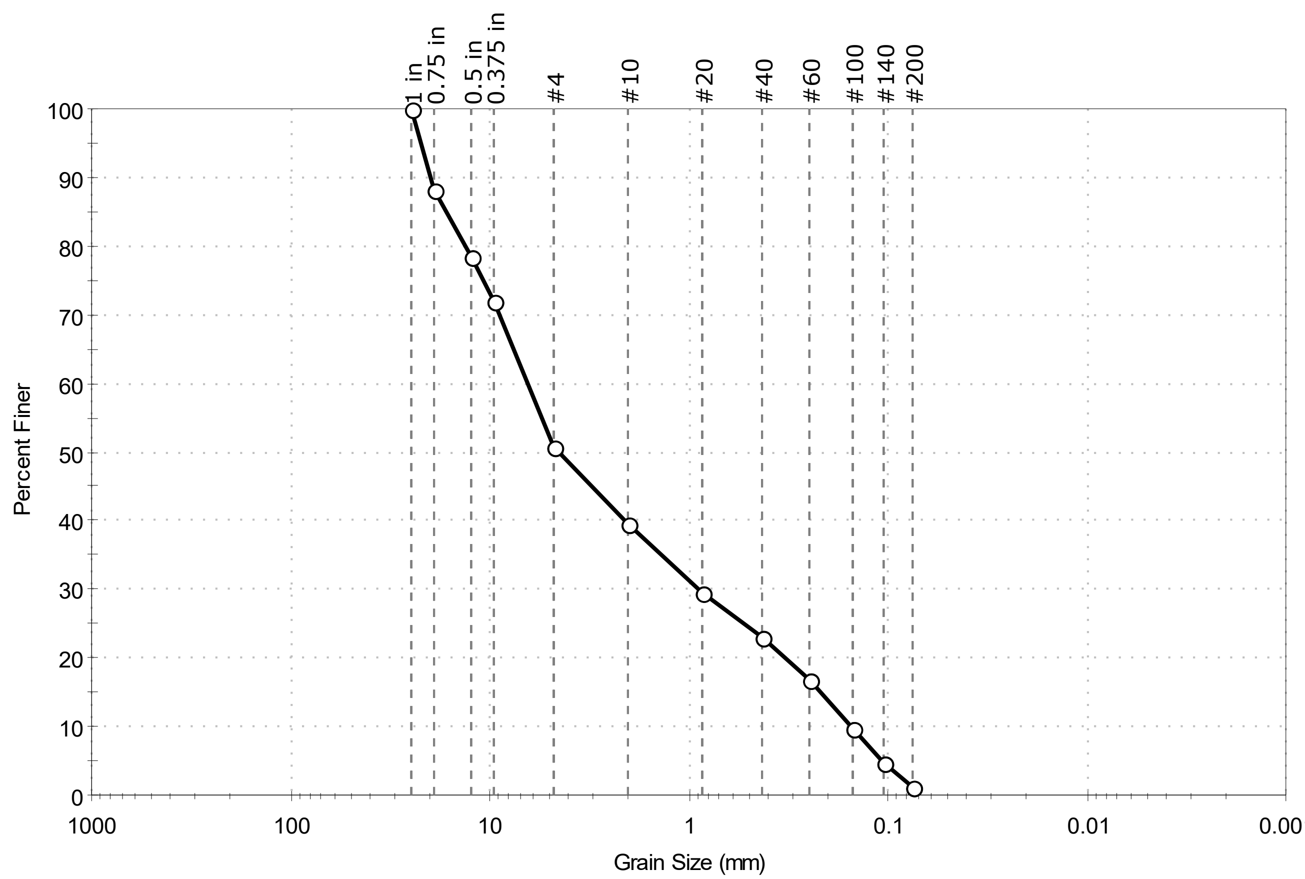
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-204	Sample Type: jar	Tested By:	ckg
Sample ID: 10D	Test Date: 03/02/20	Checked By:	emm
Depth : 19-21 ft	Test Id: 545440		
Test Comment:	---		
Visual Description:	Moist, olive brown sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	49.3	49.6	1.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	88		
0.5 in	12.50	78		
0.375 in	9.50	72		
#4	4.75	51		
#10	2.00	40		
#20	0.85	30		
#40	0.42	23		
#60	0.25	17		
#100	0.15	10		
#140	0.11	5		
#200	0.075	1.1		

Coefficients

D_{85} =16.6059 mm D_{30} =0.8795 mm

D_{60} =6.4225 mm D_{15} =0.2206 mm

D_{50} =4.4790 mm D_{10} =0.1544 mm

C_u =41.597 C_c =0.780

Classification

ASTM   Poorly graded SAND with Gravel (SP)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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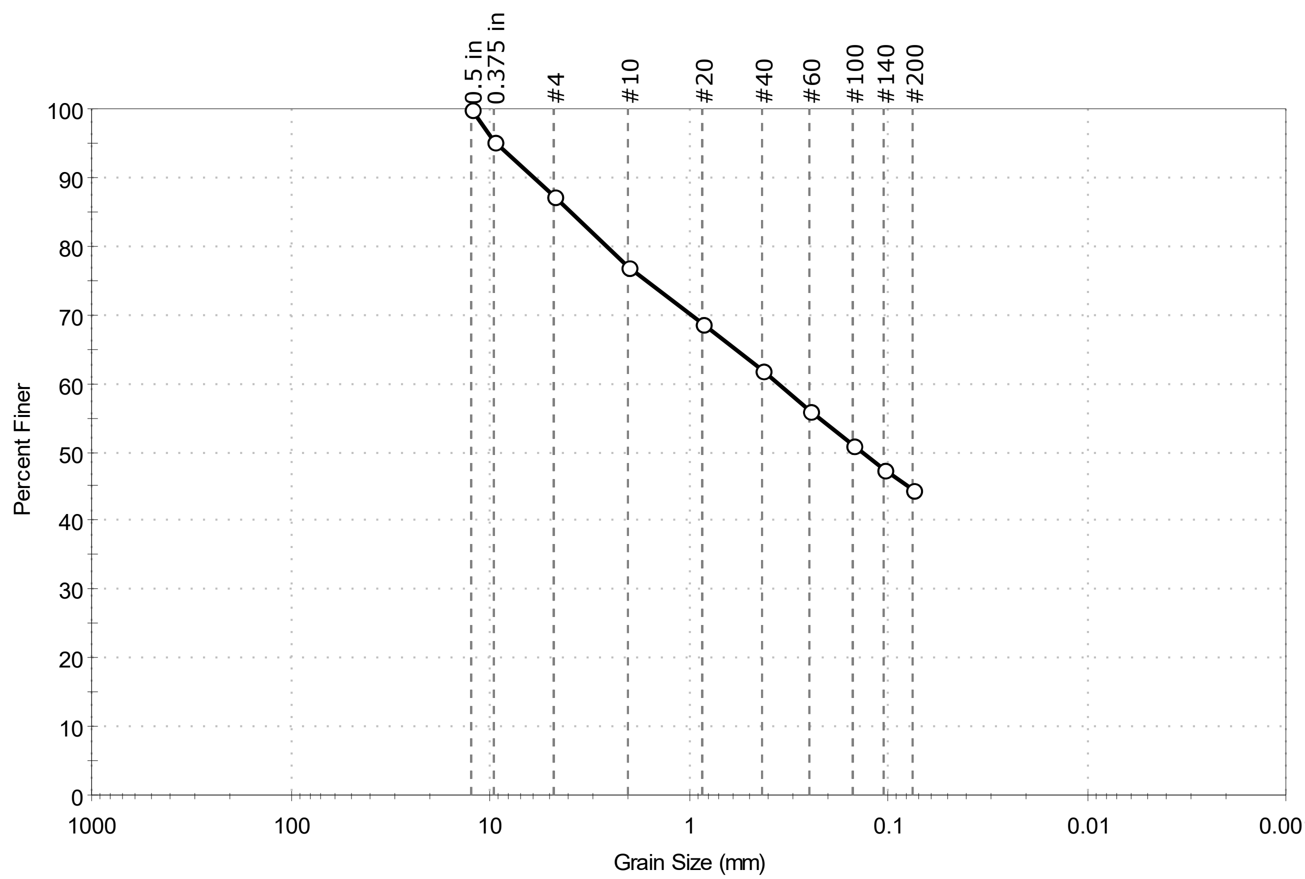
Client:	Haley & Aldrich, Inc.				
Project:	Rte 9/I-395 Conn. - Wilson St Bridge				
Location:	Brewer & Eddington, ME			Project No:	GTX-311345
Boring ID:	BB-BWS-204	Sample Type:	jar	Tested By:	ckg
Sample ID:	11D	Test Date:	03/02/20	Checked By:	emm
Depth :	21-23 ft	Test Id:	545441		
Test Comment:	---				
Visual Description:	Moist, olive gray silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.5	43.0	44.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	95		
#4	4.75	87		
#10	2.00	77		
#20	0.85	69		
#40	0.42	62		
#60	0.25	56		
#100	0.15	51		
#140	0.11	48		
#200	0.075	45		

Coefficients

D_{85} =3.8679 mm	D_{30} =N/A
D_{60} =0.3550 mm	D_{15} =N/A
D_{50} =0.1356 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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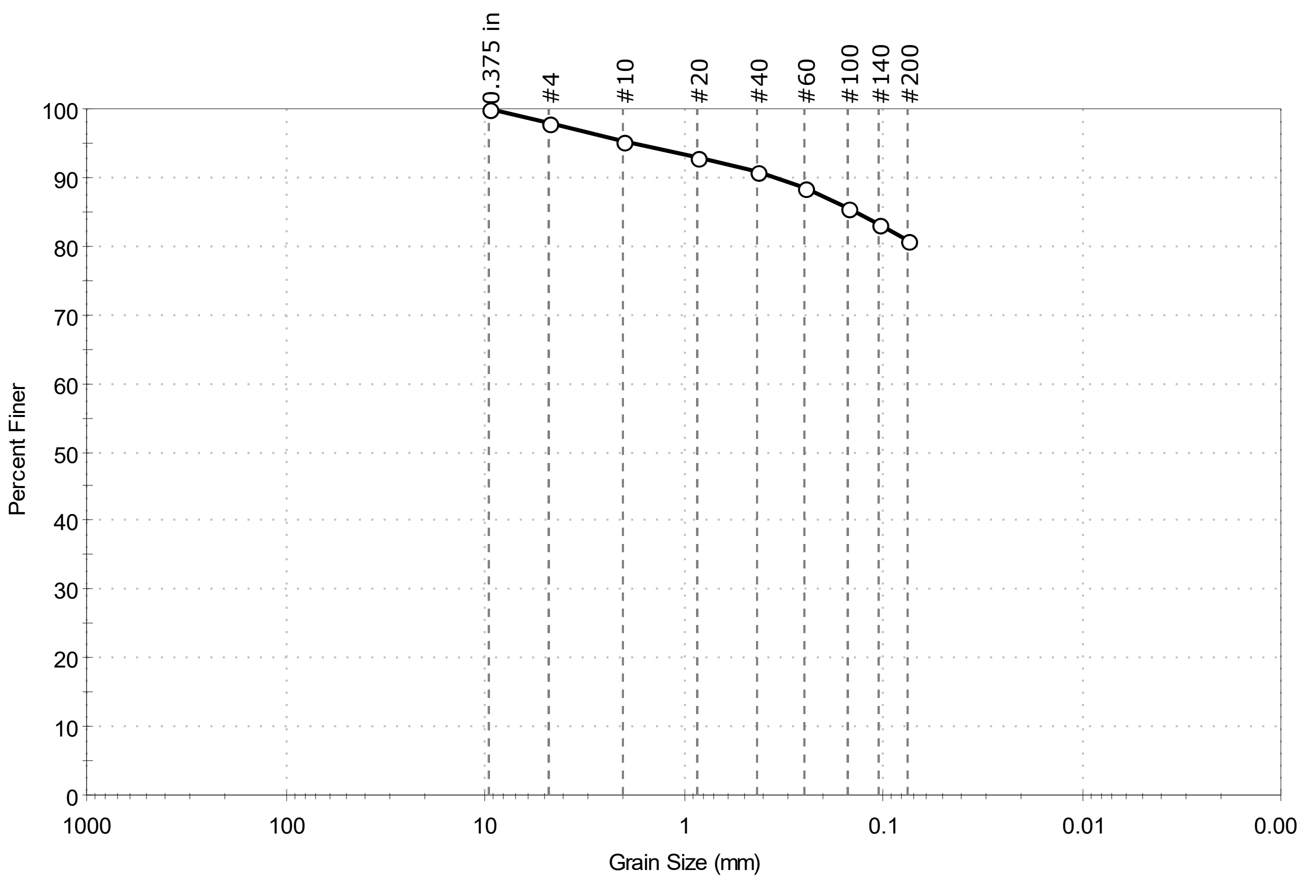
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-204	Sample Type: jar	Tested By:	ckg
Sample ID: 13D	Test Date: 03/03/20	Checked By:	emm
Depth : 25-27 ft	Test Id: 545442		
Test Comment:	---		
Visual Description:	Moist, light olive brown silt with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	2.1	17.1	80.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	98		
#10	2.00	95		
#20	0.85	93		
#40	0.42	91		
#60	0.25	89		
#100	0.15	86		
#140	0.11	83		
#200	0.075	81		

Coefficients

D_{85} =0.1363 mm | D_{30} =N/A

D_{60} =N/A | D_{15} =N/A

D_{50} =N/A | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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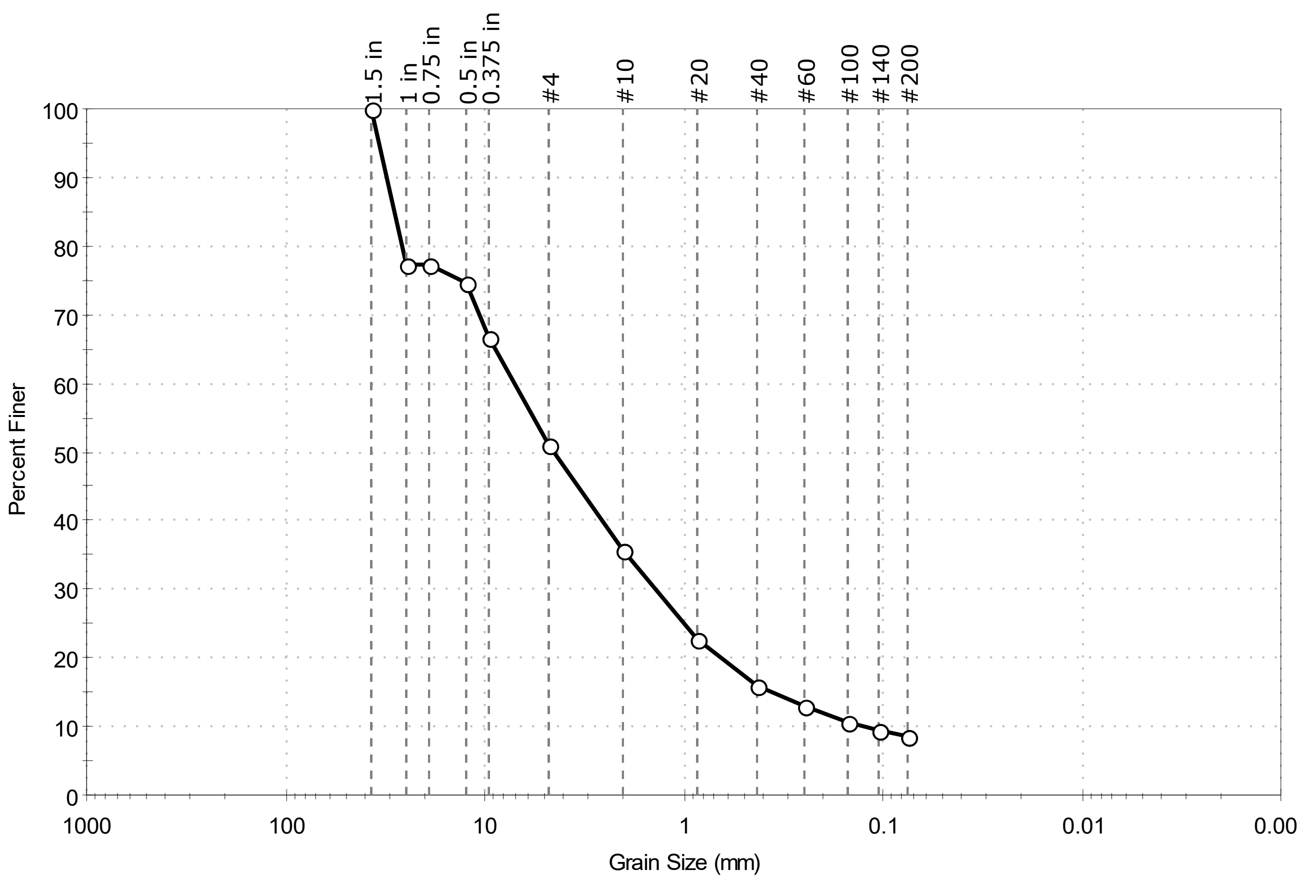
Client:	Haley & Aldrich, Inc.			
Project:	Rte 9/I-395 Conn. - Wilson St Bridge			
Location:	Brewer & Eddington, ME		Project No:	GTX-311345
Boring ID:	BB-BWS-205	Sample Type: jar	Tested By:	ckg
Sample ID:	2D	Test Date: 03/03/20	Checked By:	emm
Depth :	2-4 ft	Test Id: 545432		

Test Comment: ---

Visual Description: Moist, brown gravel with silt and sand

Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	48.9	42.7	8.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	77		
0.75 in	19.00	77		
0.5 in	12.50	75		
0.375 in	9.50	67		
#4	4.75	51		
#10	2.00	36		
#20	0.85	23		
#40	0.42	16		
#60	0.25	13		
#100	0.15	11		
#140	0.11	9		
#200	0.075	8.4		

Coefficients

D_{85} =28.7280 mm | D_{30} =1.3819 mm

D_{60} =7.0347 mm | D_{15} =0.3570 mm

D_{50} =4.4483 mm | D_{10} =0.1254 mm

C_u =56.098 | C_c =2.165

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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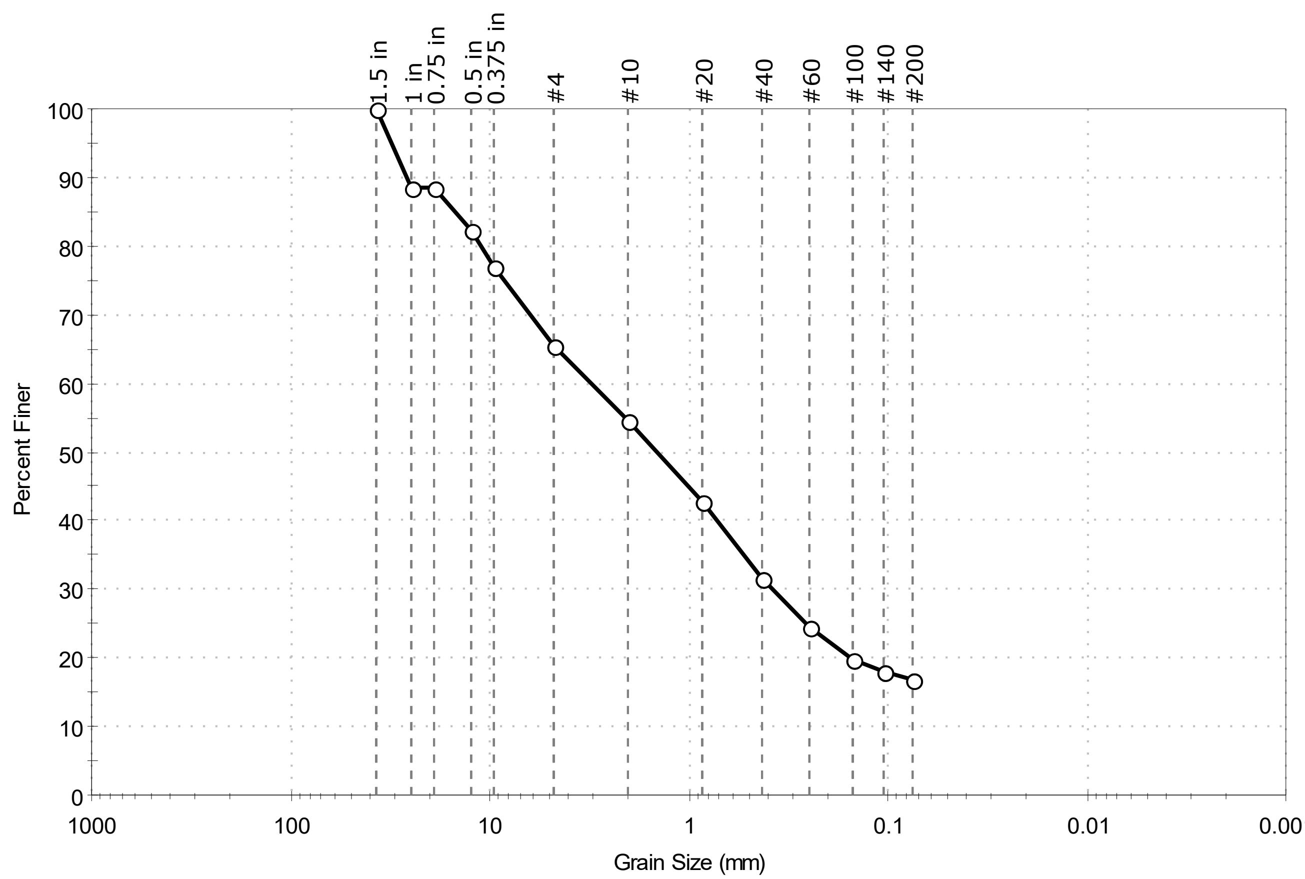
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID:	BB-BWS-205	Sample Type: jar	Tested By: ckg
Sample ID:	4D	Test Date: 03/03/20	Checked By: emm
Depth :	6-8 ft	Test Id: 545434	
Test Comment:	---		
Visual Description:	Moist, dark olive brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.5	48.8	16.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	88		
0.75 in	19.00	88		
0.5 in	12.50	82		
0.375 in	9.50	77		
#4	4.75	66		
#10	2.00	55		
#20	0.85	43		
#40	0.42	32		
#60	0.25	24		
#100	0.15	20		
#140	0.11	18		
#200	0.075	17		

Coefficients

D_{85} =15.0568 mm D_{30} =0.3755 mm

D_{60} =3.0554 mm D_{15} =N/A

D_{50} =1.4239 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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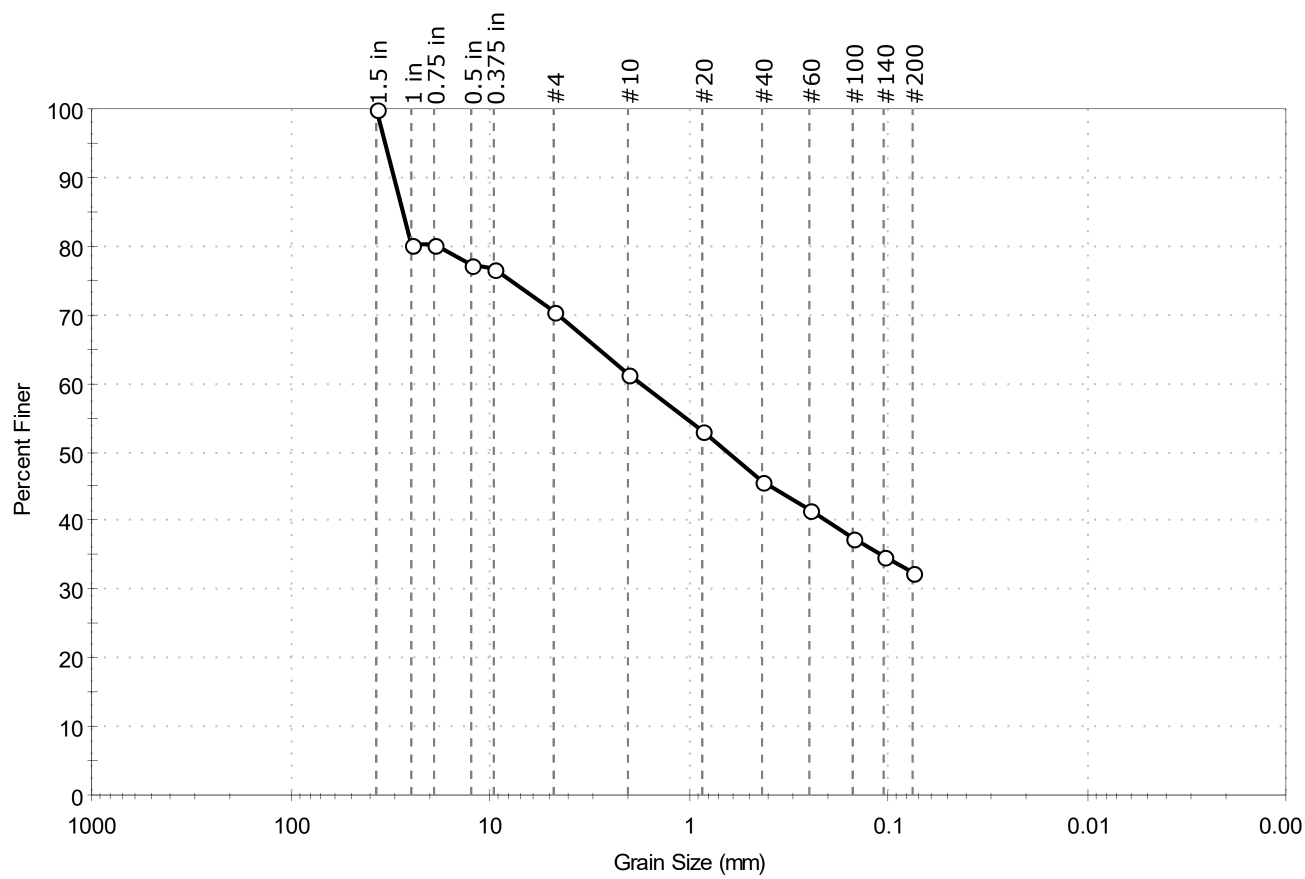
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-205	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 03/03/20	Checked By:	emm
Depth : 8-10 ft	Test Id: 545435		
Test Comment:	---		
Visual Description:	Moist, olive brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	29.7	37.9	32.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	80		
0.75 in	19.00	80		
0.5 in	12.50	77		
0.375 in	9.50	77		
#4	4.75	70		
#10	2.00	61		
#20	0.85	53		
#40	0.42	46		
#60	0.25	42		
#100	0.15	37		
#140	0.11	35		
#200	0.075	32		

Coefficients

D_{85} =27.5656 mm	D_{30} =N/A
D_{60} =1.7284 mm	D_{15} =N/A
D_{50} =0.6363 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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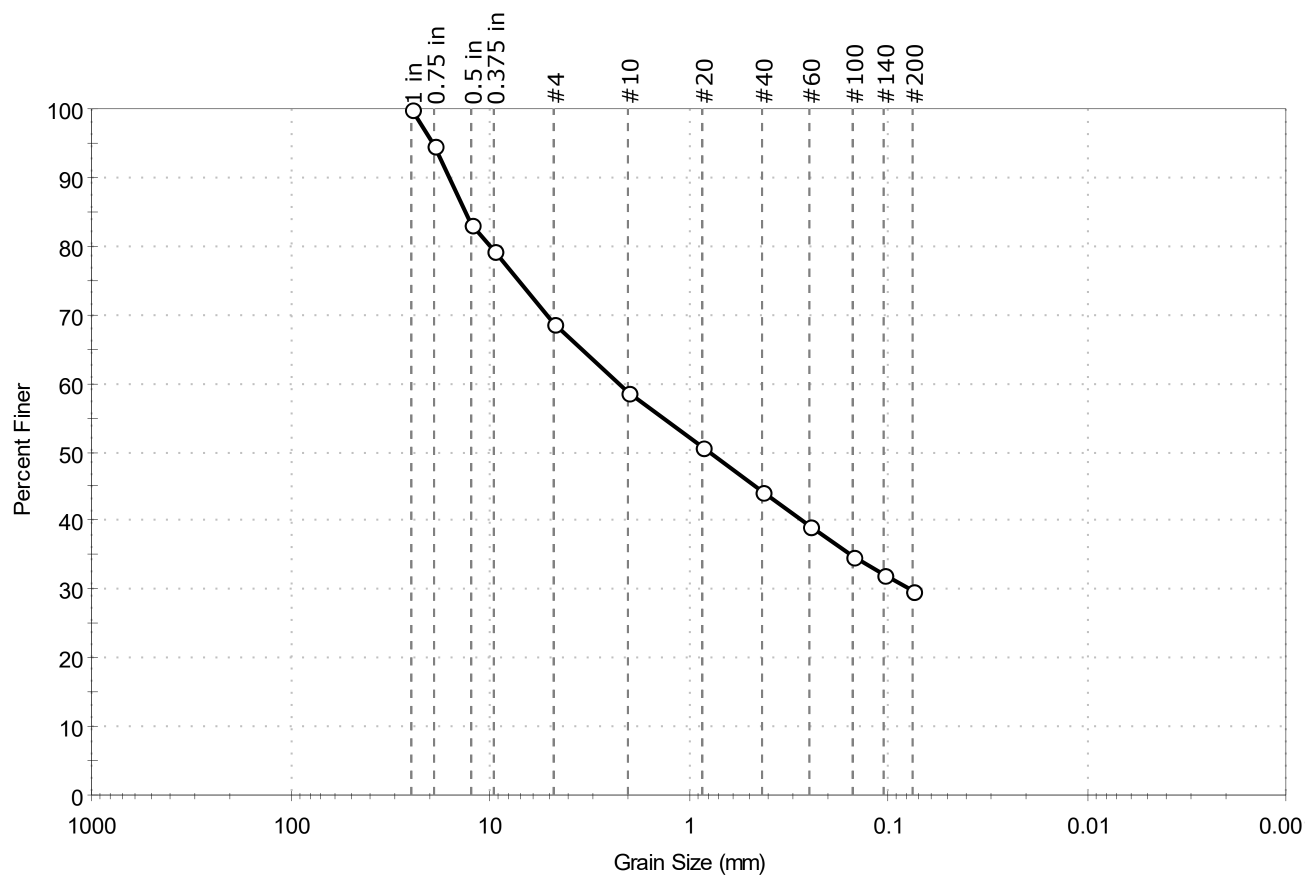
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID:	BB-BWS-205	Sample Type: jar	Tested By: ckg
Sample ID:	6D	Test Date: 03/03/20	Checked By: emm
Depth :	10-12 ft	Test Id: 545436	
Test Comment:	---		
Visual Description:	Moist, olive brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	31.4	38.9	29.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	95		
0.5 in	12.50	83		
0.375 in	9.50	79		
#4	4.75	69		
#10	2.00	59		
#20	0.85	51		
#40	0.42	44		
#60	0.25	39		
#100	0.15	35		
#140	0.11	32		
#200	0.075	30		

Coefficients

D_{85} =13.3002 mm D_{30} =0.0785 mm

D_{60} =2.2265 mm D_{15} =N/A

D_{50} =0.7889 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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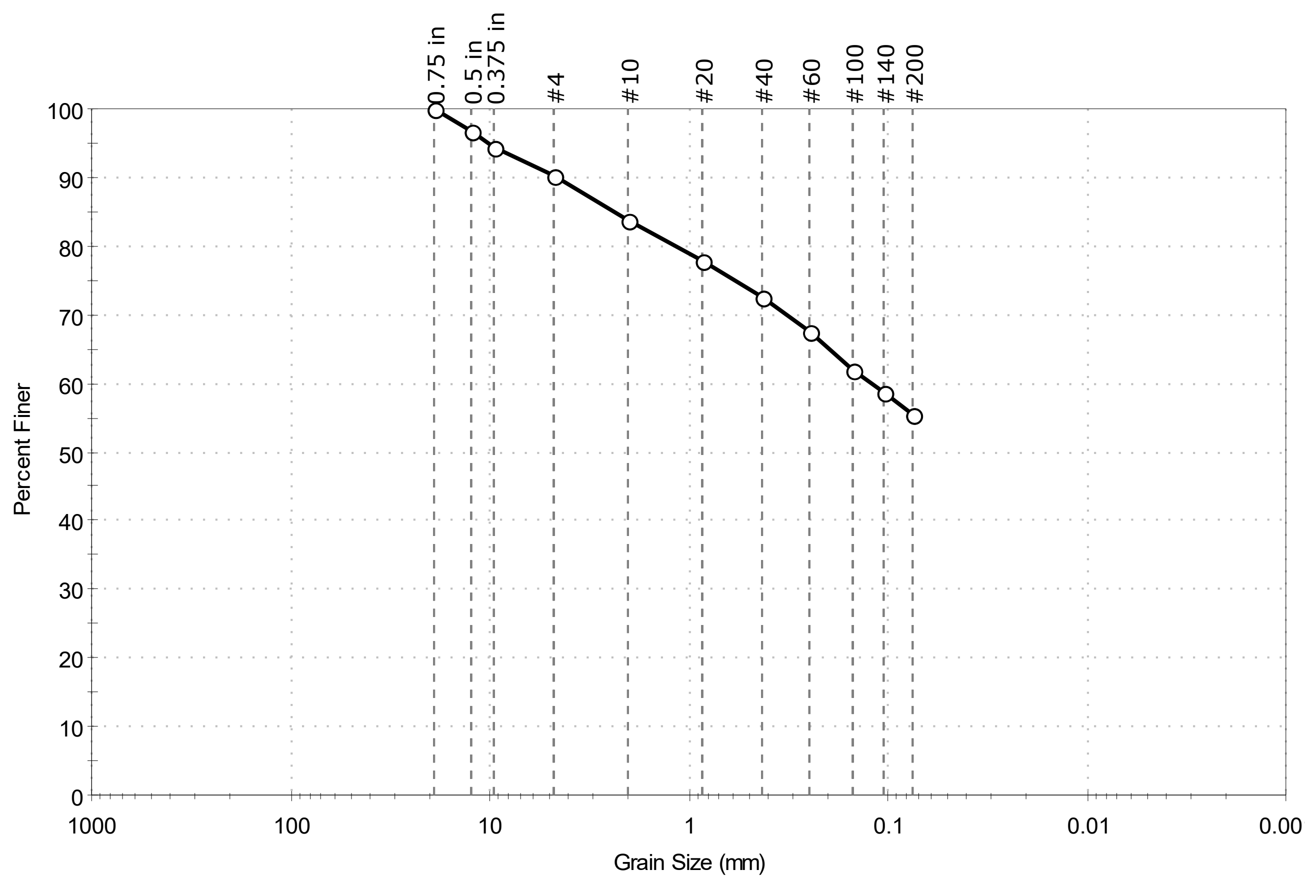
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.				
Project:	Rte 9/I-395 Conn. - Wilson St Bridge				
Location:	Brewer & Eddington, ME			Project No:	GTX-311345
Boring ID:	BB-BWS-205	Sample Type:	jar	Tested By:	ckg
Sample ID:	10D	Test Date:	03/03/20	Checked By:	emm
Depth :	18-20 ft	Test Id:	545439		
Test Comment:	---				
Visual Description:	Moist, olive brown sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.8	34.8	55.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	94		
#4	4.75	90		
#10	2.00	84		
#20	0.85	78		
#40	0.42	73		
#60	0.25	67		
#100	0.15	62		
#140	0.11	59		
#200	0.075	55		

Coefficients

D_{85} =2.3331 mm
D_{30} =N/A
D_{60} =0.1227 mm
D_{15} =N/A
D_{50} =N/A
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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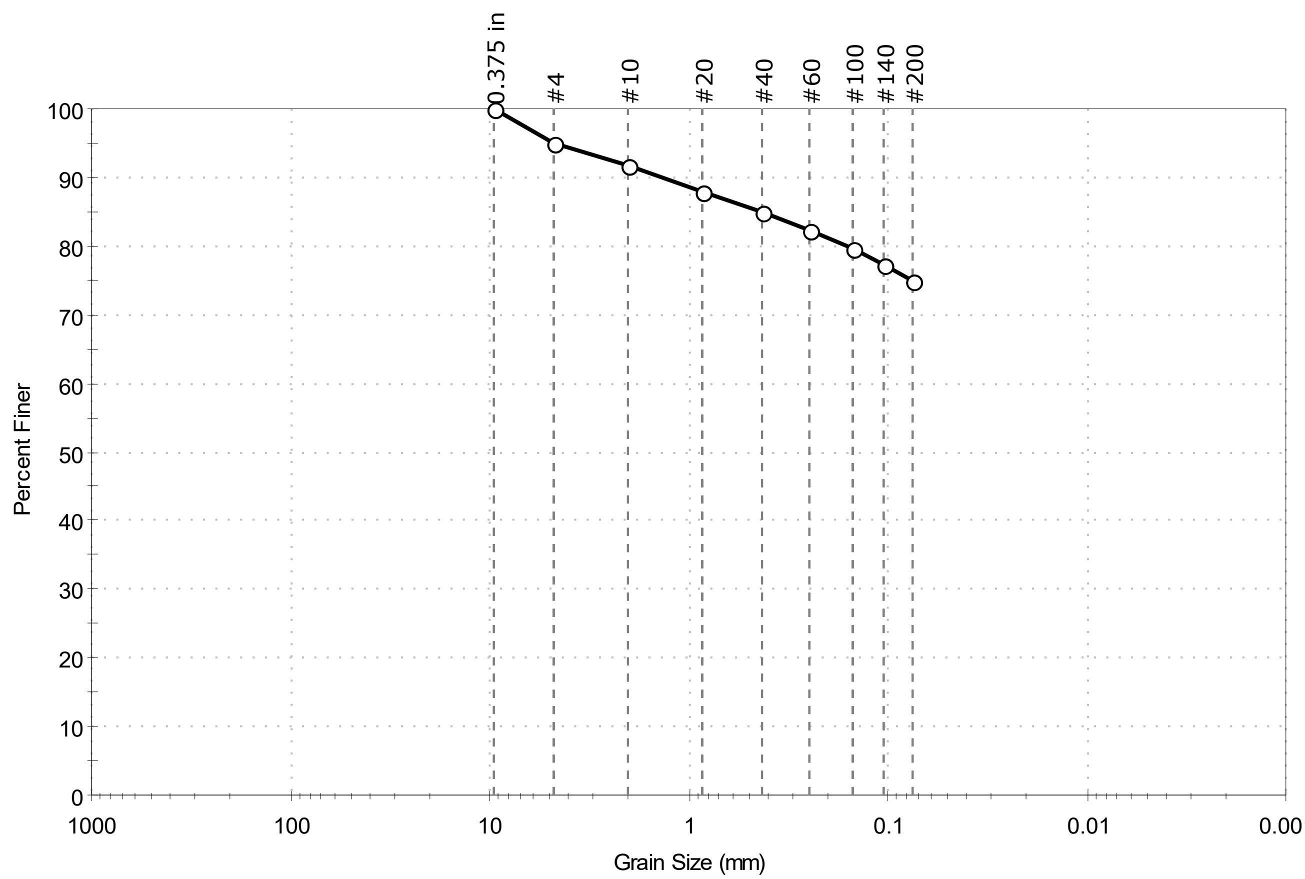
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-301	Sample Type: jar	Tested By:	ckg
Sample ID: 2D	Test Date: 03/03/20	Checked By:	emm
Depth : 5-7 ft	Test Id: 545430		
Test Comment:	---		
Visual Description:	Moist, olive silt with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.1	19.9	75.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	95		
#10	2.00	92		
#20	0.85	88		
#40	0.42	85		
#60	0.25	82		
#100	0.15	80		
#140	0.11	77		
#200	0.075	75		

Coefficients

D_{85} =0.4320 mm | D_{30} =N/A

D_{60} =N/A | D_{15} =N/A

D_{50} =N/A | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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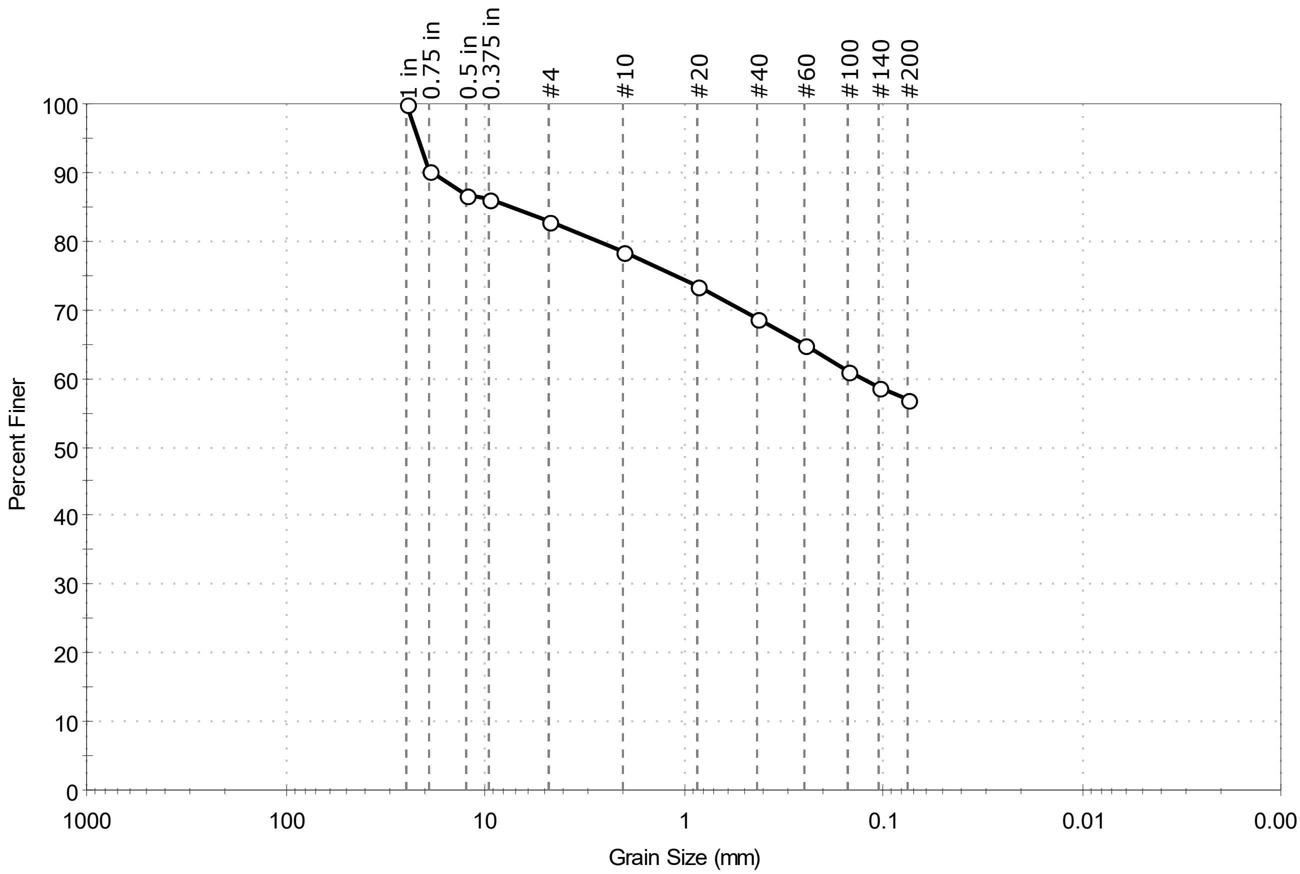
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-301	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 03/03/20	Checked By:	emm
Depth : 10-12 ft	Test Id: 545431		
Test Comment:	---		
Visual Description:	Moist, olive gray sandy silt with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.0	26.0	57.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	90		
0.5 in	12.50	87		
0.375 in	9.50	86		
#4	4.75	83		
#10	2.00	78		
#20	0.85	73		
#40	0.42	69		
#60	0.25	65		
#100	0.15	61		
#140	0.11	59		
#200	0.075	57		

Coefficients

D_{85} =7.5034 mm D_{30} =N/A

D_{60} =0.1269 mm D_{15} =N/A

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

C_u =N/A C_c =N/A

Classification

ASTM   N/A

AASHTO   Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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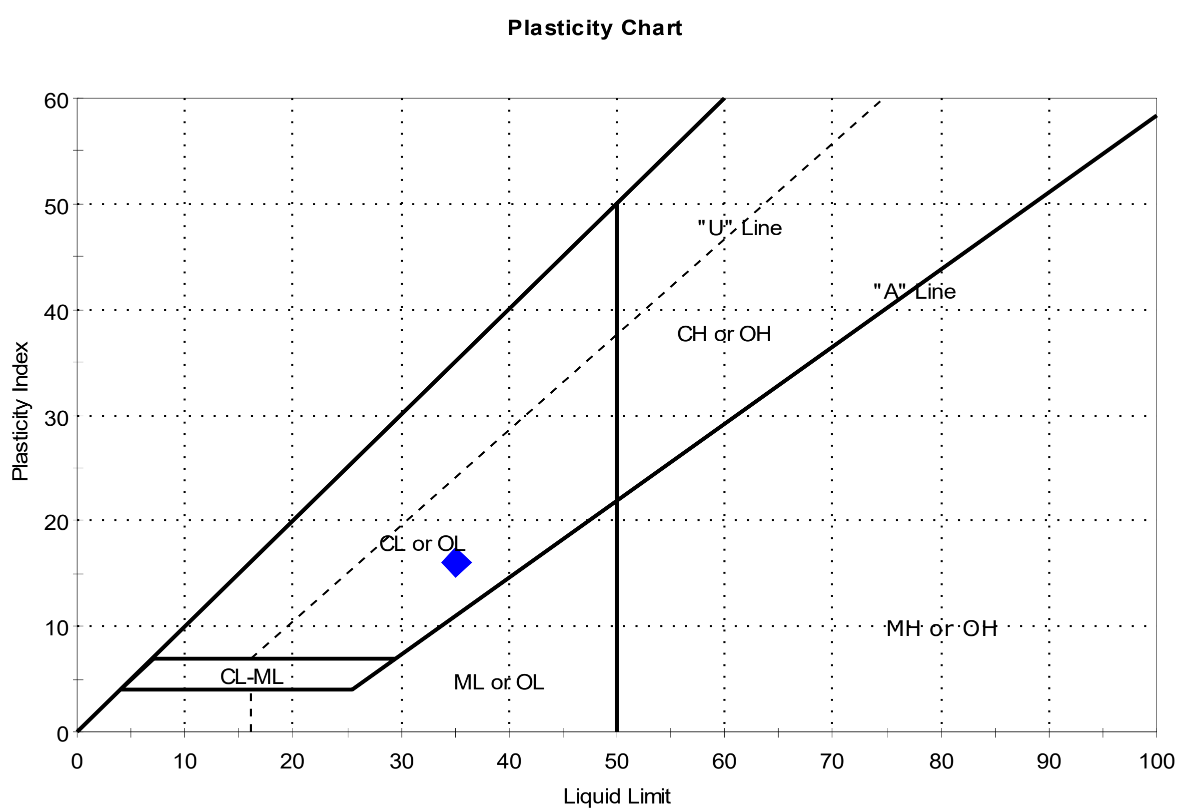
GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-301	Sample Type: tube	Tested By:	cam
Sample ID: 3U	Test Date: 03/02/20	Checked By:	emm
Depth : 29-31 ft	Test Id: 545426		
Test Comment:	---		
Visual 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
◆	3U	3-BWS-3(	29-31 ft	32	35	19	16	0.8	

Sample Prepared using the WET method

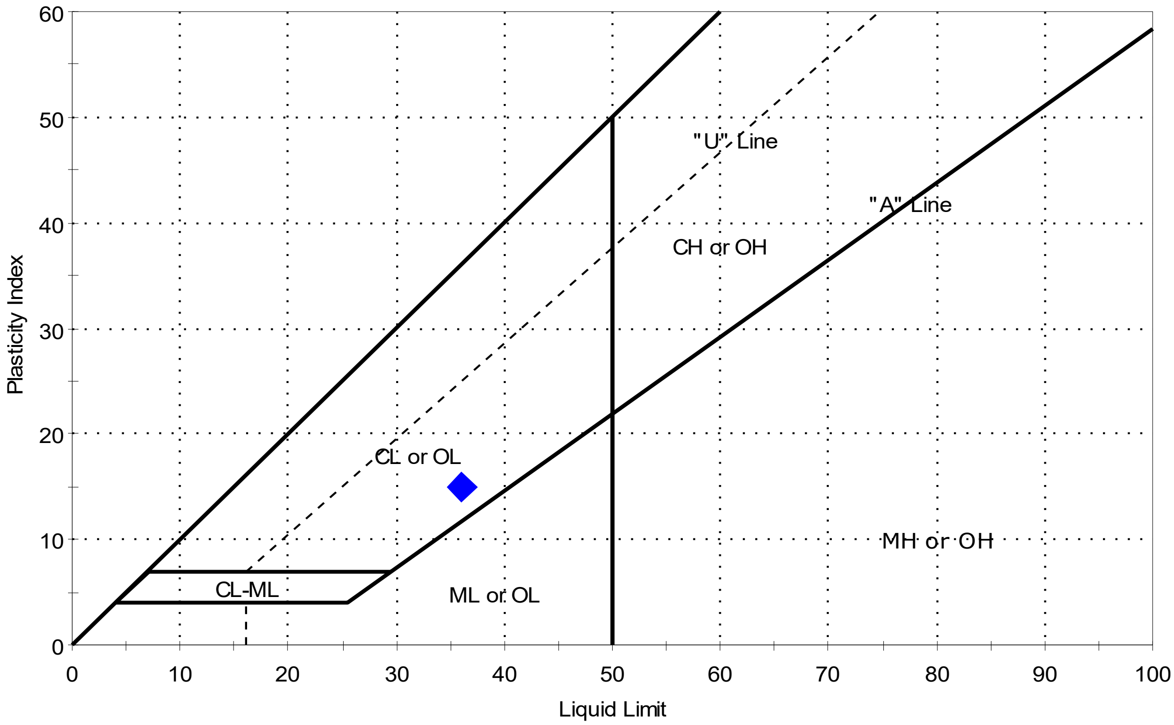
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client: Haley & Aldrich, Inc.			
Project: Rte 9/I-395 Conn. - Wilson St Bridge			
Location: Brewer & Eddington, ME		Project No:	GTX-311345
Boring ID: BB-BWS-301	Sample Type: jar	Tested By:	cam
Sample ID: 5D	Test Date: 03/03/20	Checked By:	emm
Depth : 19-21 ft	Test Id: 545427		
Test Comment: ---			
Visual Description: Moist, olive clay			
Sample Comment: ---			

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	5D	3-BWS-3(	19-21 ft	24	36	21	15	0.2	

Sample Prepared using the WET method

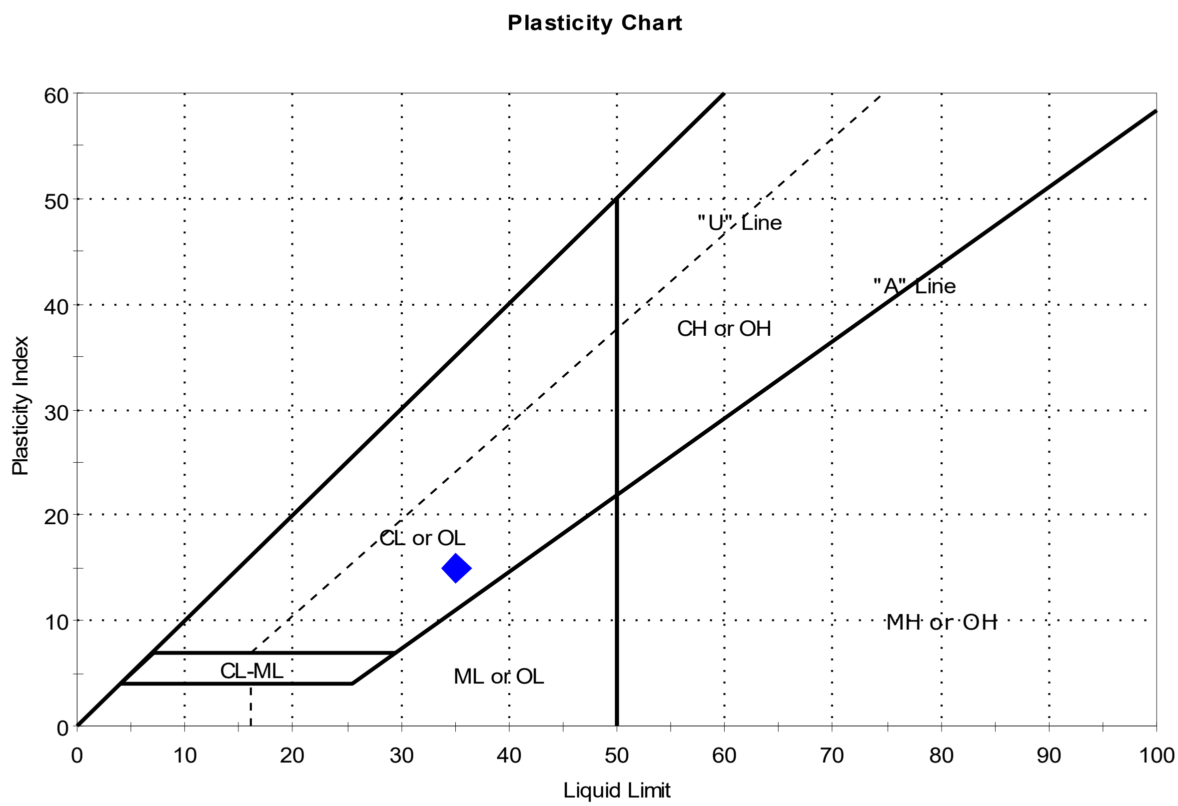
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Haley & Aldrich, Inc.				
Project:	Rte 9/I-395 Conn. - Wilson St Bridge				
Location:	Brewer & Eddington, ME			Project No:	GTX-311345
Boring ID:	BB-BWS-301	Sample Type:	jar	Tested By:	cam
Sample ID:	6D	Test Date:	03/03/20	Checked By:	emm
Depth :	24-26 ft	Test Id:	545428		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6D	3-BWS-3(	24-26 ft	29	35	20	15	0.6	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

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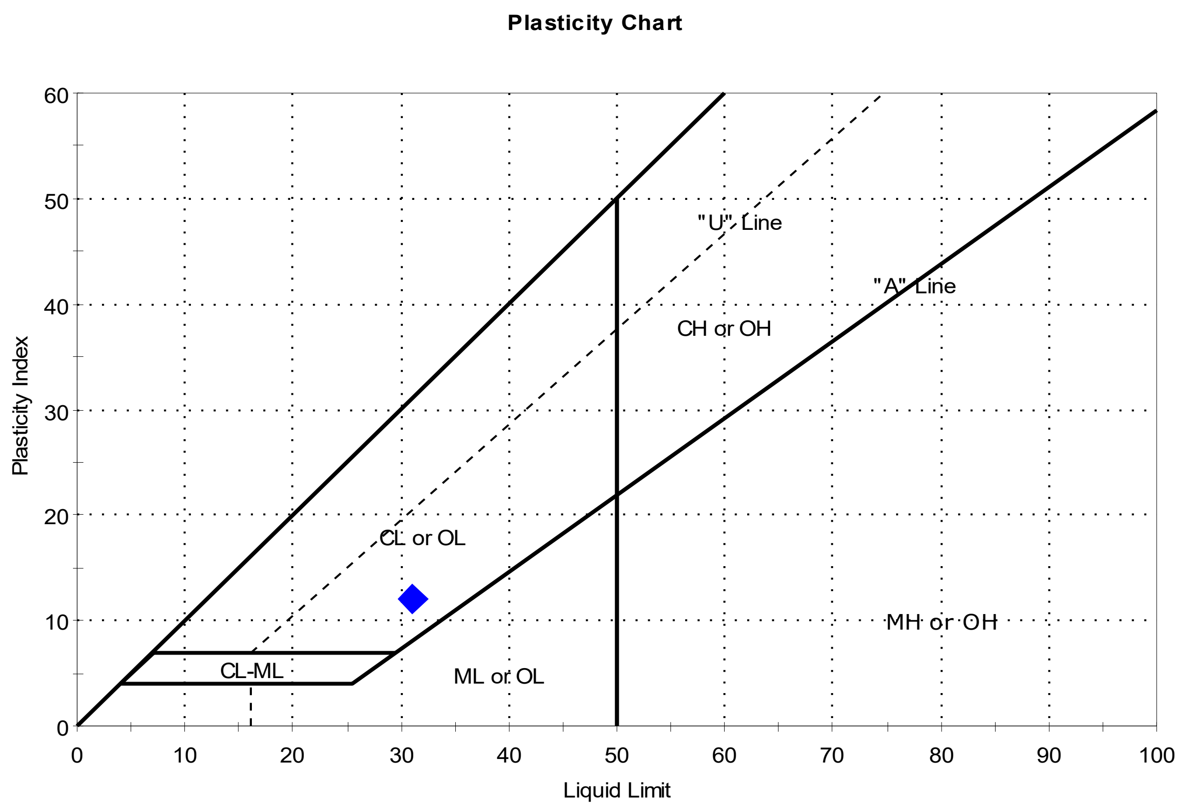
Client:	Haley & Aldrich, Inc.		
Project:	Rte 9/I-395 Conn. - Wilson St Bridge		
Location:	Brewer & Eddington, ME	Project No:	GTX-311345
Boring ID: BB-BWS-301	Sample Type: jar	Tested By:	cam
Sample ID: 7D	Test Date: 03/03/20	Checked By:	emm
Depth : 34-36 ft	Test Id: 545429		
Test Comment:	---		
Visual Description:	Moist, olive 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
◆	7D	3-BWS-3(	34-36 ft	33	31	19	12	1.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

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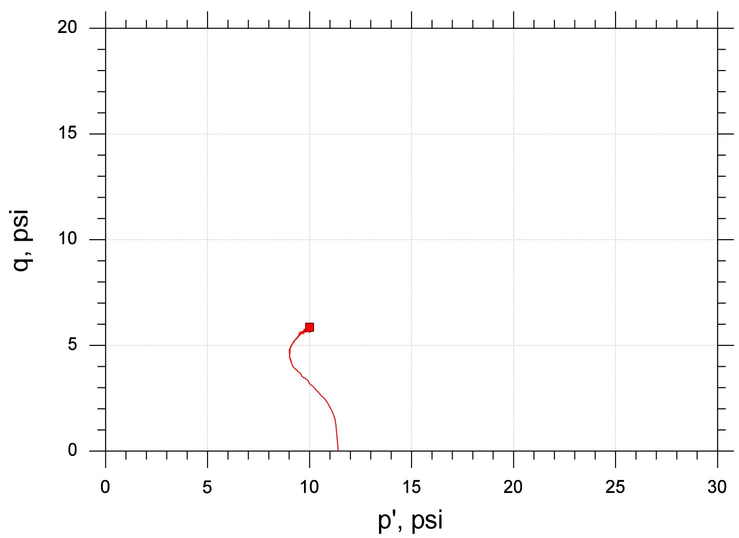
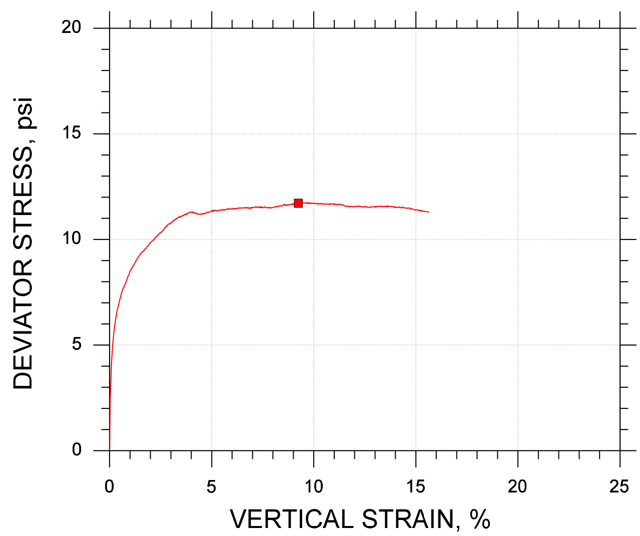
GeoTesting EXPRESS	Client: Haley & Aldrich, Inc.
	Project Name: Rte-9/I-395 Conn. - Wilson St
	Project Location: Brewer & Eddington, ME
	Project Number: GTX-311345
	Tested By: md — Checked By: njh
	Boring ID: BB-BWS-301
	Preparation: intact
	Description: Moist, gray clay
	Classification: ---
	Group Symbol: ---
	Liquid Limit: 35 — Plastic Limit: 19
	Plasticity Index: 16 — Estimated Specific Gravity: 2.7

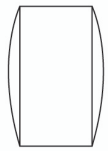
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		■			
Sample ID		3U			
Depth, ft		29-31			
Test Number		CU-1-1			
Initial	Height, in	4.530			
	Diameter, in	2.030			
	Moisture Content (from Cuttings), %	32.4			
	Dry Density, pcf	88.7			
	Saturation (Wet Method), %	97.2			
	Void Ratio	0.900			
Before Shear	Moisture Content, %	32.0			
	Dry Density, pcf	90.4			
	Cross-sectional Area (Method A), in²	3.189			
	Saturation, %	100.0			
	Void Ratio	0.864			
	Back Pressure, psi	154.9			
Vertical Effective Consolidation Stress, psi		11.38			
Horizontal Effective Consolidation Stress, psi		11.41			
Vertical Strain after Consolidation, %		0.4404			
Volumetric Strain after Consolidation, %		1.952			
Time to 50% Consolidation, min		49.00			
Shear Strength, psi		5.856			
Strain at Failure, %		9.24			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psi		11.71			
Effective Minor Principal Stress at Failure, psi		4.140			
Effective Major Principal Stress at Failure, psi		15.85			
B-Value		0.95			

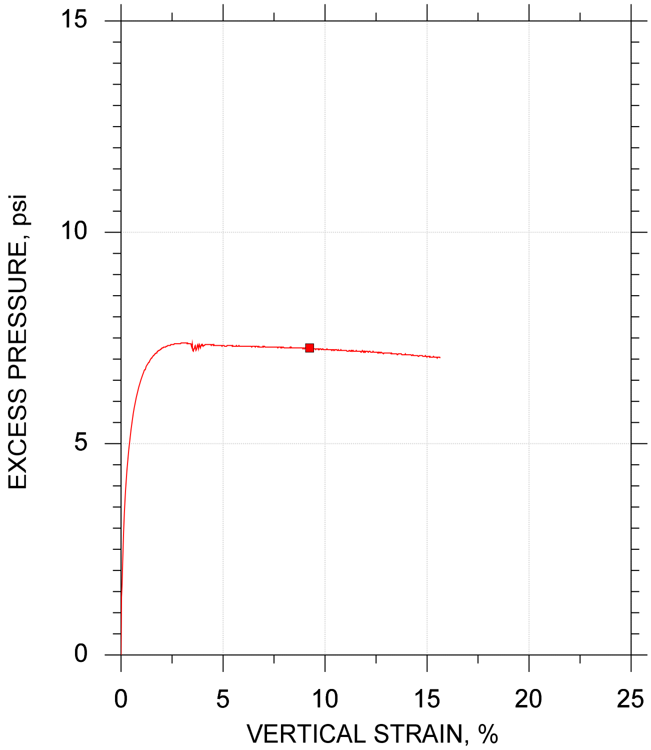
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

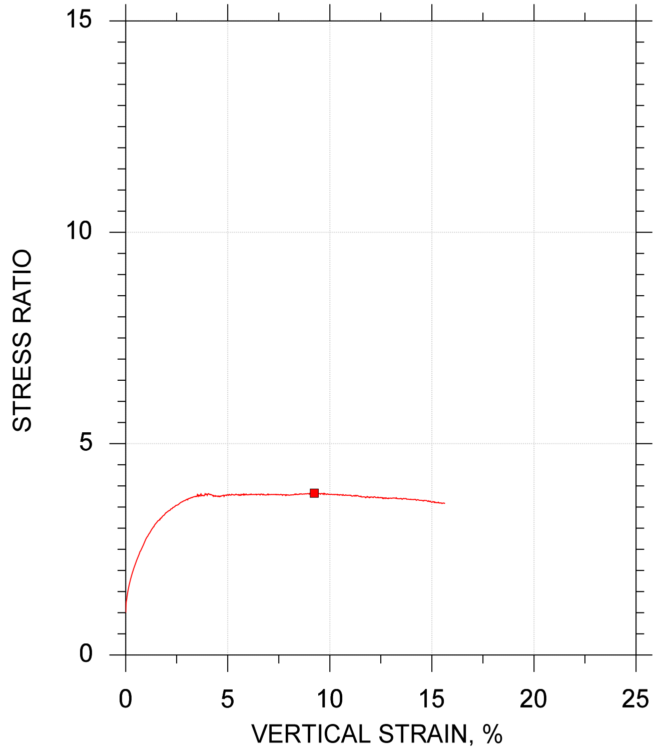


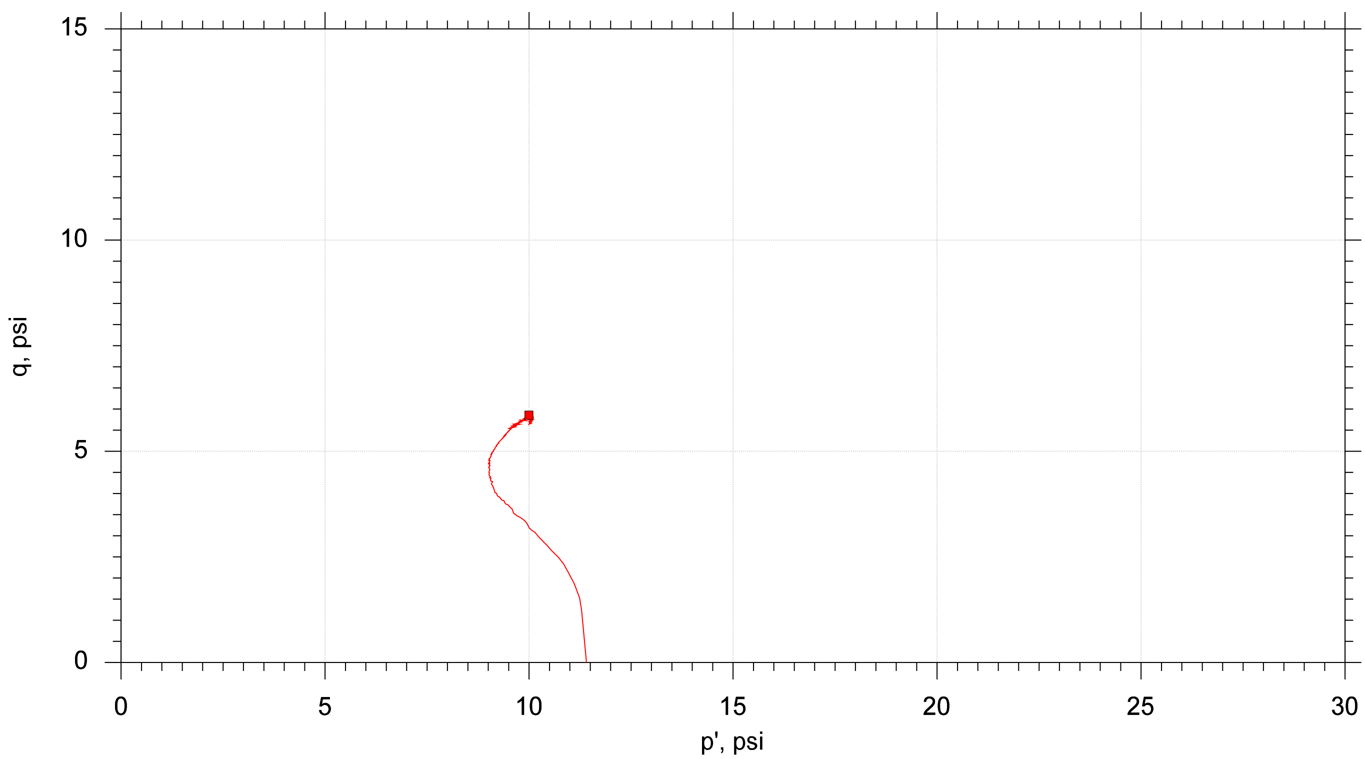
Remarks:

System RR

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





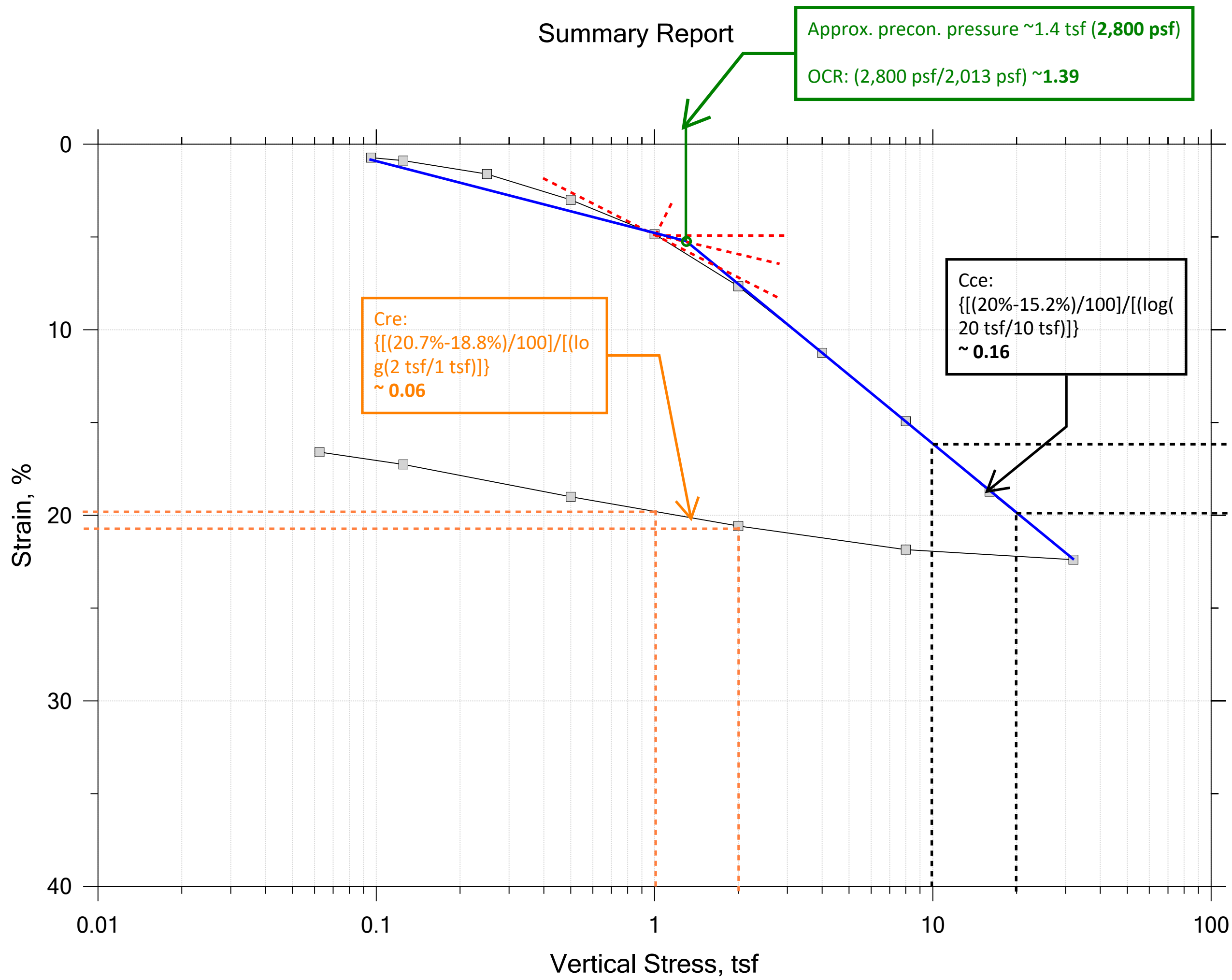


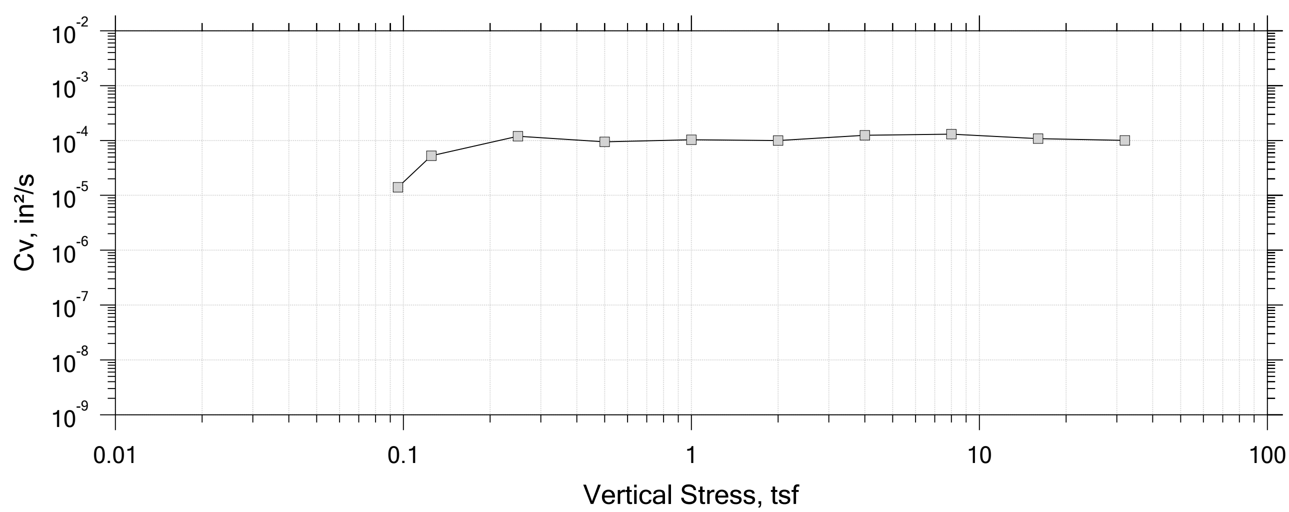
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	3U	CU-1-1	29-31	md	02/18/20	njh	---	311345-CU-1-1n.dat

Project: Rte-9/I-395 Conn. - Wilson St	Location: Brewer & Eddington, ME	Project No.: GTX-311345
Boring No.: BB-BWS-301	Sample Type: intact	
Description: Moist, gray clay		
Remarks: System RR		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

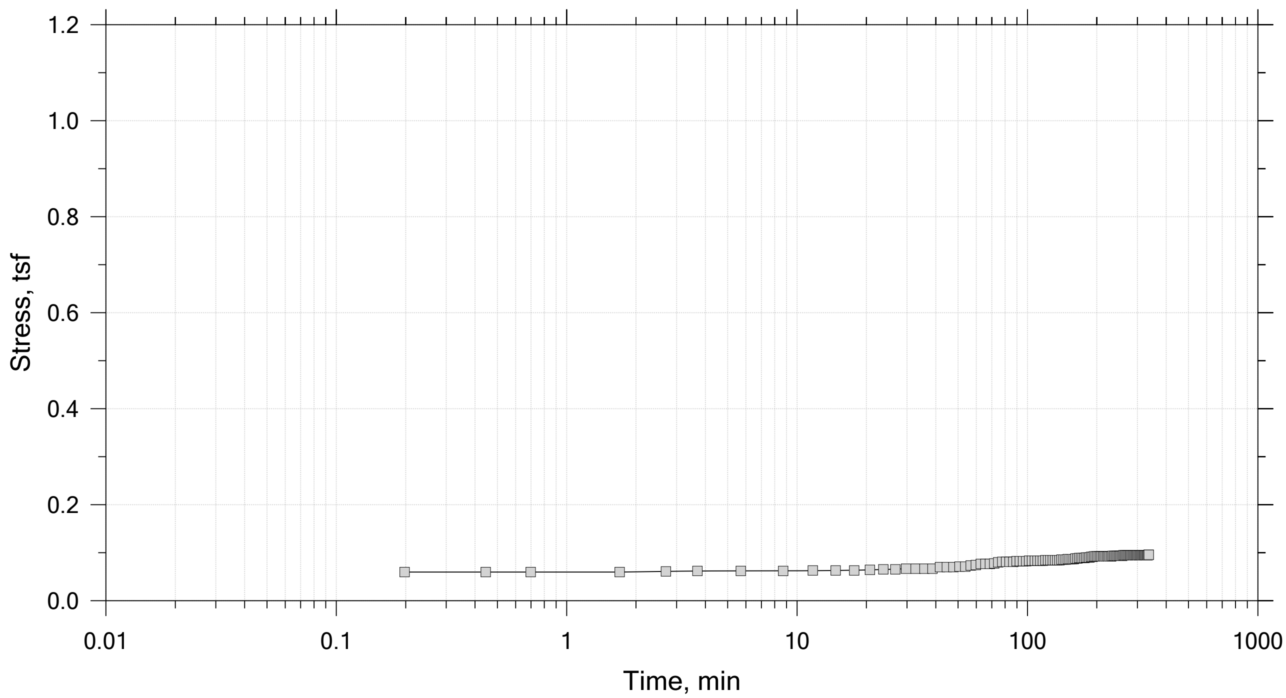


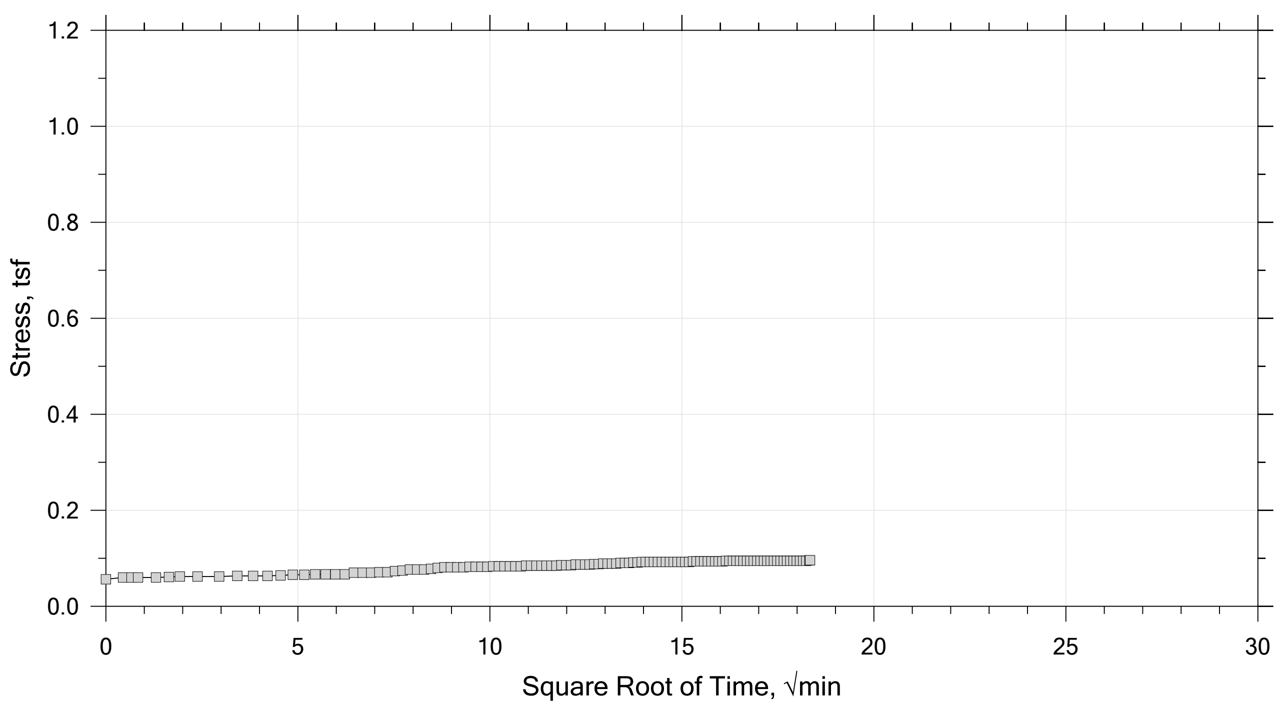


GeoTesting EXPRESS			
	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 15
Constant Volume Step
Stress: 0.0958 tsf

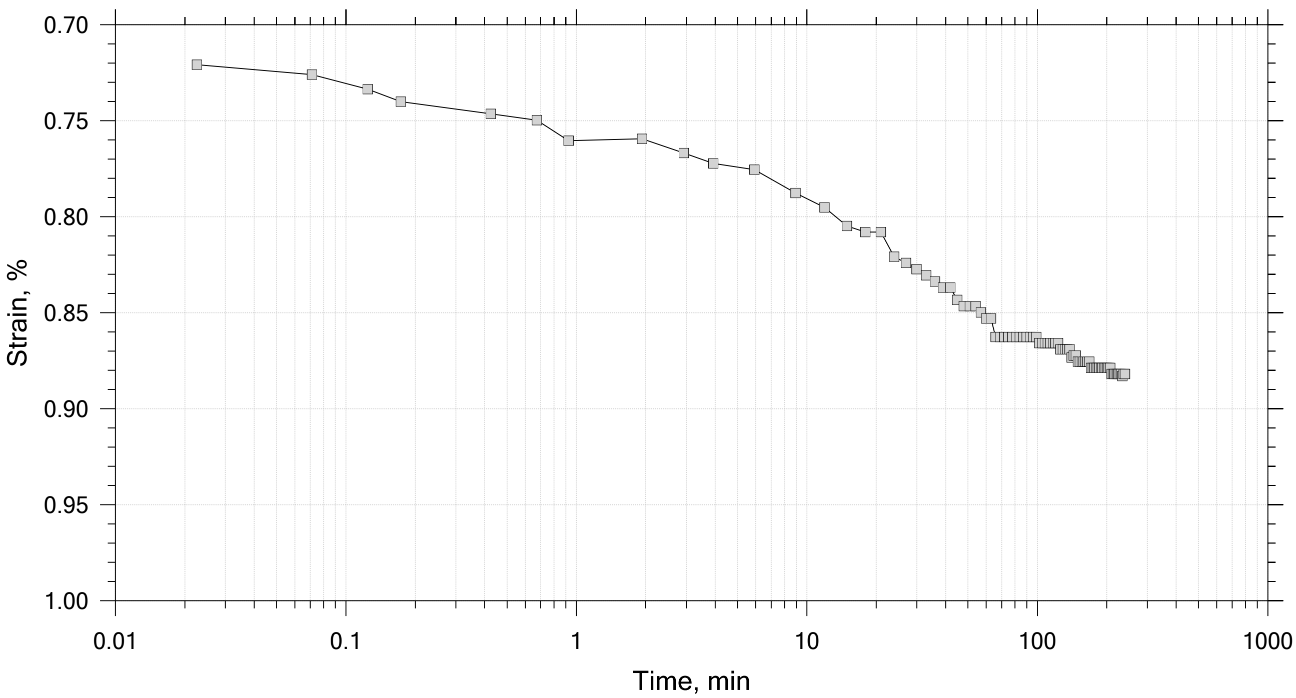


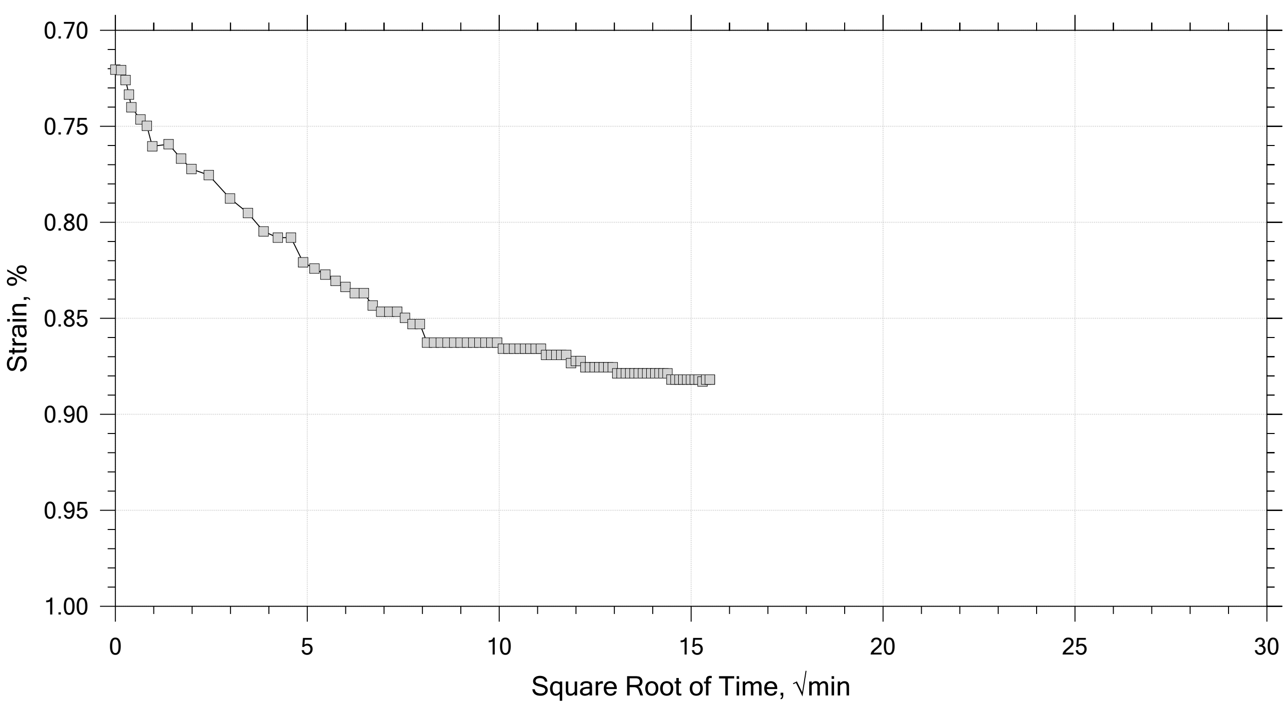


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 15
Constant Load Step
Stress: 0.125 tsf





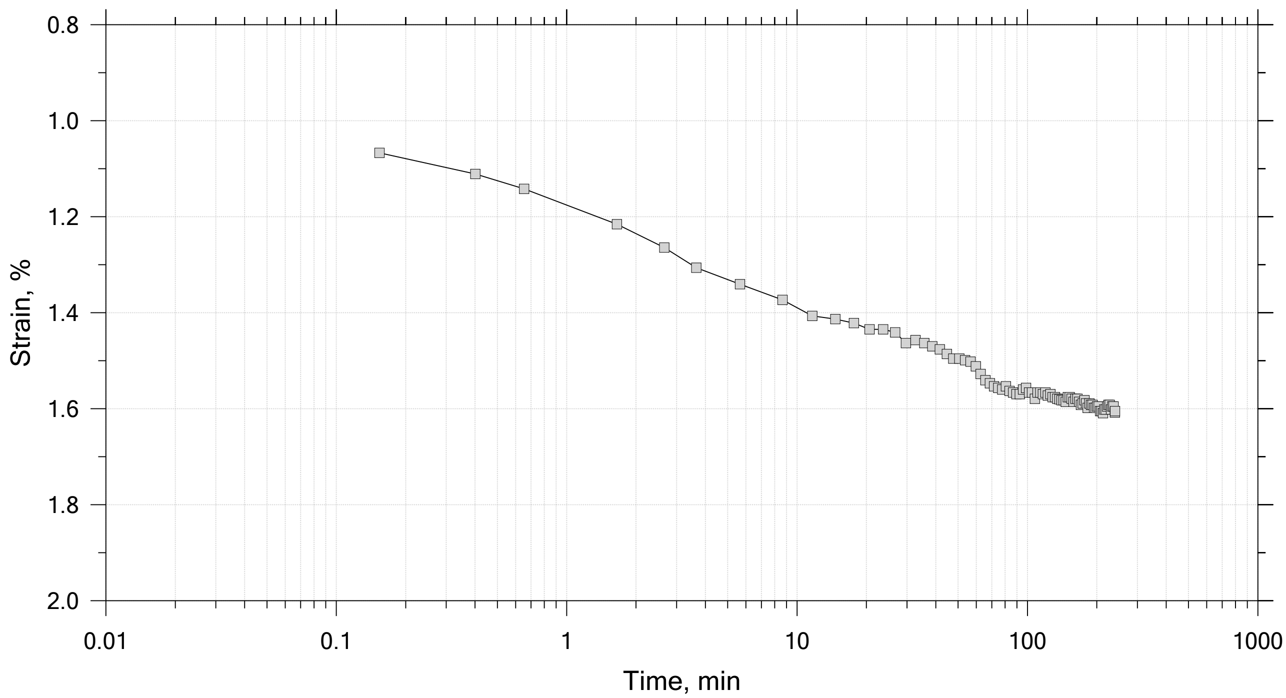
GeoTesting EXPRESS			
	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

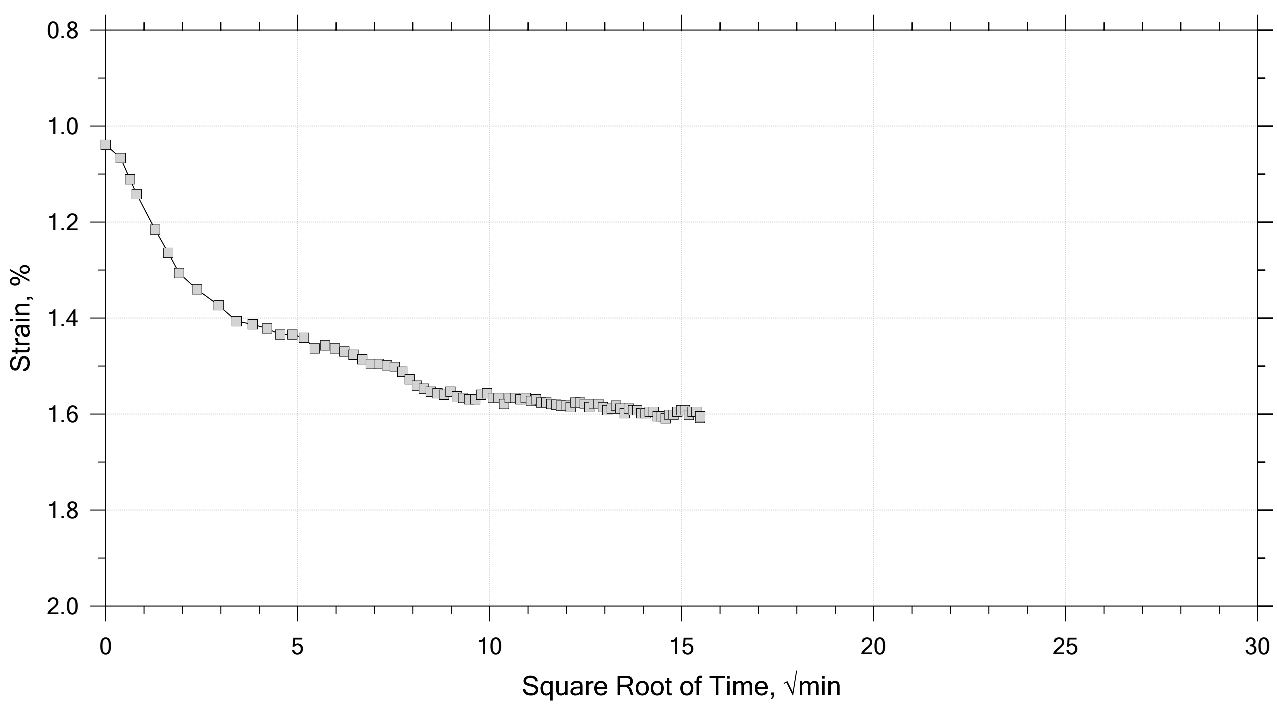
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 15

Constant Load Step

Stress: 0.25 tsf

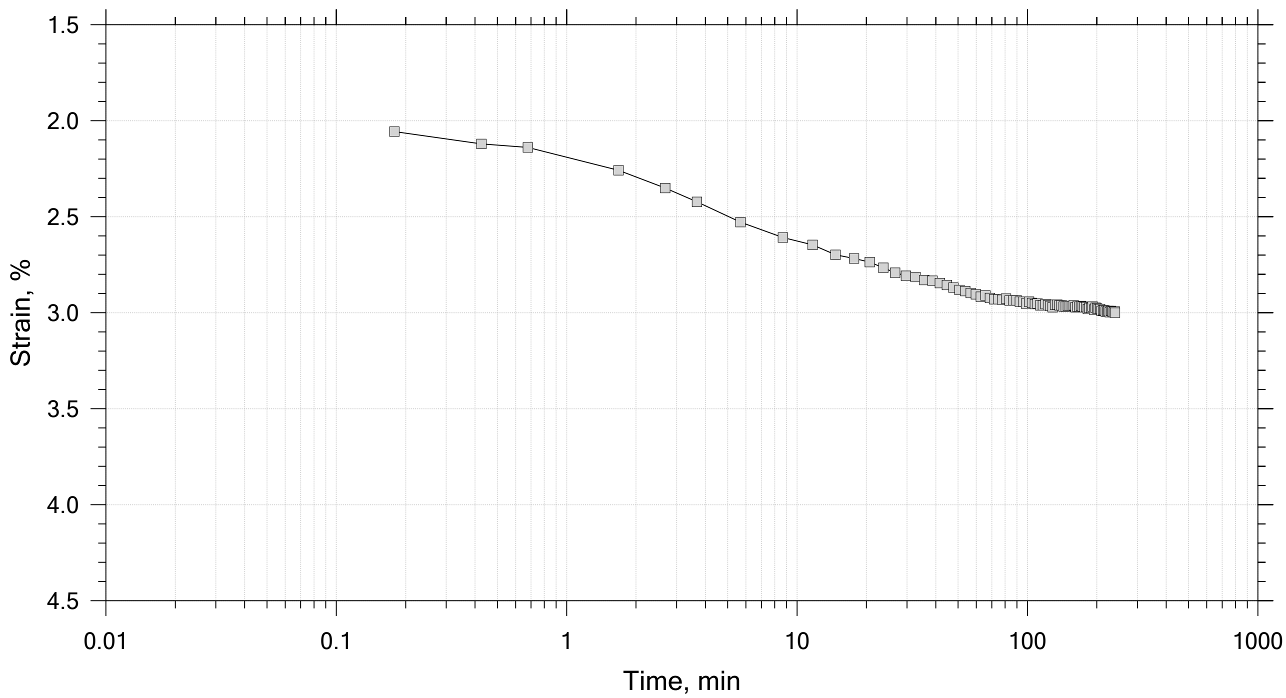


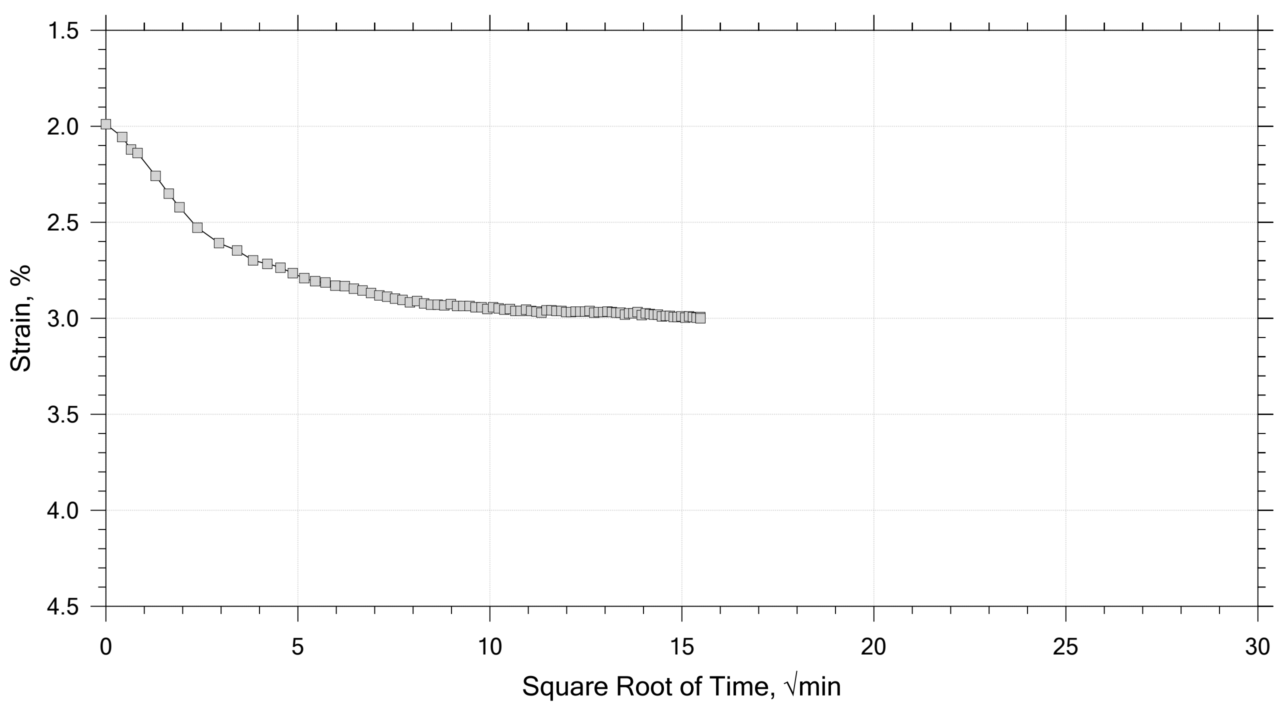


Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
Test No.: IP-1	Sample Type: tube	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 15
Constant Load Step
Stress: 0.5 tsf





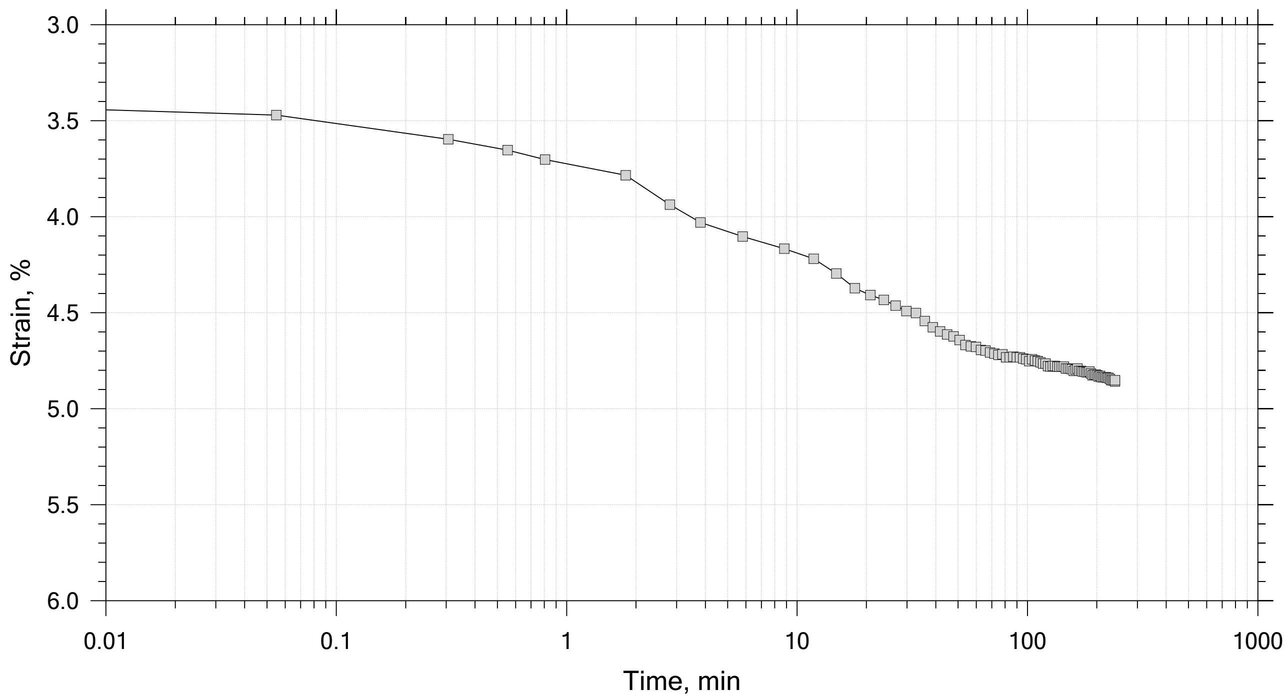
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

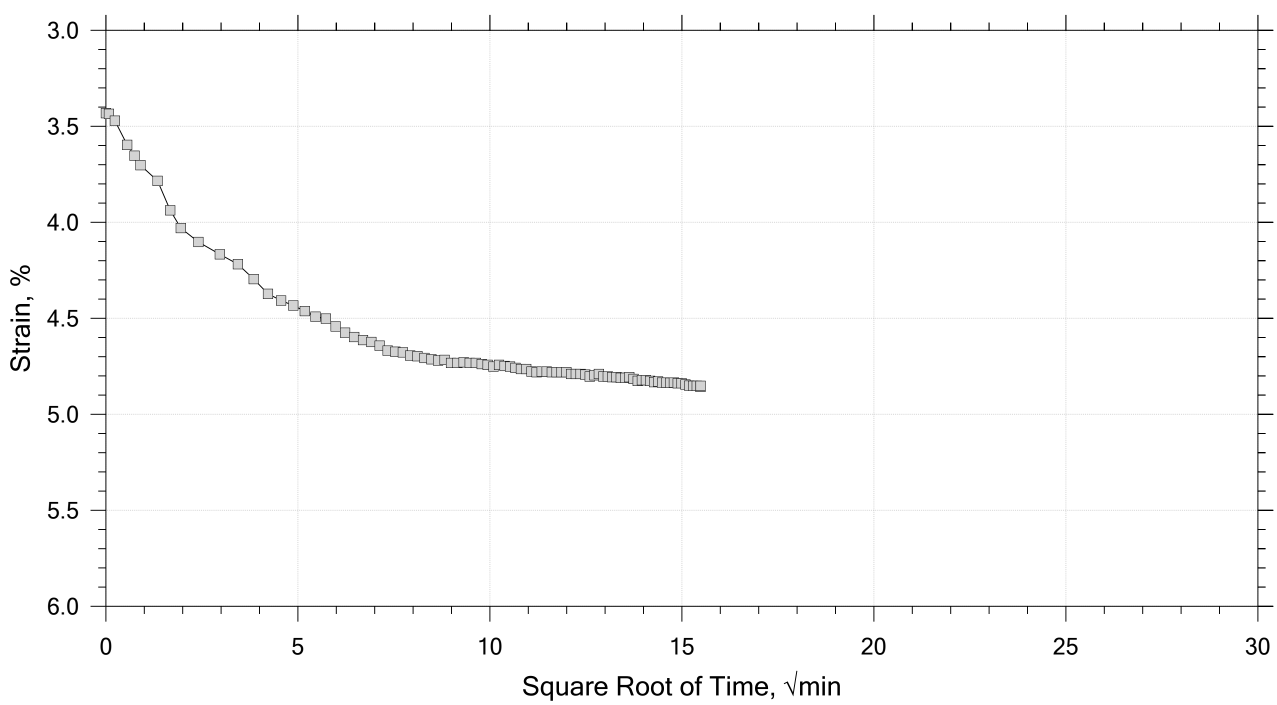
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 15

Constant Load Step

Stress: 1 tsf





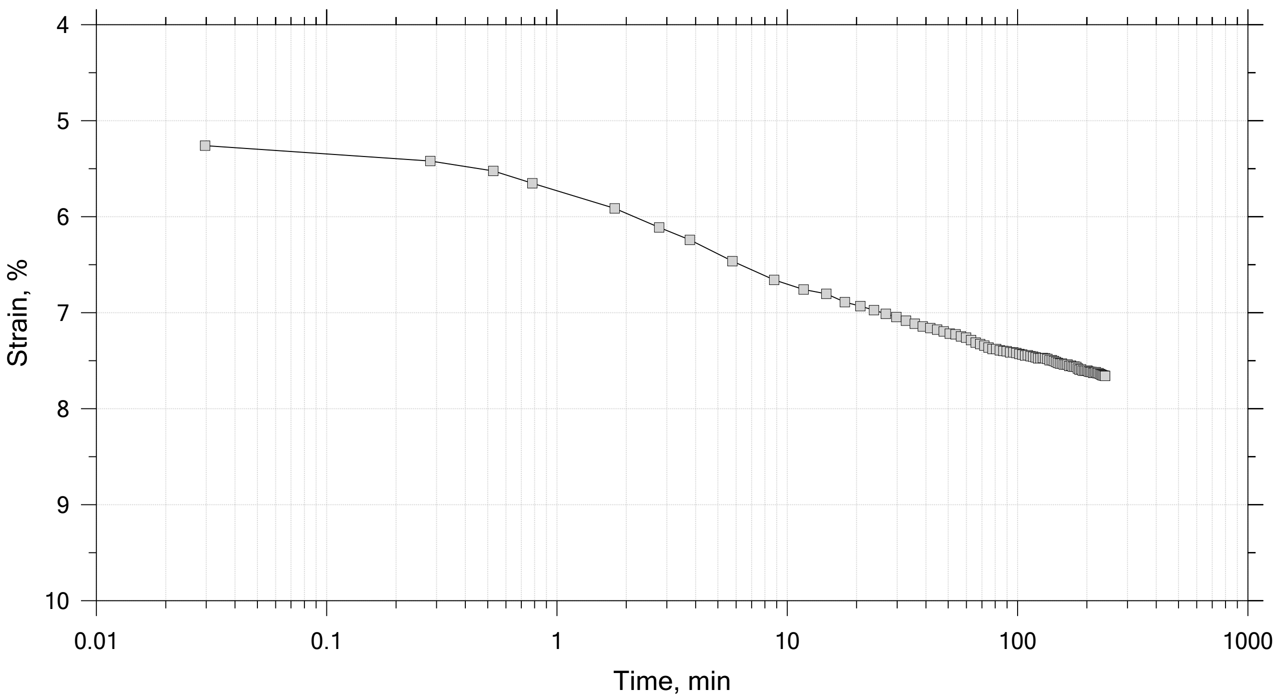
GeoTesting EXPRESS			
	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

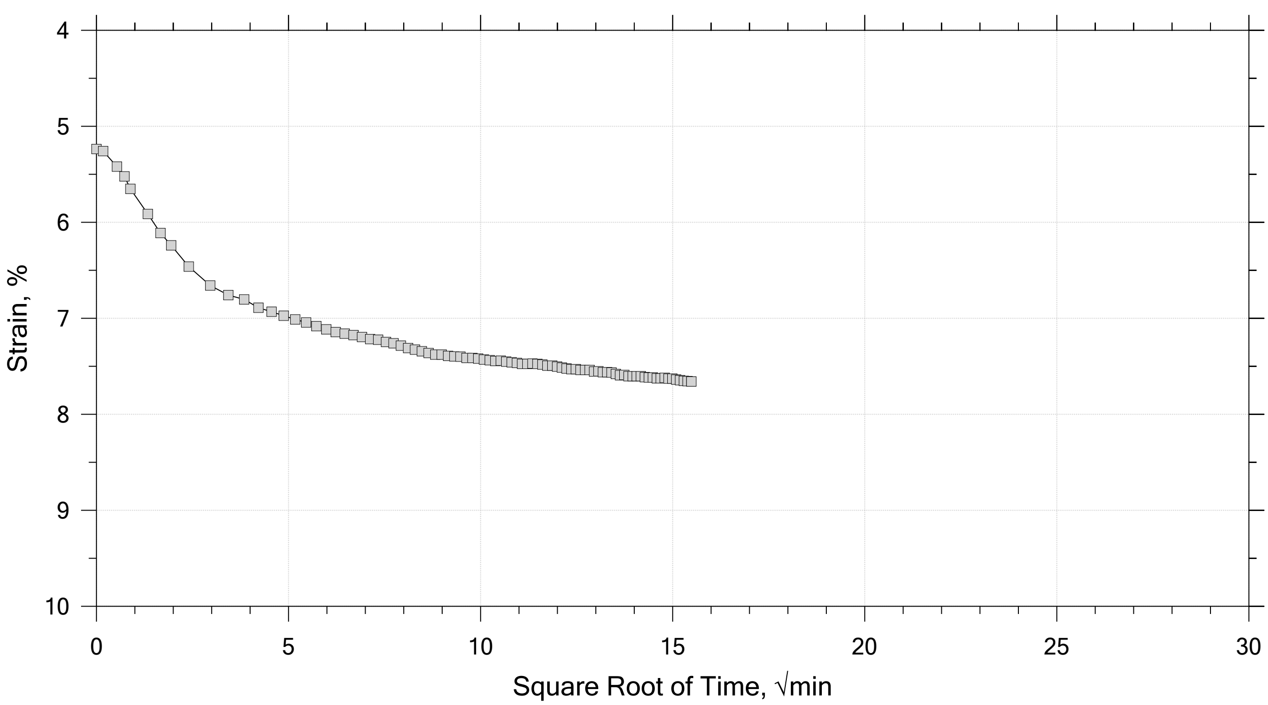
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 15

Constant Load Step

Stress: 2 tsf





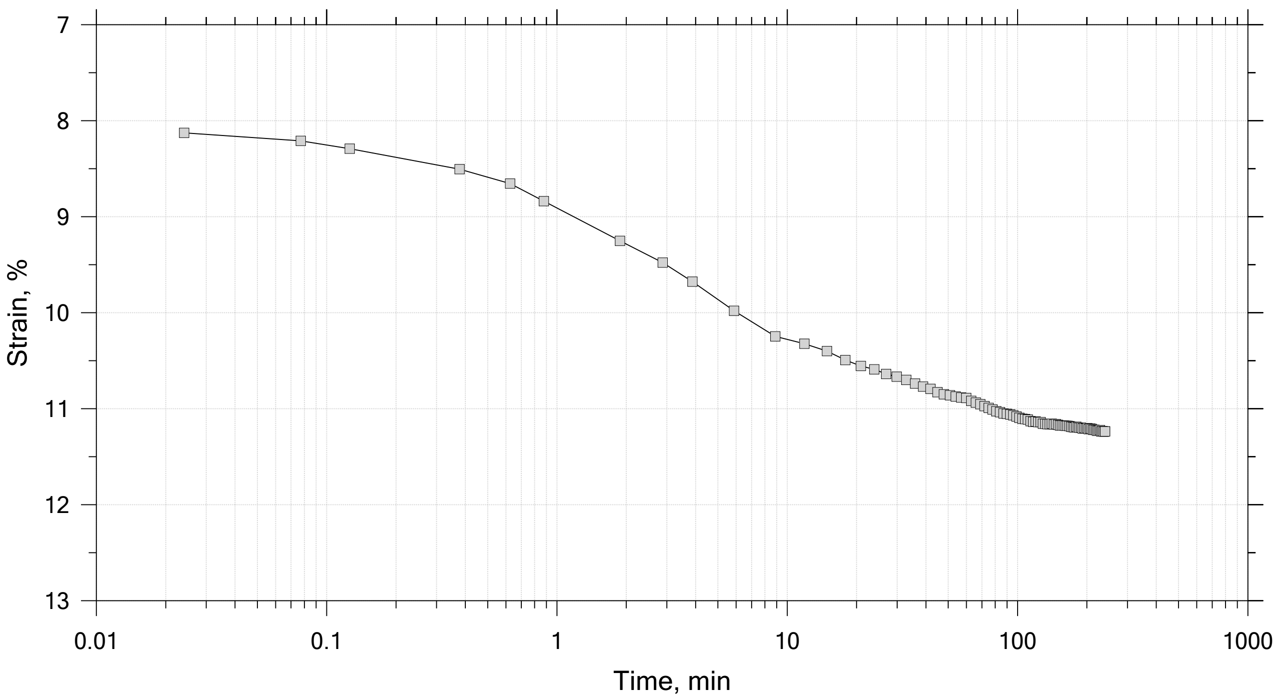
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

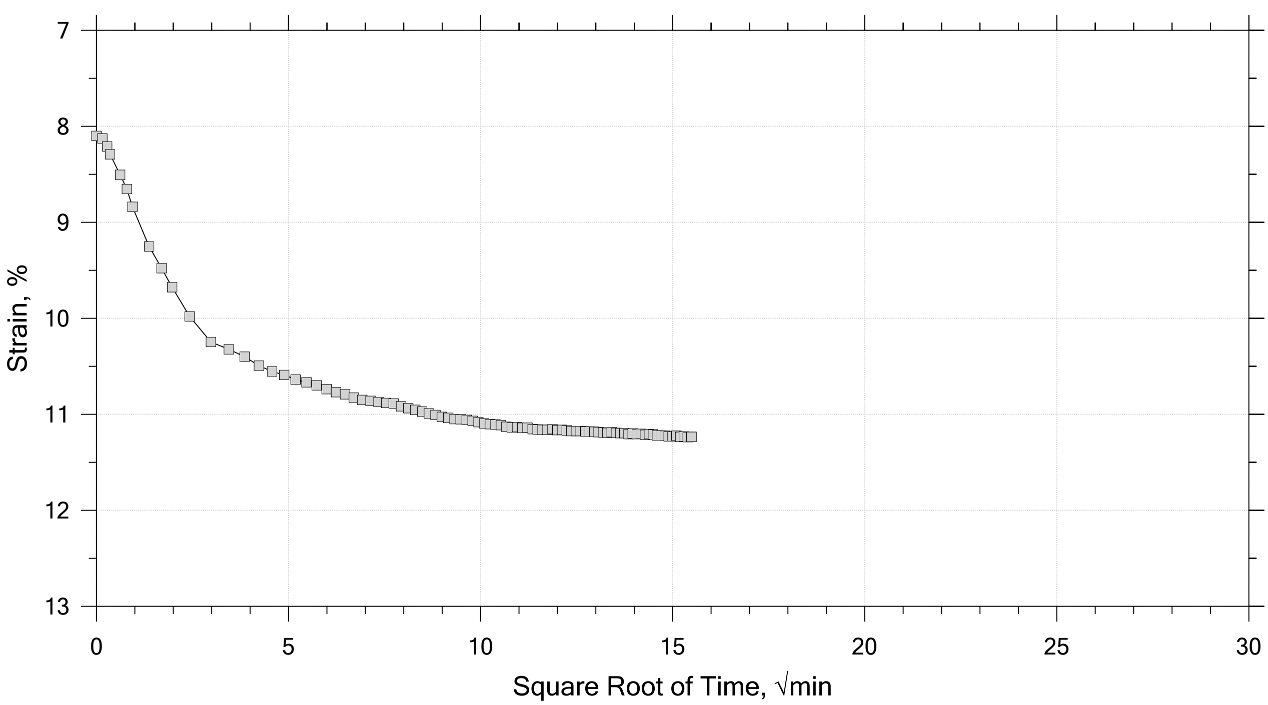
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 15

Constant Load Step

Stress: 4 tsf

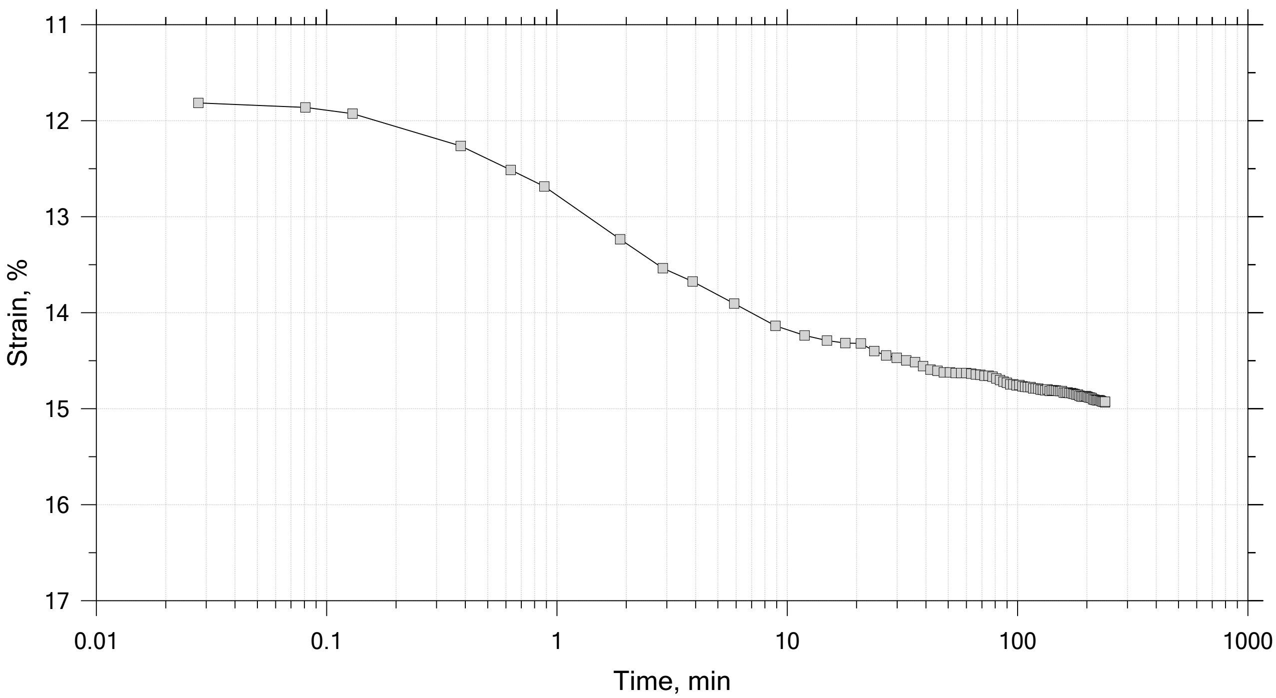


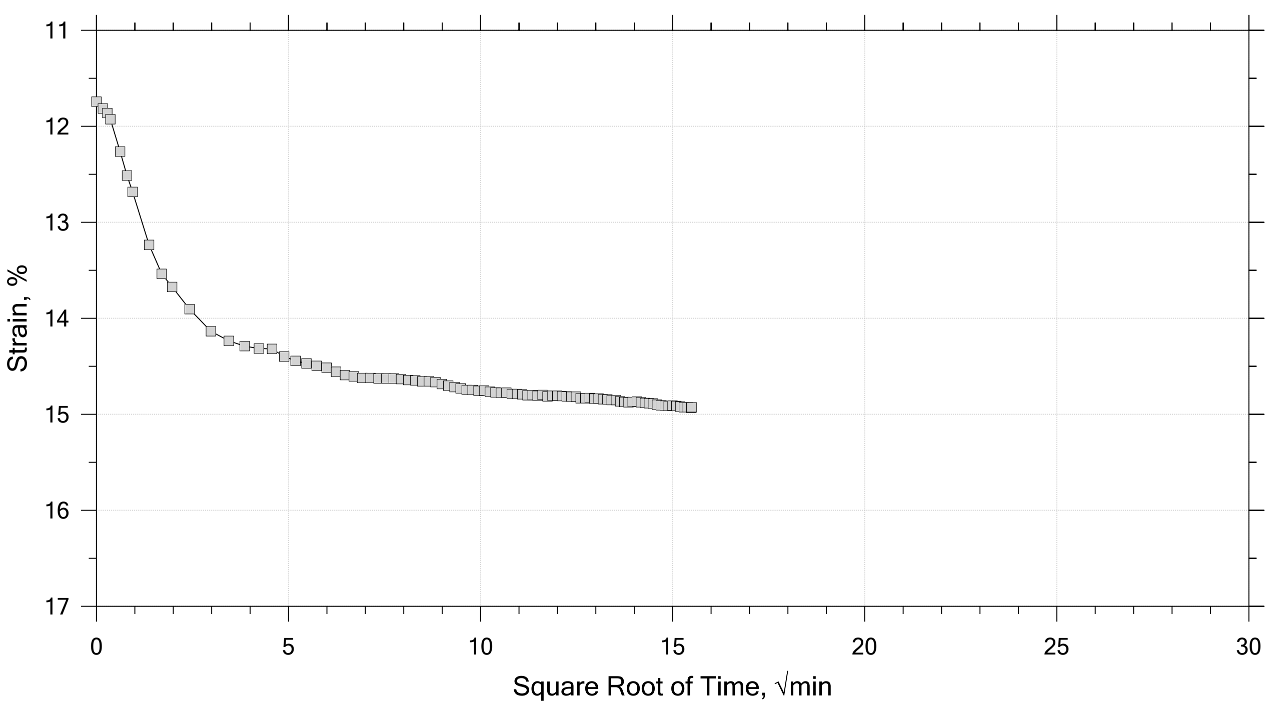


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 15
Constant Load Step
Stress: 8 tsf





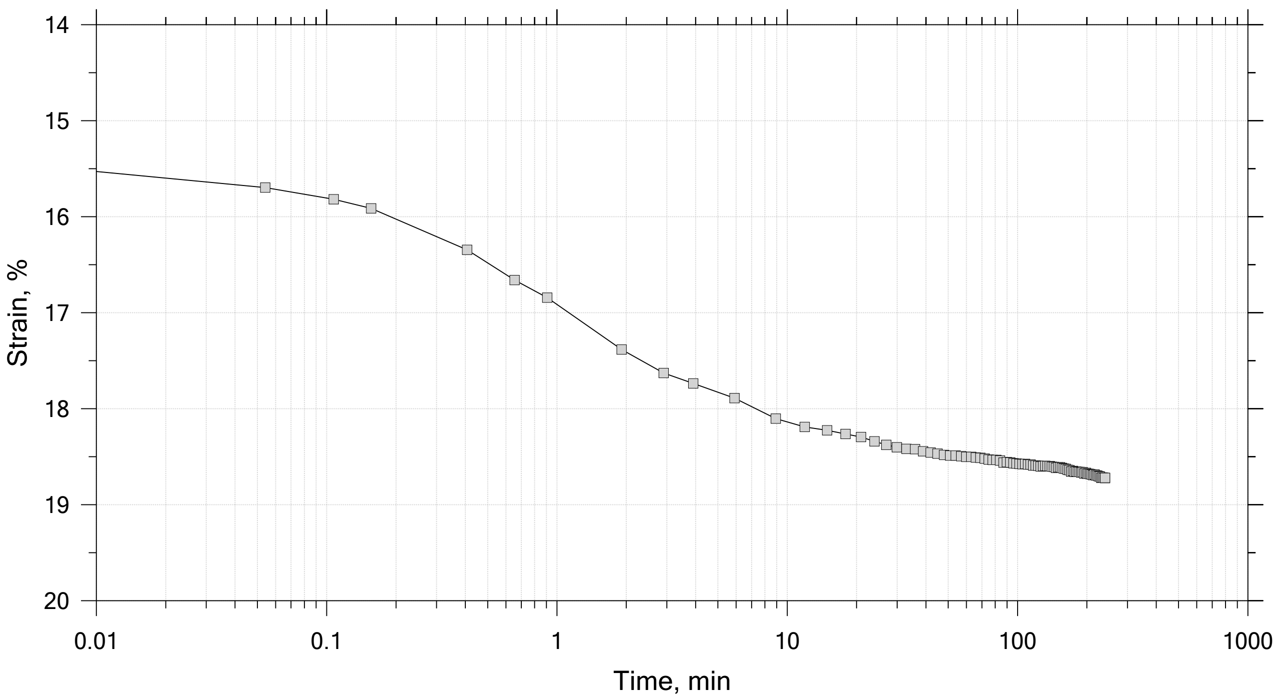
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

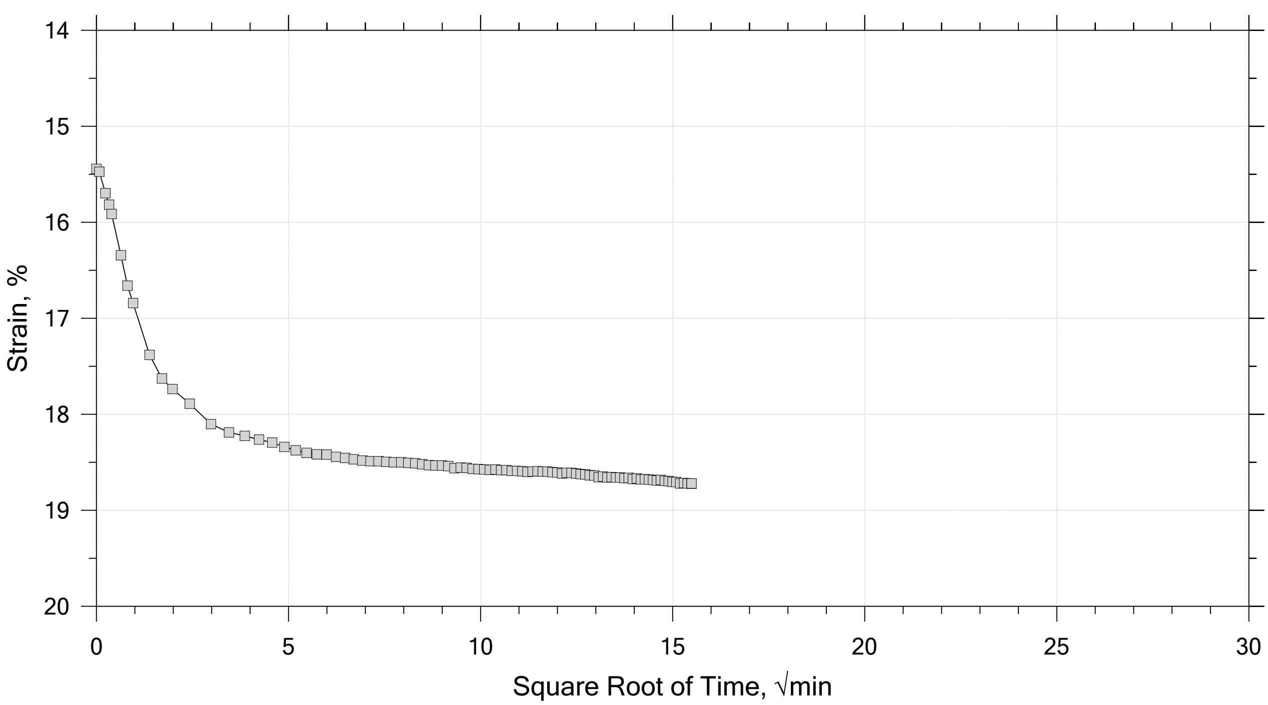
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 15

Constant Load Step

Stress: 16 tsf





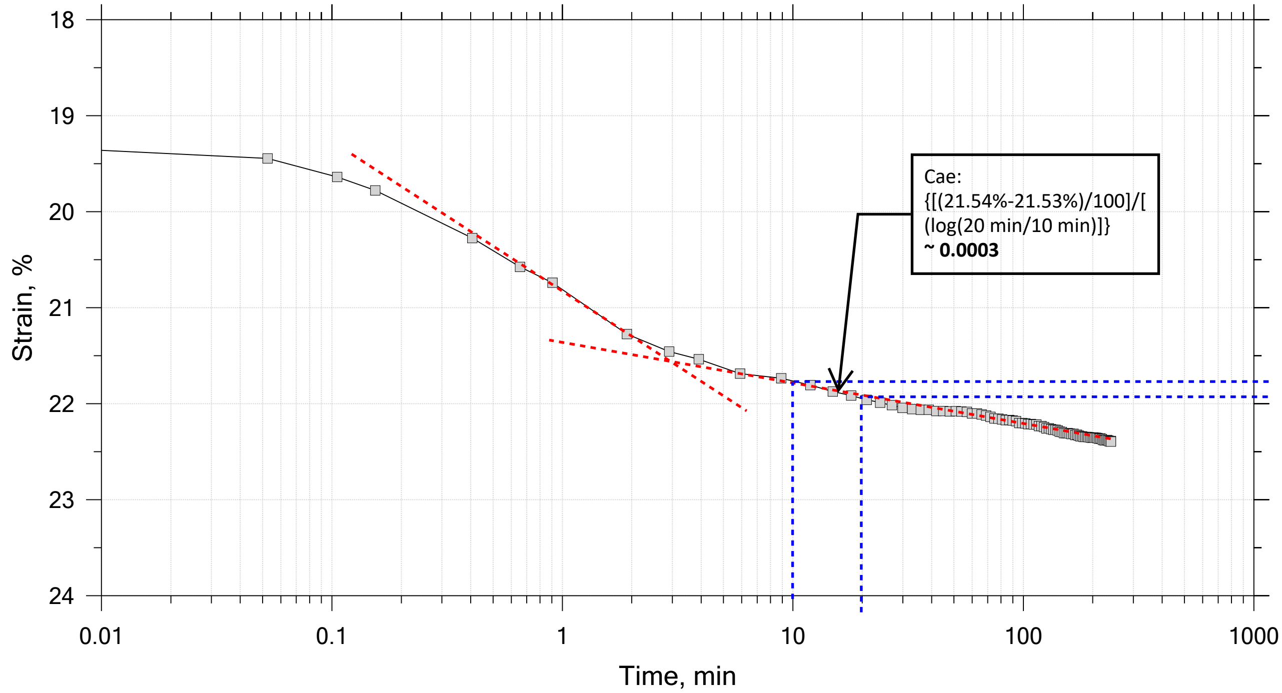
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

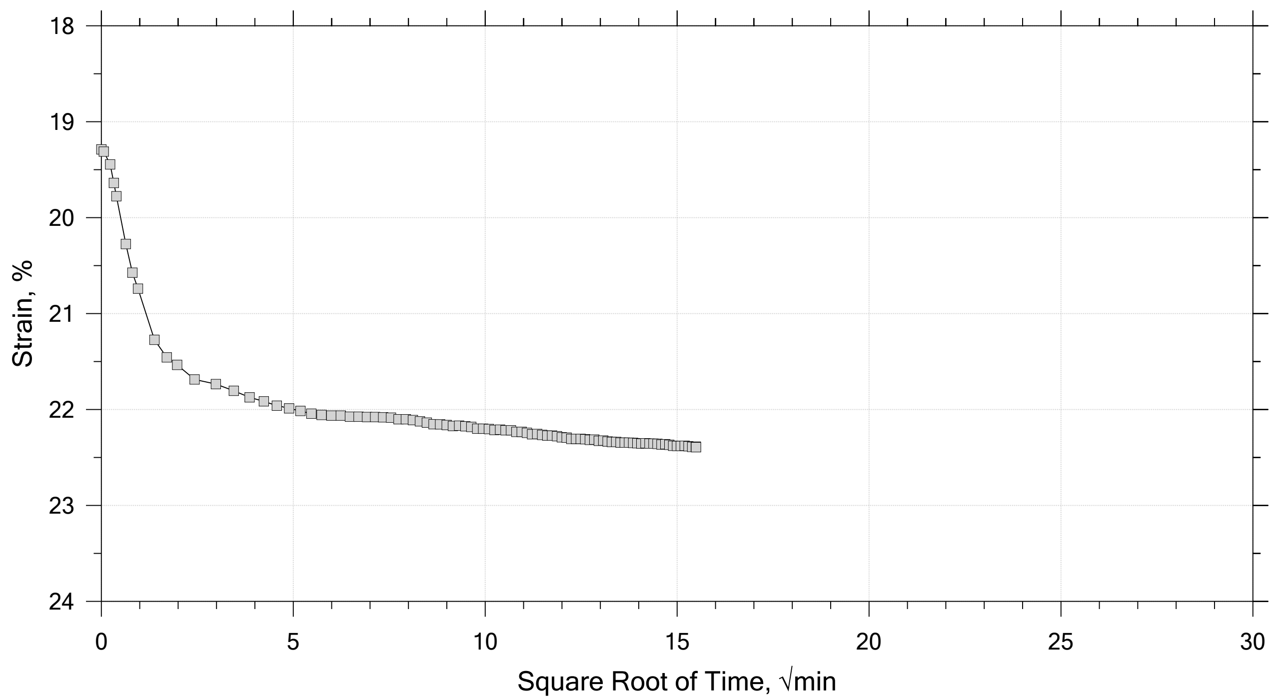
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 15

Constant Load Step

Stress: 32 tsf





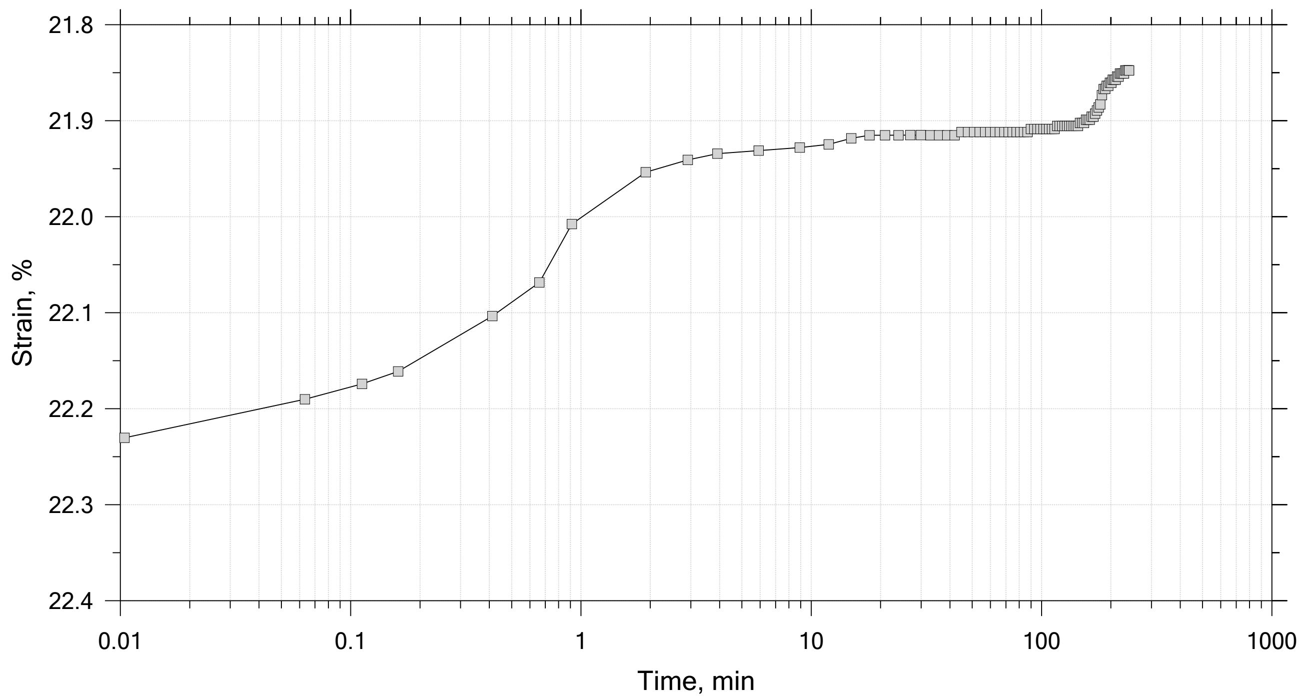
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

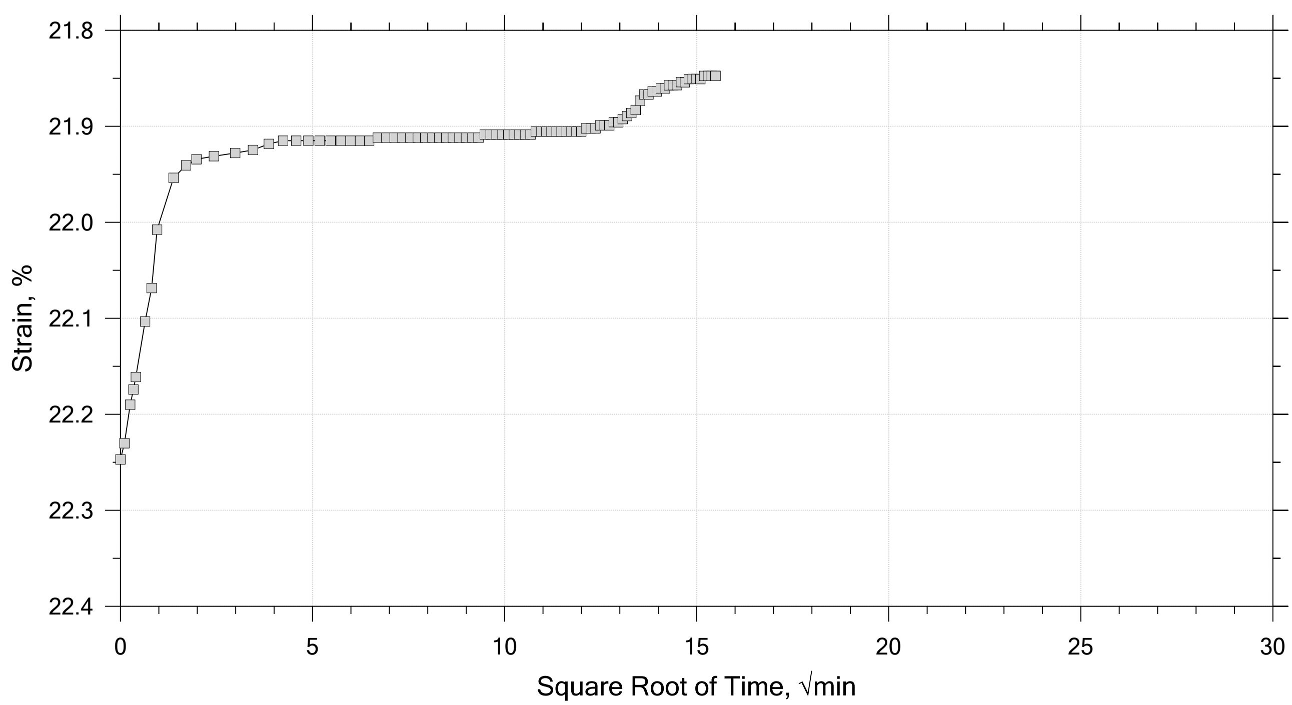
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 15

Constant Load Step

Stress: 8 tsf





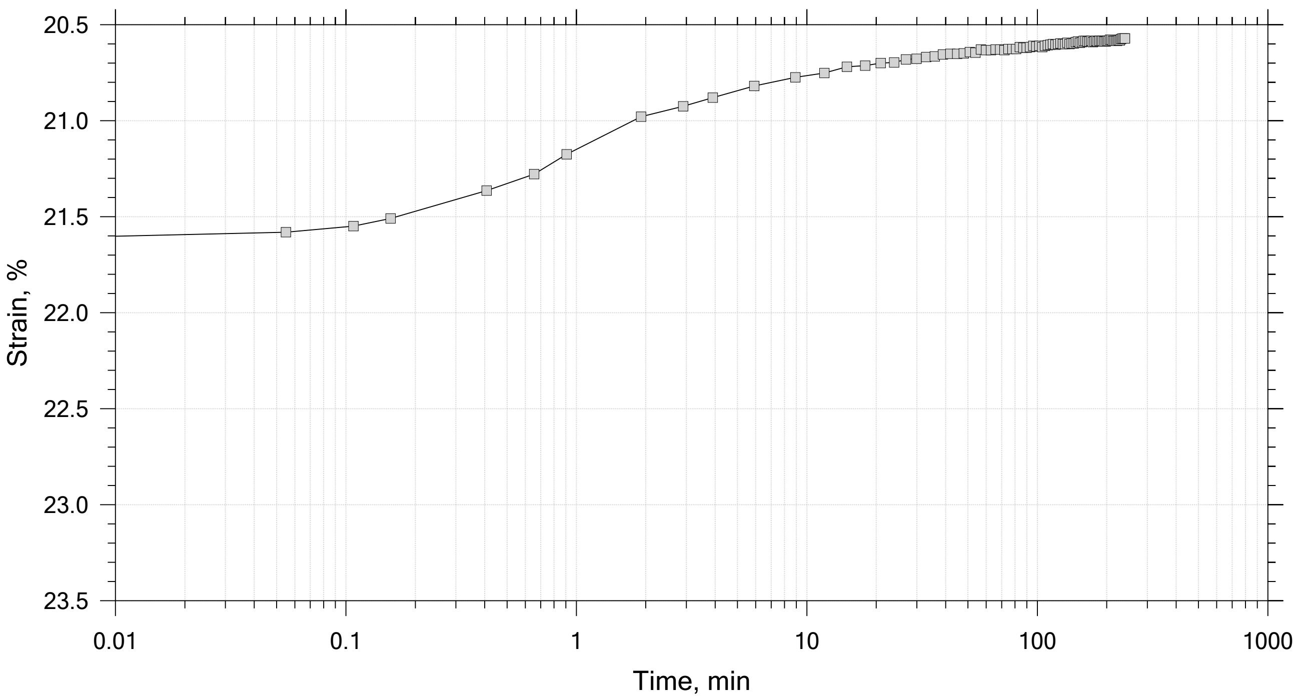
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

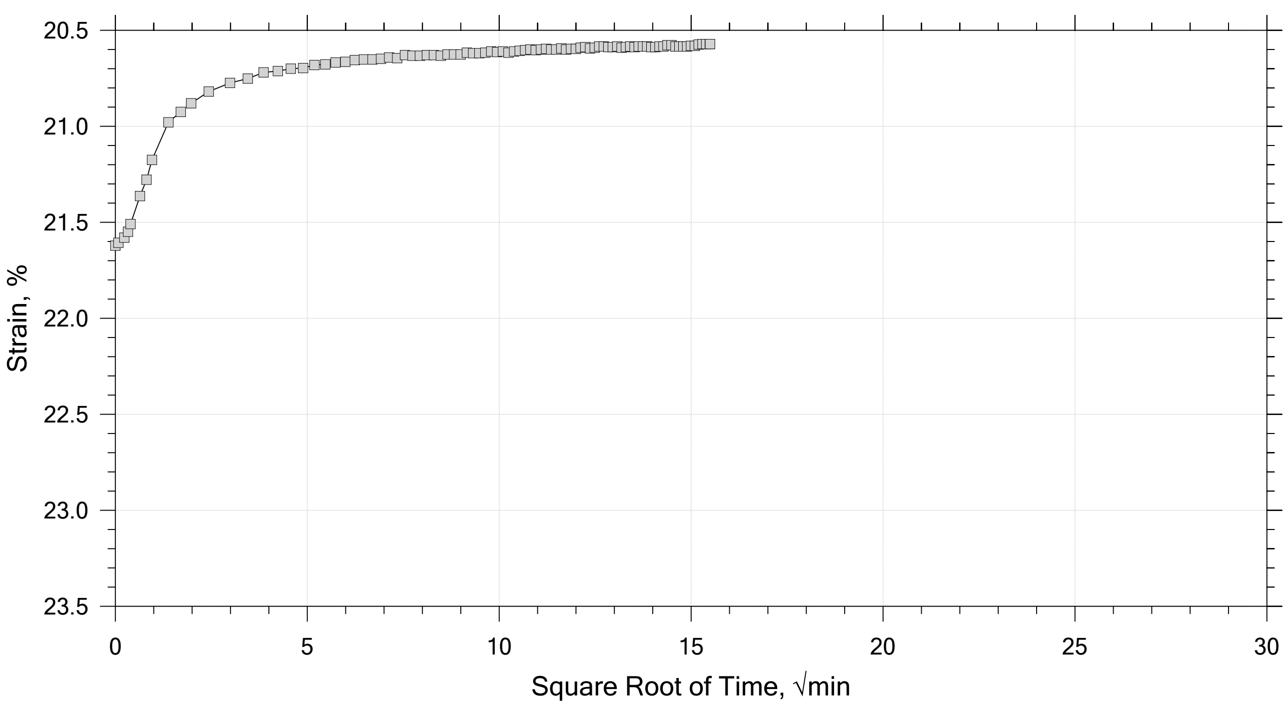
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 15

Constant Load Step

Stress: 2 tsf





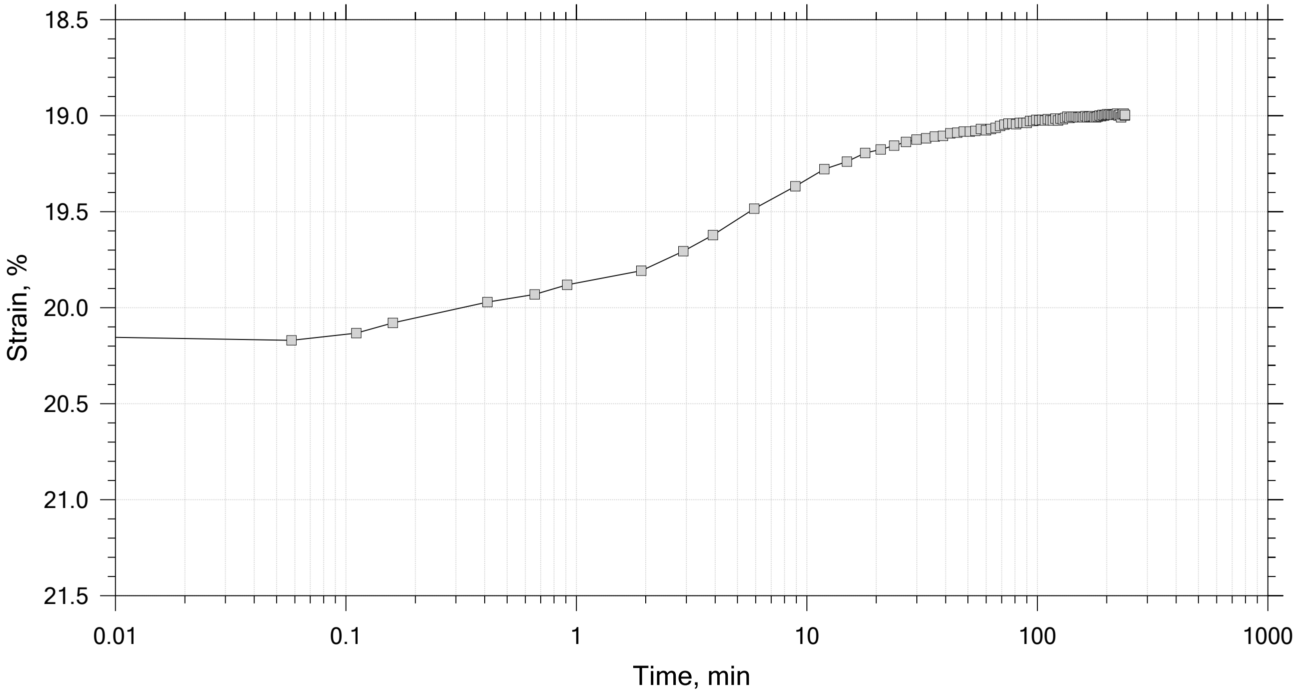
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

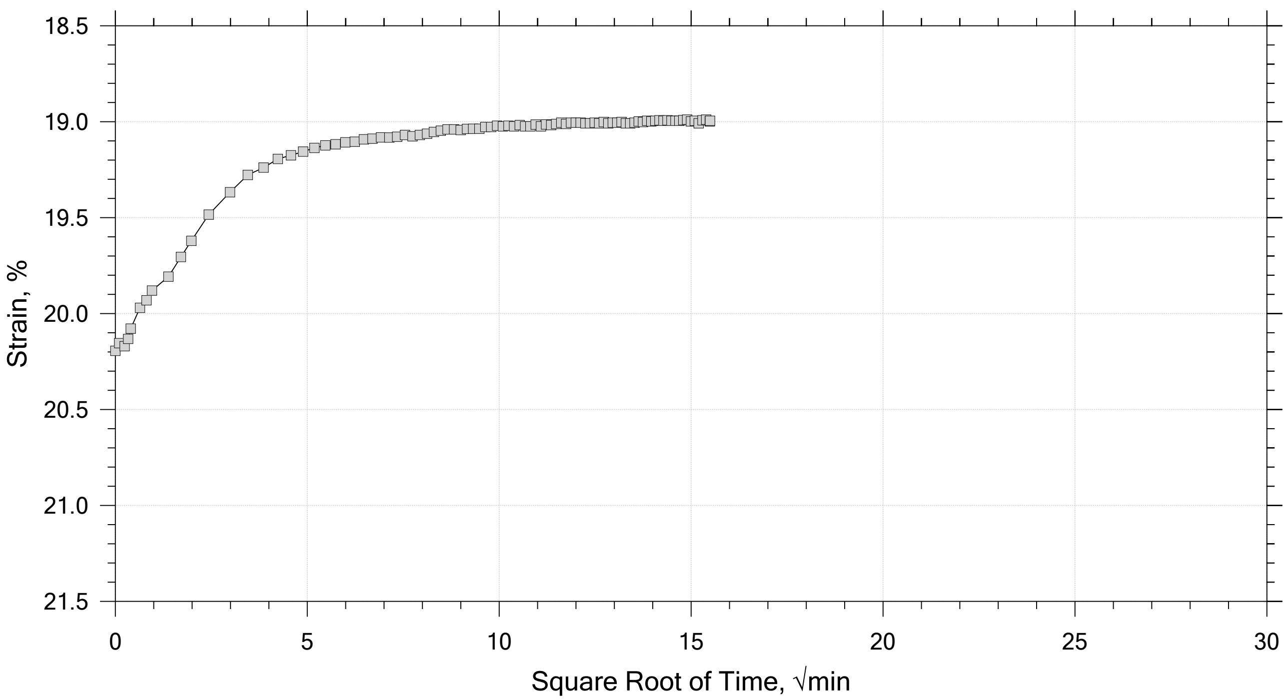
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 15

Constant Load Step

Stress: 0.5 tsf

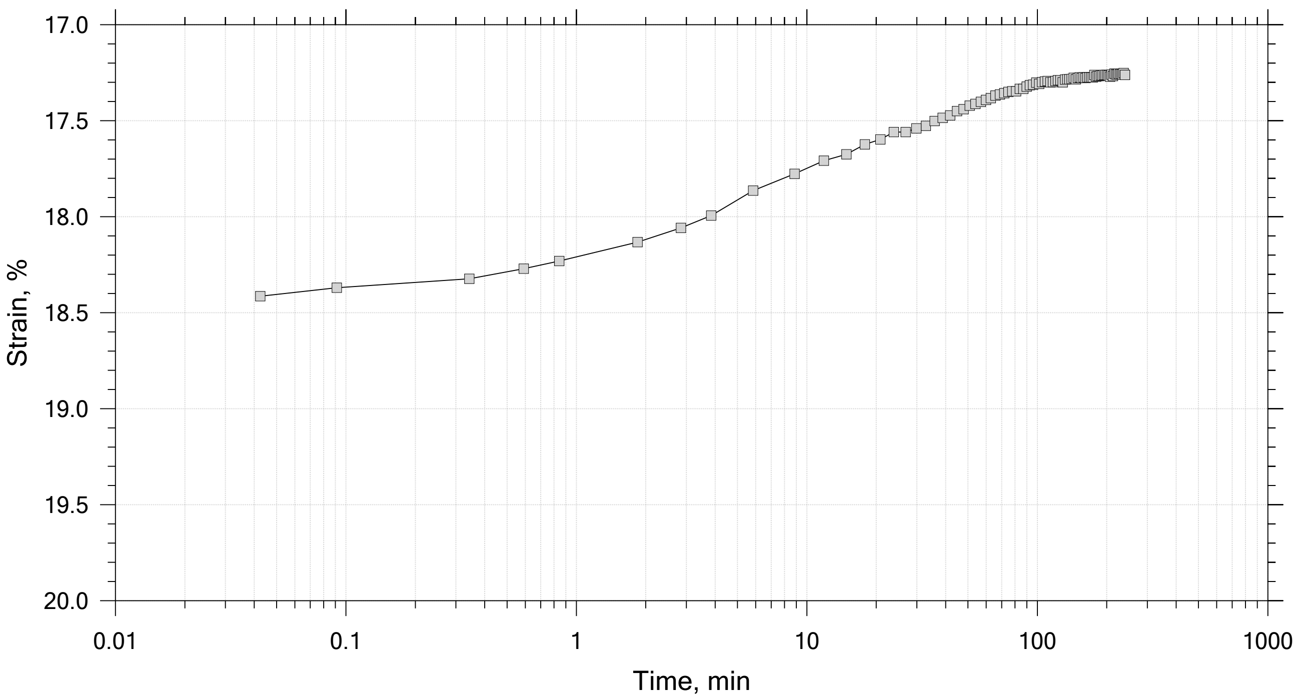


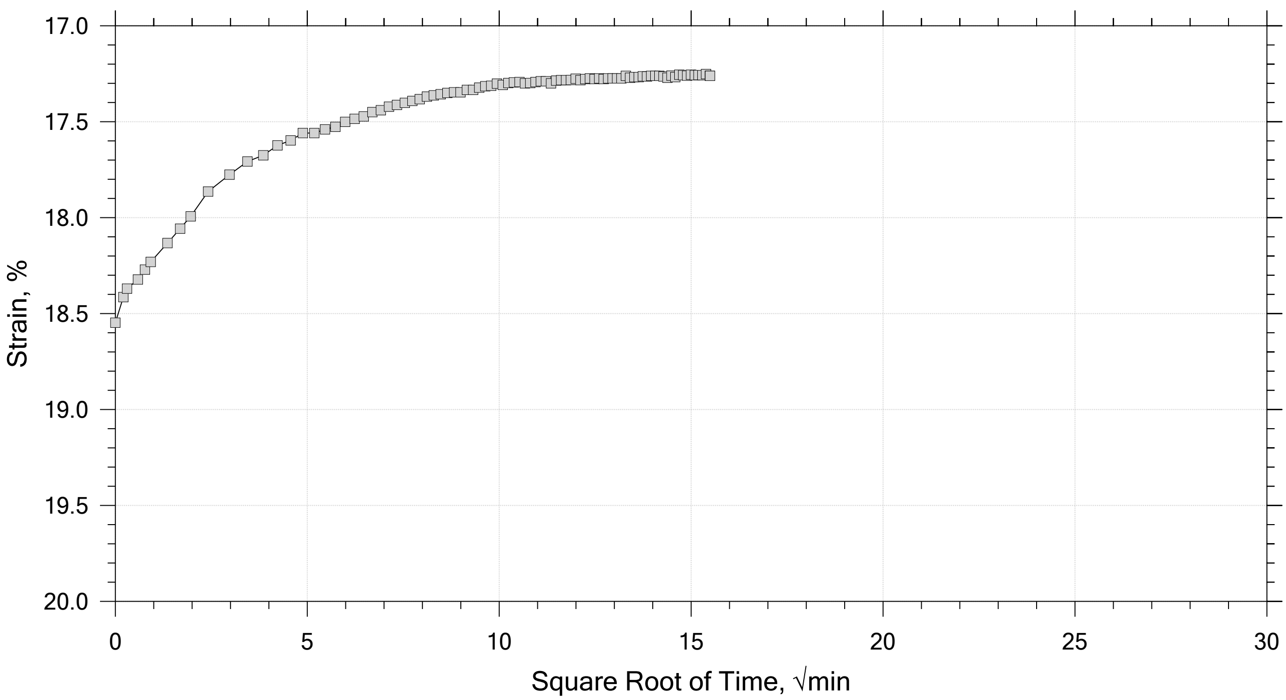


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 15
Constant Load Step
Stress: 0.125 tsf





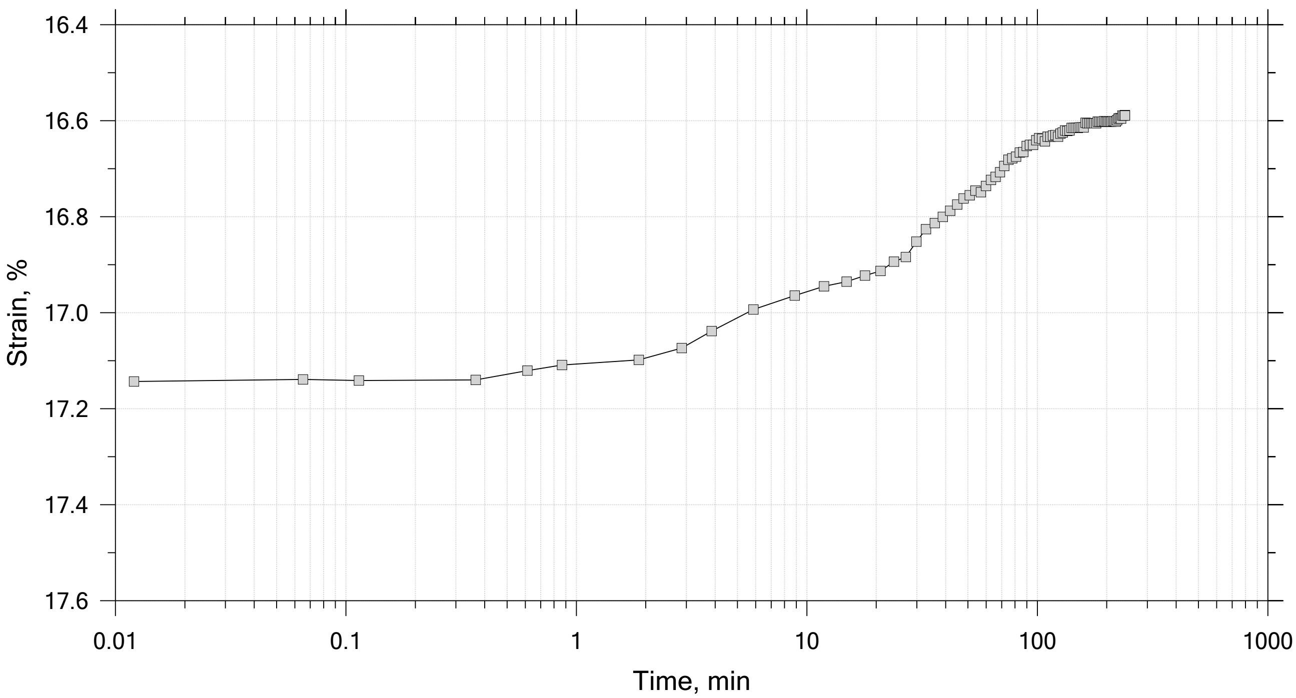
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

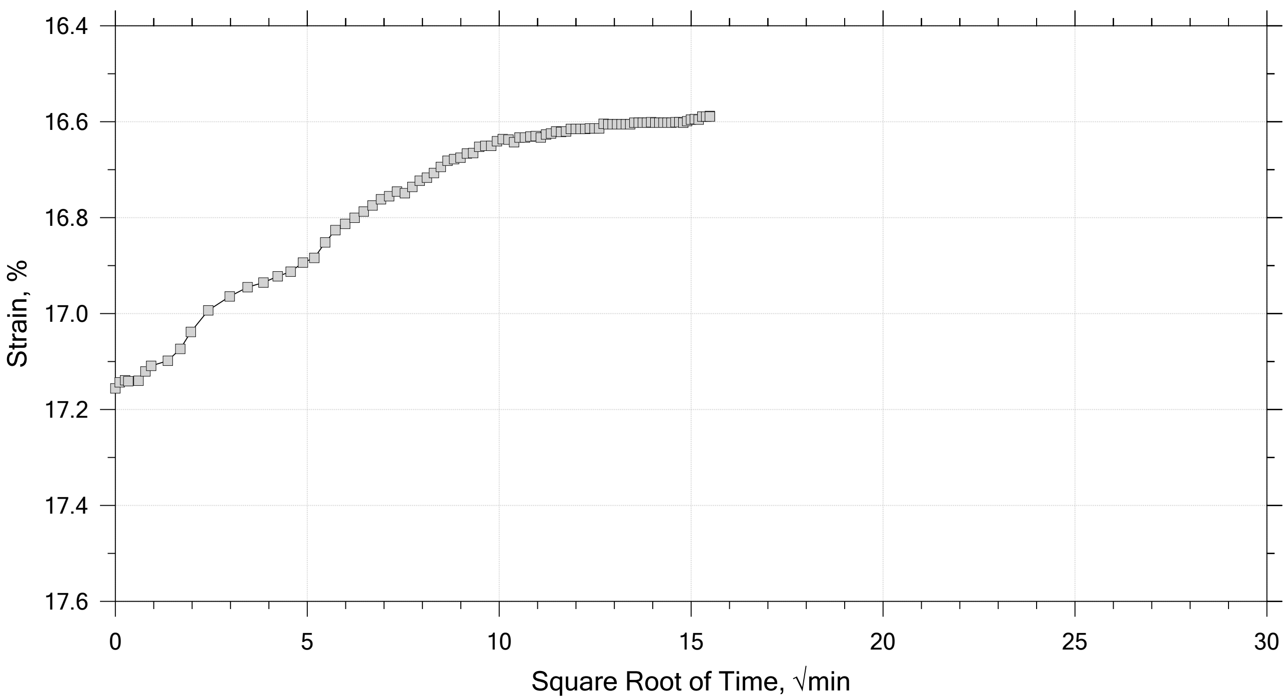
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 15

Constant Load Step

Stress: 0.0625 tsf





GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.77	Liquid Limit: 35
Initial Height: 1.00 in	Initial Void Ratio: 0.9	Plastic Limit: 19
Final Height: 0.85 in	Final Void Ratio: 0.615	Plasticity Index: 16

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	D-745	RING		D-1069
Mass Container, gm	8.38	109.71	109.71	8.29
Mass Container + Wet Soil, gm	336.34	264.28	253	150.91
Mass Container + Dry Soil, gm	256.25	226.97	226.97	125
Mass Dry Soil, gm	247.87	117.26	117.26	116.71
Water Content, %	32.31	31.82	22.20	22.20
Void Ratio	---	0.90	0.61	---
Degree of Saturation, %	---	97.94	100.00	---
Dry Unit Weight, pcf	---	91.002	107.06	---

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.

GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Square Root of Time Coefficients

Cvr (recompression) = 9.48e-05 in2/sec (~0.05688 ft2/day)

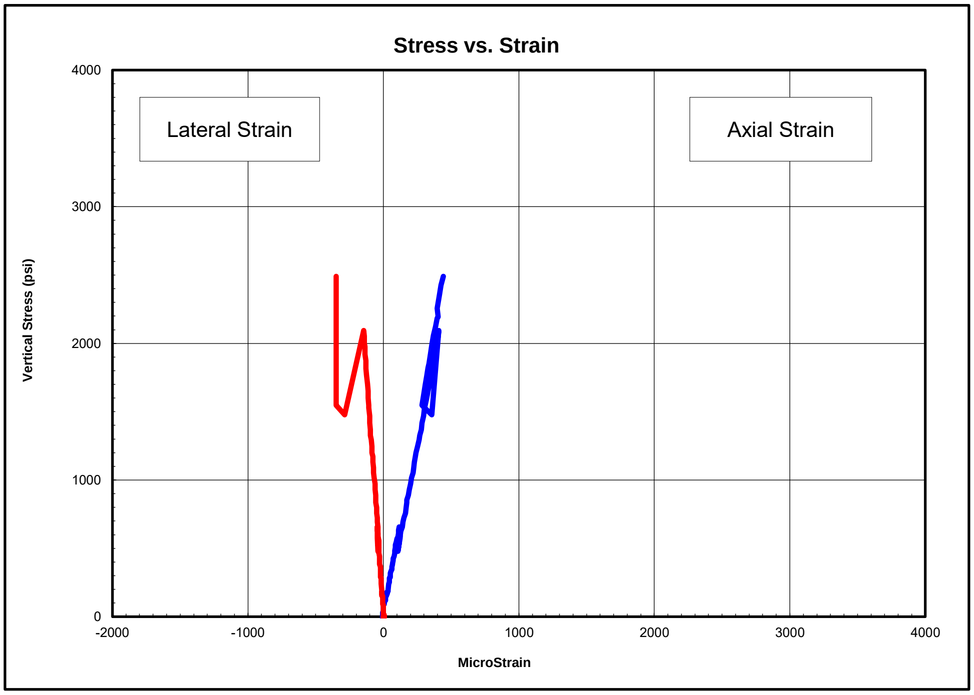
Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.0958	0.007203	0.886	0.720	249.560	1.41e-05	7.52e-02	1.98e-05
2	0.125	0.008819	0.883	0.882	65.922	5.27e-05	5.53e-02	5.46e-05
3	0.250	0.01605	0.869	1.60	28.908	1.19e-04	5.78e-02	1.29e-04
4	0.500	0.03000	0.843	3.00	35.576	9.48e-05	5.58e-02	9.91e-05
5	1.00	0.04852	0.808	4.85	31.720	1.03e-04	3.70e-02	7.13e-05
6	2.00	0.07658	0.754	7.66	31.074	9.99e-05	2.81e-02	5.25e-05
7	4.00	0.1124	0.686	11.2	23.209	1.25e-04	1.79e-02	4.18e-05
8	8.00	0.1493	0.616	14.9	20.407	1.31e-04	9.23e-03	2.26e-05
9	16.0	0.1872	0.544	18.7	22.614	1.08e-04	4.74e-03	9.60e-06
10	32.0	0.2239	0.474	22.4	22.276	1.00e-04	2.30e-03	4.31e-06
11	8.00	0.2185	0.485	21.8	181.159	1.18e-05	2.28e-04	5.04e-08
12	2.00	0.2057	0.509	20.6	22.397	9.79e-05	2.13e-03	3.90e-06
13	0.500	0.1900	0.539	19.0	27.557	8.25e-05	1.05e-02	1.62e-05
14	0.125	0.1726	0.572	17.3	29.496	8.03e-05	4.62e-02	6.96e-05
15	0.0625	0.1659	0.585	16.6	115.940	2.10e-05	1.07e-01	4.23e-05

Cv (virgin) = 1.31e-04 in2/sec (~0.0786 ft2/day)

GeoTesting EXPRESS			
	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B, Swell Pressure = 0.0958 tsf		
	Displacement at End of Increment		

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Wilson St Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308858
Test Date:	10/1/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BWS-104
Sample ID:	R1
Depth, ft:	41.3-41.9
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 2,490 psi

The strain gauges picked up an initial failure within the specimen and then continued reading until total failure occured.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
200-900	4,240,000	0.28
900-1600	5,230,000	0.41
1600-2000	5,490,000	0.38

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

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/I-395 Wilson St Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308858		
Boring ID:	BB-BWS-104		
Sample ID:	R1		
Depth:	41.3-41.9 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.38	4.38	4.38		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	594.33				
Bulk Density, lb/ft^3	168			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? 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	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040

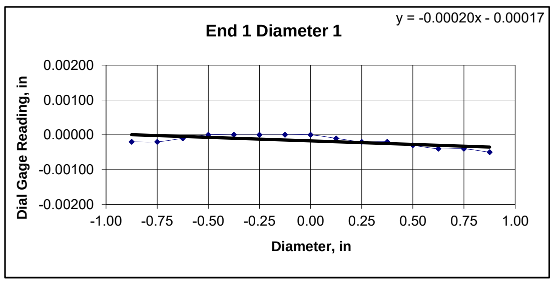
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00050

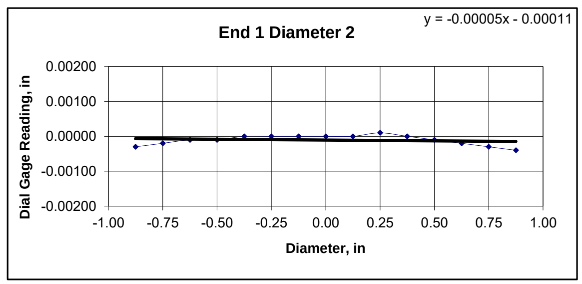
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010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

Difference between max and min readings, in:
0° = 0.0005 90° = 0.0005

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025

Flatness Tolerance Met? **YES**





DIAMETER 1

End 1:

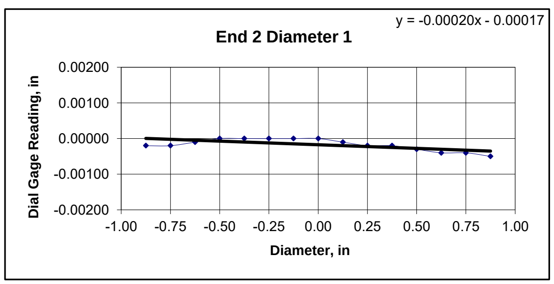
Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01162

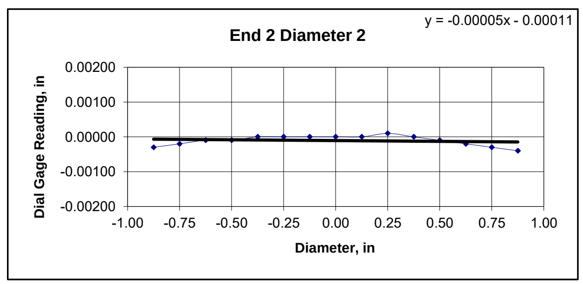
End 2:

Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01162

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00262

End 2:

Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00262

Maximum Angular Difference: 0.00000

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	1.980	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	1.980	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00050	1.980	0.00025	0.014	YES

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

Perpendicularity Tolerance Met? **YES**

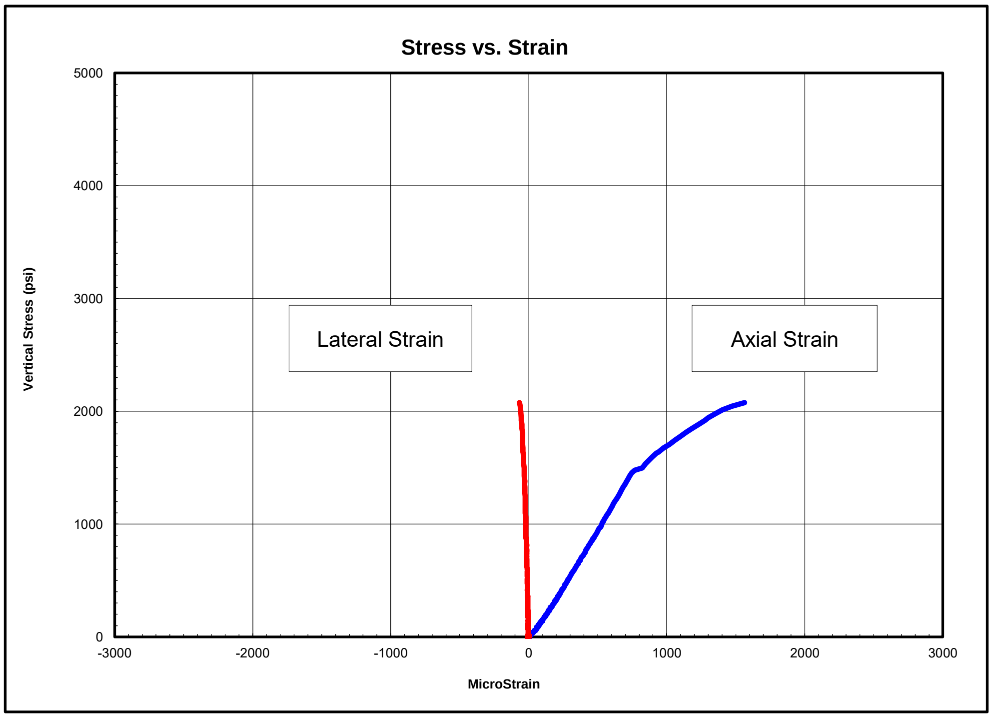
Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Wilson St Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308858
Test Date:	10/1/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BWS-104
Sample ID:	R1
Depth, ft:	41.3-41.9

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-202
Sample ID:	R3
Depth, ft:	79.7-80.2
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 2,077 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
200-700	1,980,000	0.05
700-1300	2,070,000	0.05
1300-1900	986,000	0.04

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

GeoTesting EXPRESS

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-202		
Sample ID:	R3		
Depth:	79.7-80.2 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.22	4.22	4.22
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	597.46		
Bulk Density, lb/ft^3	173	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

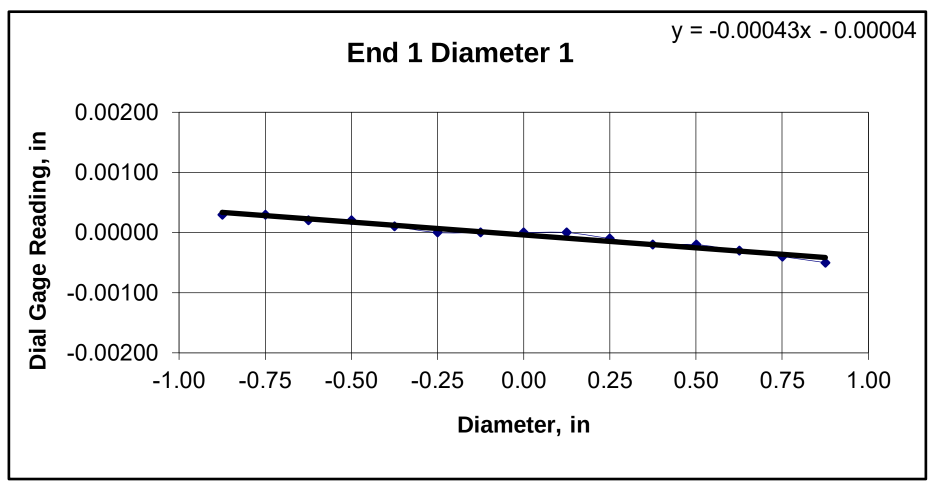
END FLATNESS AND PARALLELISM (Procedure FP1)

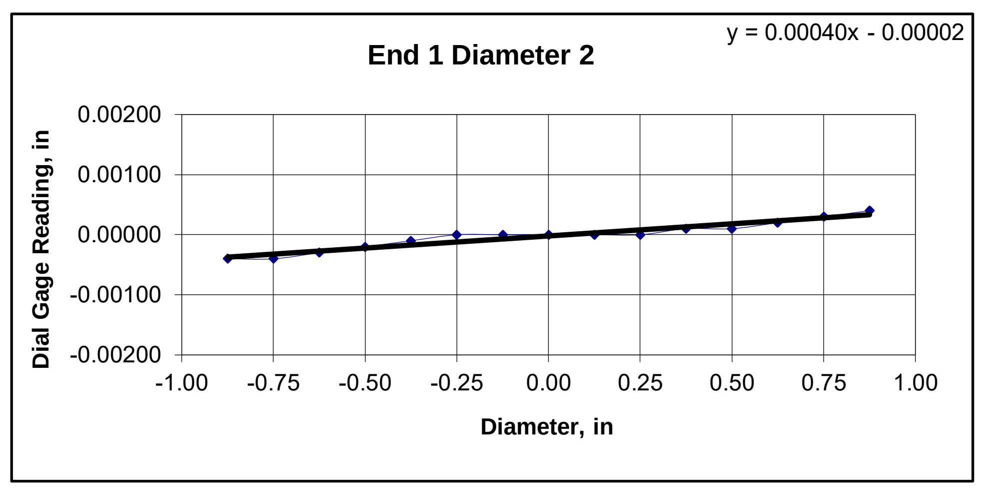
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00030	-0.00020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

Difference between max and min readings, in:
0° = 0.00080 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	0.00050	0.00040	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	0.00010	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040

Difference between max and min readings, in:
0° = 0.001 90° = 0.0008
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00050
Flatness Tolerance Met? **YES**





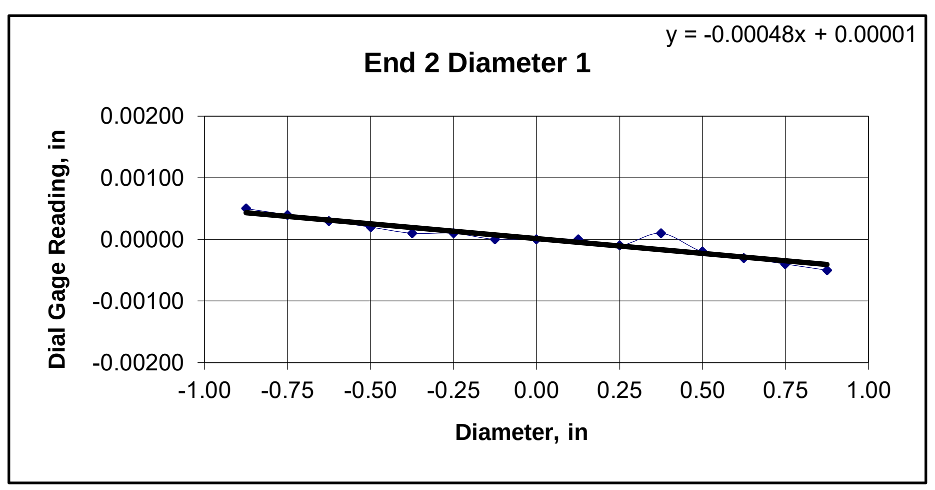
DIAMETER 1

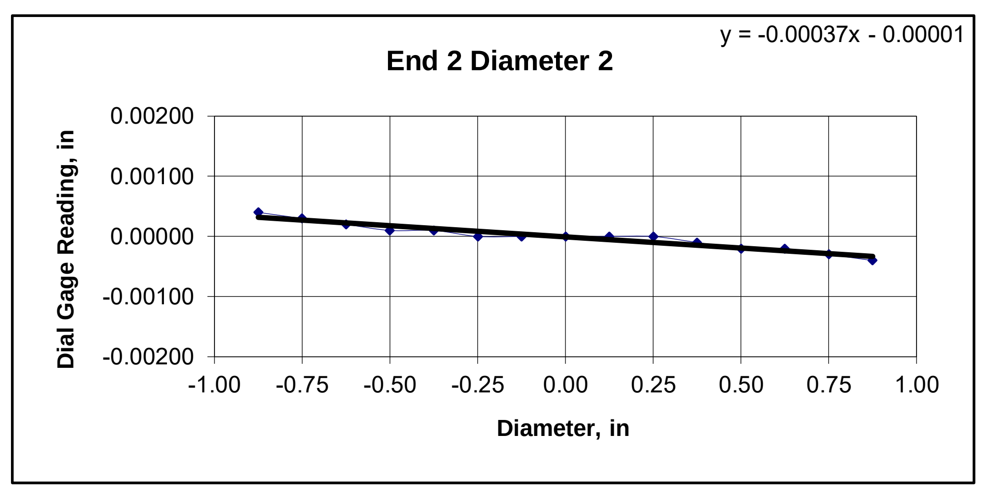
End 1:
Slope of Best Fit Line 0.00043
Angle of Best Fit Line: 0.02456

End 2:
Slope of Best Fit Line 0.00048
Angle of Best Fit Line: 0.02750

Maximum Angular Difference: 0.00295

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00040
Angle of Best Fit Line: 0.02308

End 2:
Slope of Best Fit Line 0.00037
Angle of Best Fit Line: 0.02128

Maximum Angular Difference: 0.00180

Parallelism Tolerance Met? **YES**
Spherically Seated

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

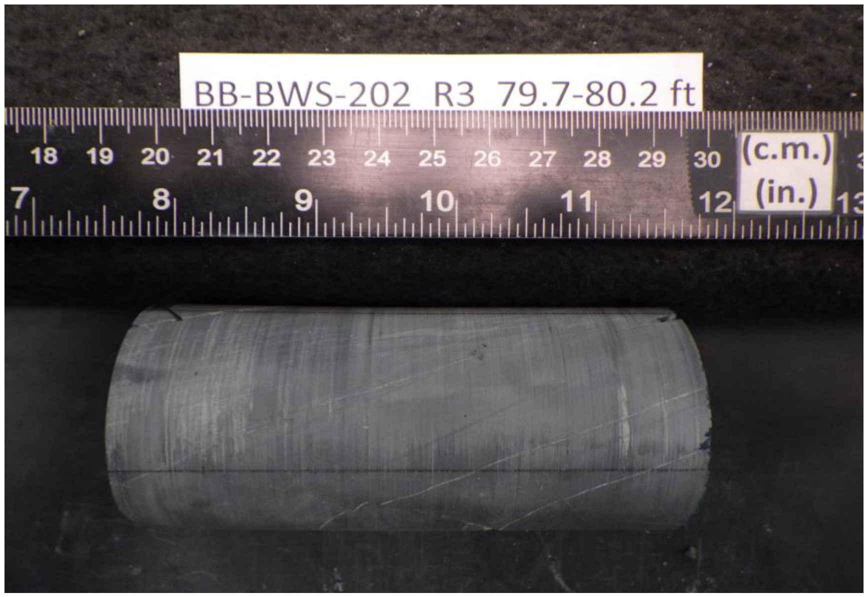
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00080	1.990	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES
END 2					
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES

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

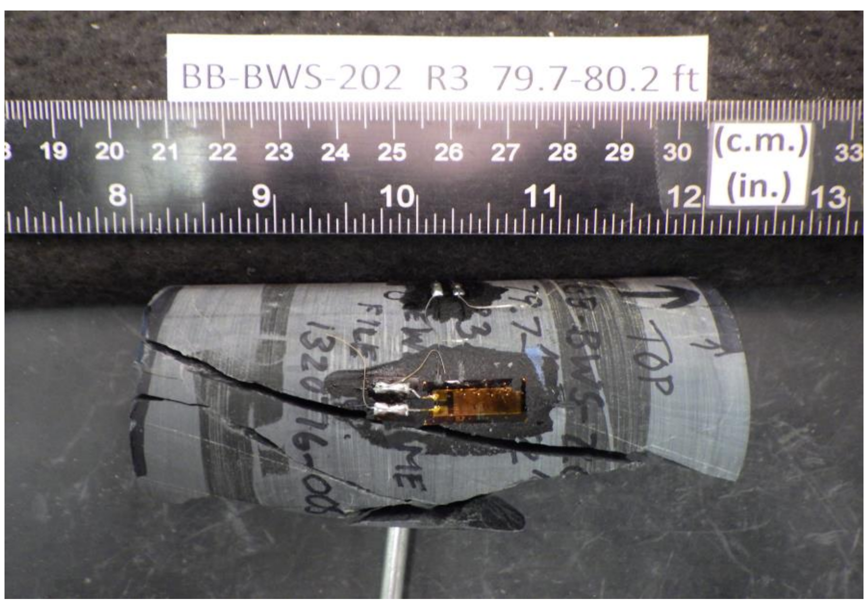
Perpendicularity Tolerance Met? **YES**

GeoTesting EXPRESS

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-202
Sample ID:	R3
Depth, ft:	79.7-80.2



After cutting and grinding

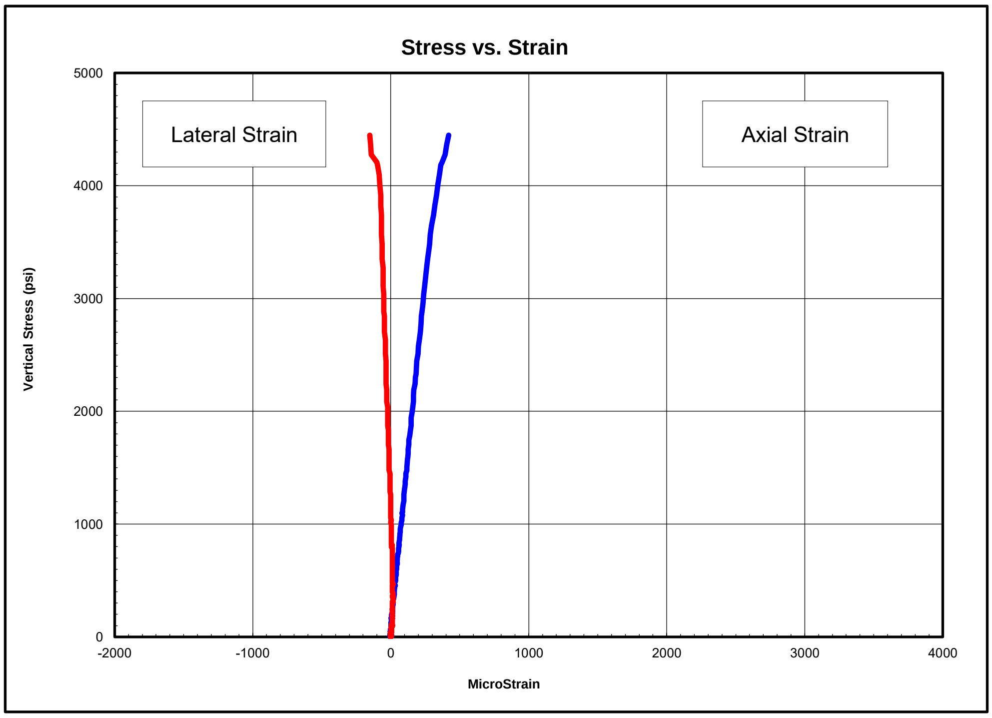


After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-203
Sample ID:	R1
Depth, ft:	45.3-45.9
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 4,448 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
400-1600	12,000,000	0.27
1600-2800	12,300,000	0.36
2800-4000	10,100,000	0.26

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

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-203		
Sample ID:	R1		
Depth:	45.3-45.9 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.53	4.53	4.53
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	612.63		
Bulk Density, lb/ft^3	165	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

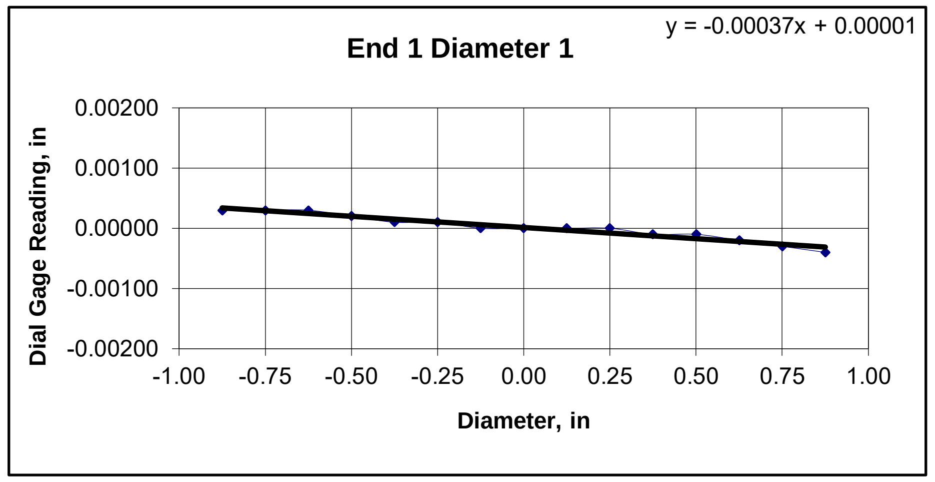
END FLATNESS AND PARALLELISM (Procedure FP1)

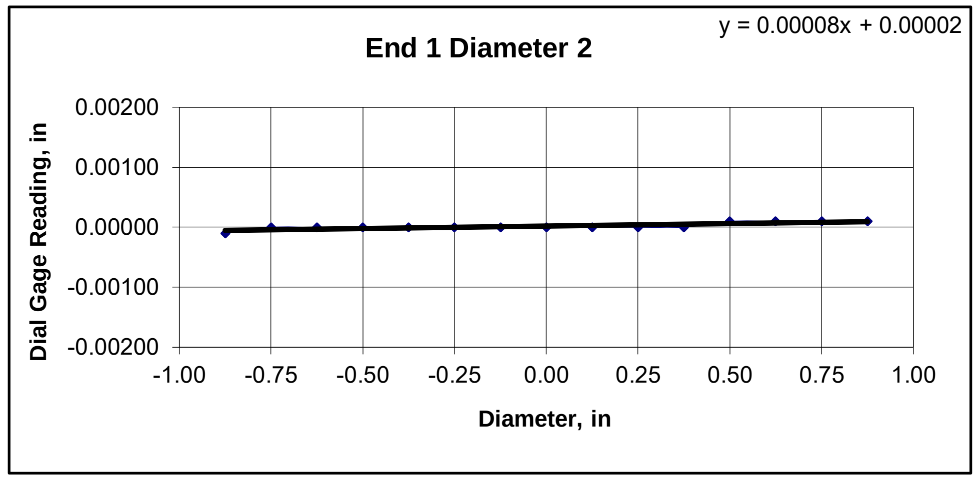
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000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

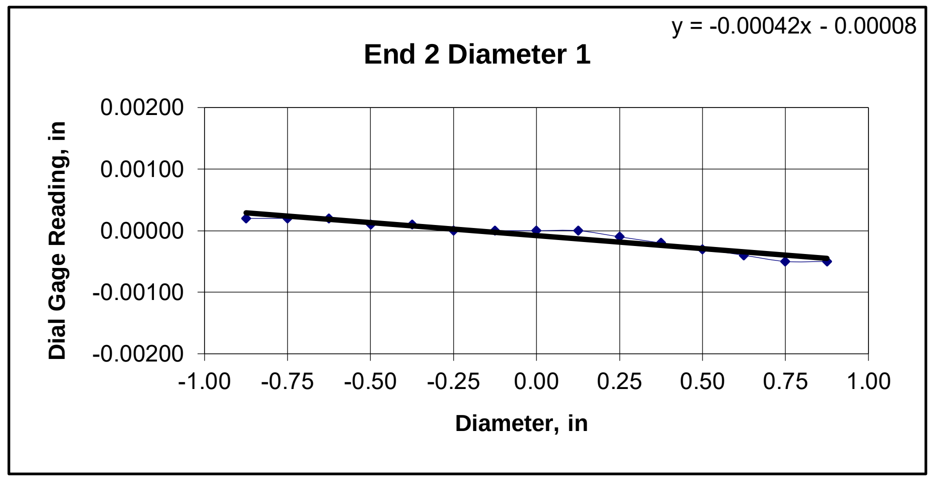
Difference between max and min readings, in:
0° = 0.00070 90° = 0.00020

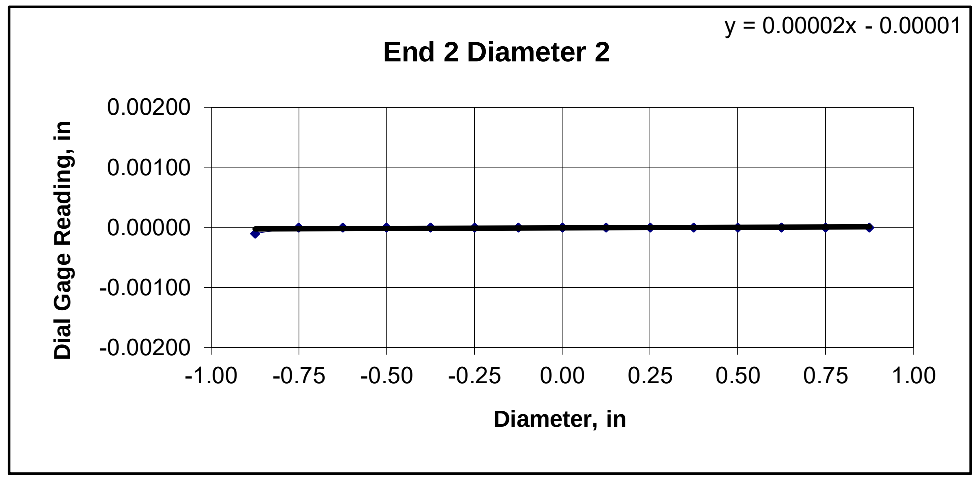
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0.0007 90° = 0.0001
Maximum difference must be < 0.0020 in. Difference = ± 0.00035
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00037
Angle of Best Fit Line: 0.02128

End 2:
Slope of Best Fit Line 0.00042
Angle of Best Fit Line: 0.02423

Maximum Angular Difference: 0.00295

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00008
Angle of Best Fit Line: 0.00475

End 2:
Slope of Best Fit Line 0.00002
Angle of Best Fit Line: 0.00115

Maximum Angular Difference: 0.00360

Parallelism Tolerance Met? YES
Spherically Seated

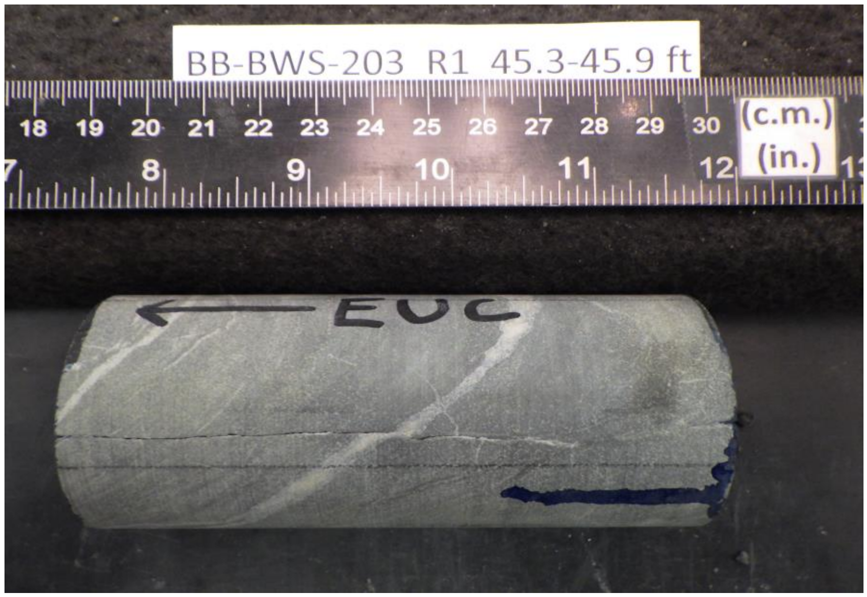
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES

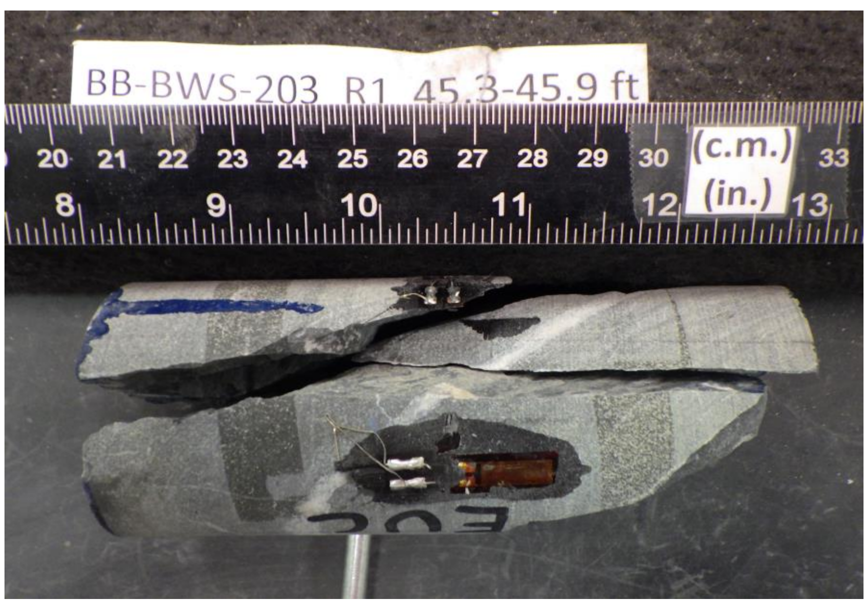
Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-203
Sample ID:	R1
Depth, ft:	45.3-45.9



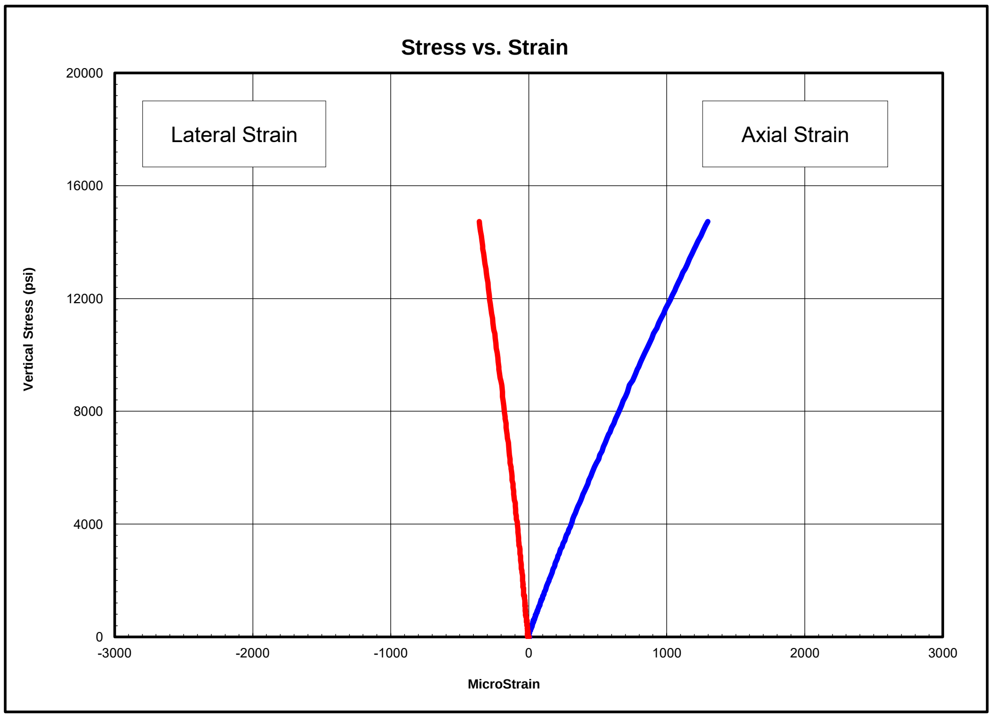
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	jsc
Boring ID:	BB-BWS-206
Sample ID:	R1
Depth, ft:	79.8-80.5
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 14,729 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1500-5400	12,400,000	0.26
5400-9300	11,300,000	0.27
9300-13300	10,300,000	0.28

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

Client:	Haley Aldrich, Inc.	Test Date:	2/17/2020
Project Name:	Rte 9/I-396 Conn. - Wilson St Bridge	Tested By:	cmh/kdp
Project Location:	Brewer Eddington, ME	Checked By:	smd
GTX #:	311345		
Boring ID:	BB-BWS-206		
Sample ID:	R1		
Depth:	79.8-80.5 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.55	4.56	4.56
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	630.45		
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

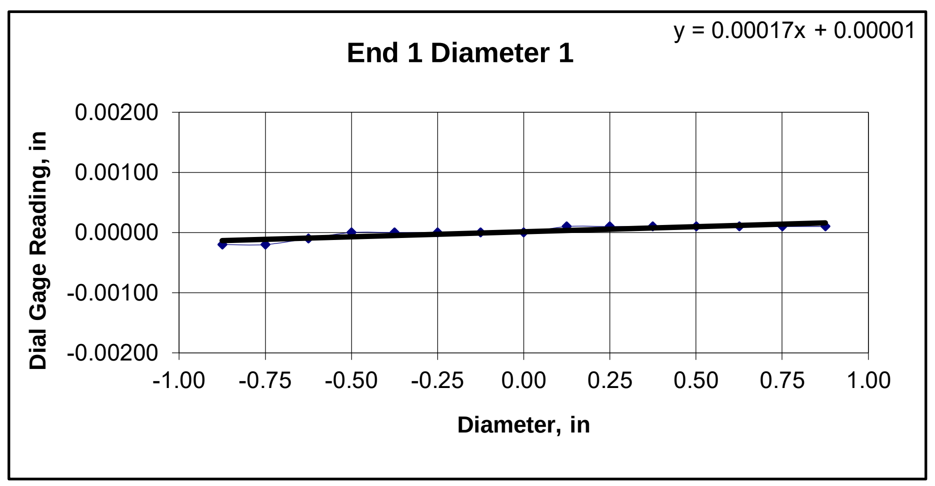
END FLATNESS AND PARALLELISM (Procedure FP1)

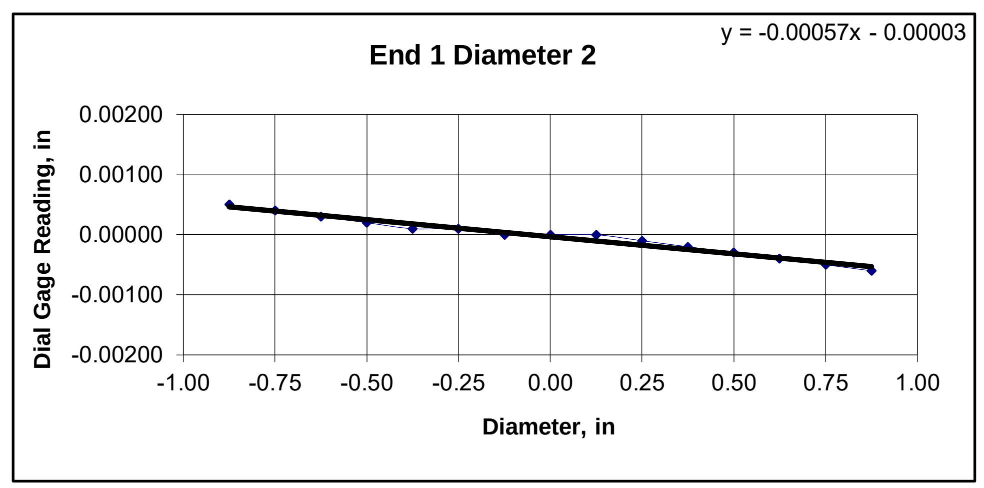
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00050	0.00040	0.00030	0.00020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

Difference between max and min readings, in:
0° = 0.00030 90° = 0.00110

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

Difference between max and min readings, in:
0° = 0.0004 90° = 0.0012
Maximum difference must be < 0.0020 in. Difference = ± 0.00060
Flatness Tolerance Met? YES





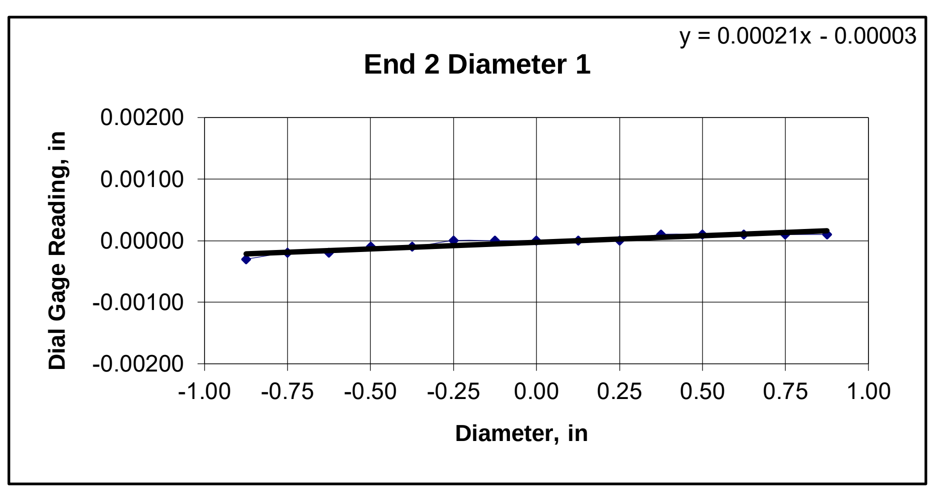
DIAMETER 1

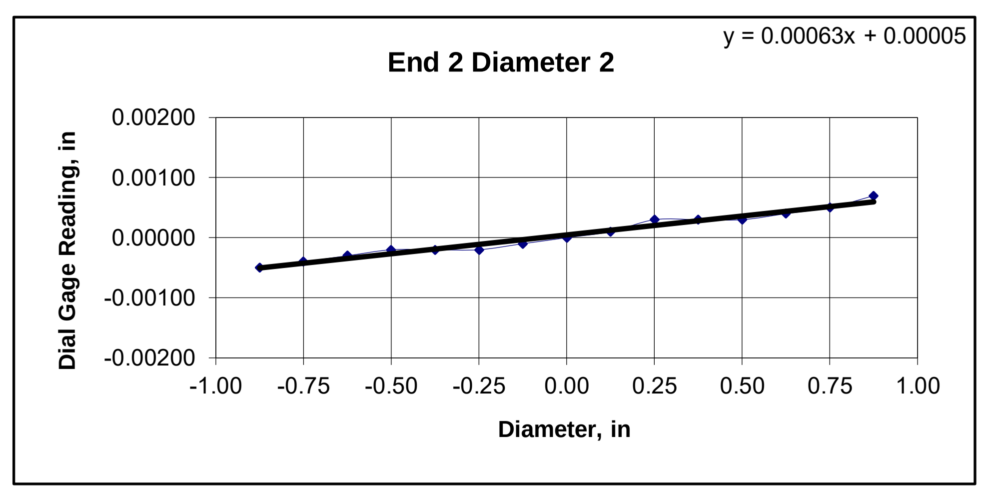
End 1:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00966

End 2:
Slope of Best Fit Line 0.00021
Angle of Best Fit Line: 0.01228

Maximum Angular Difference: 0.00262

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00057
Angle of Best Fit Line: 0.03258

End 2:
Slope of Best Fit Line 0.00063
Angle of Best Fit Line: 0.03601

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES
Spherically Seated

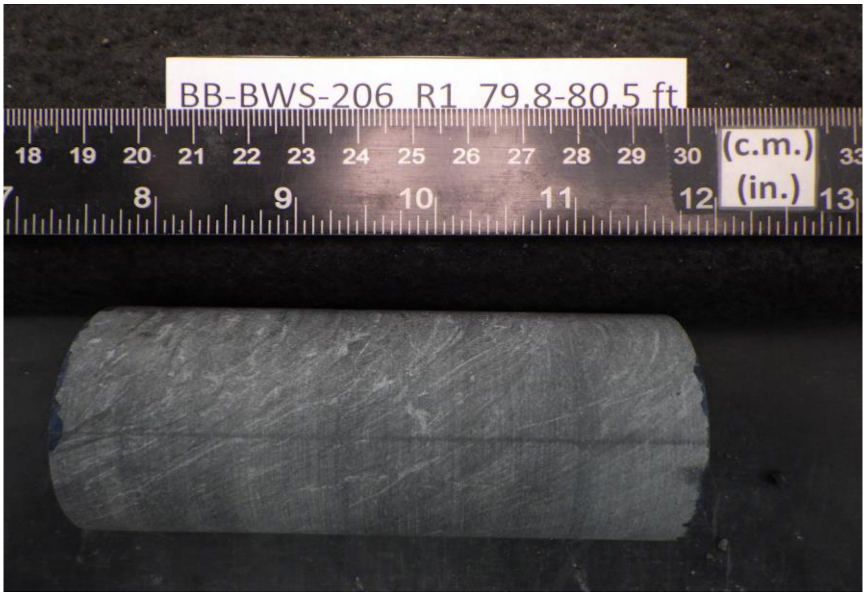
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00110	1.990	0.00055	0.032	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00120	1.990	0.00060	0.035	YES

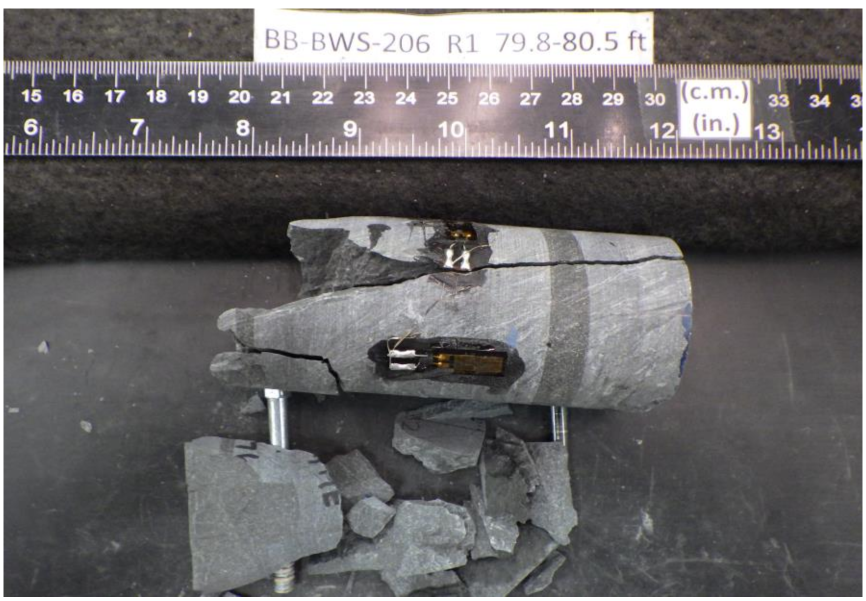
Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley Aldrich, Inc.
Project Name:	Rte 9/I-395 Conn. - Wilson St Bridge
Project Location:	Brewer Eddington, ME
GTX #:	311345
Test Date:	2/19/2020
Tested By:	cmh/kdp
Checked By:	smd
Boring ID:	BB-BWS-206
Sample ID:	R1
Depth, ft:	79.8-80.5



After cutting and grinding



After break

GeoTesting EXPRESS

SOIL CHAIN OF CUSTODY & TEST REQUEST

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT		INVOICE (complete if different from Client)	
Company: Haley & Aldrich, Inc.		Company:	
Address: 75 Washington Avenue, Suite 1A		Address:	
City, State, Zip: Portland, Maine 04101-2317		City, State, Zip:	
Contact: Bryan C. Steinert, P.E.	Phone: 207-482-4607	Contact:	Phone:
E-mail: bsteinert@haleyaldrich.com	Cell: 207-415-8322	E-mail:	Cell:

PROJECT		
Project Name: Route 9/I-395 Connector-Wilson Street Bridge	Client Project #: 132076-005	Purchase Order#:
Project Location: Brewer and Eddington, Maine	GTX Sales Order #:	Requested Turnaround:
On-site Contact: NA	E-mail: NA	Phone: NA

SOIL

Boring ID	Sample ID	Depth	Atterberg Limits (ASTM D 4318)	USCS – Classification (ASTM D 2487)	Grain Size: ASTM D 422 ☐ Sieve Only ☑ Sieve & Hydrometer ☐	Density: ASTM D 2937 ☐ ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction: Standard – ASTM D 698 ☐ Modified – ASTM D 1557 ☐	California Bearing Ratio * (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* : UU – ASTM D 2850 ☐ CU – ASTM D 4767 ☐ CD – ASTM D7181 ☐	Incremental Consolidation* (ASTM D 2435)	Permeability/ Hydraulic Conductivity* : Fixed Wall – ASTM D 2434 ☐ Flexible Wall – ASTM D 5084 ☐	Unconfined Compression (ASTM D 2166)	Other:	Other:
BB-BWS-205	4D	6'-8'			x															
BB-BWS-205	5D	8'-10'			x															
BB-BWS-205	6D	10'-12'			x															
BB-BWS-204	7D	13'-15'			x															
BB-BWS-204	8D	15'-17'			x															
BB-BWS-205	10D	18'-20'			x															
BB-BWS-204	10D	19'-21'			x															
BB-BWS-204	11D	21'-23'			x															
BB-BWS-204	13D	25'-27'			x															

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

*Unconfined Compression Tests with Elastic Modulus

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: bsteinert@haleyaldrich.com Digitally signed by bsteinert@haleyaldrich.com DN: cn=bsteinert@haleyaldrich.com Date: 2014.08.01 16:44:17 -04'00' PRINT NAME: Bryan Steinert DATE: 02/14/20

For GTX Use Only
Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By: Marley Snow	DATE: 2/14/2020 TIME: 11:05 Am	Received By: [signature]	DATE: 2/14/20 TIME: 11:15 am
Relinquished By: [signature]	DATE: 2/14/20 TIME: 1:30 pm	Received By: [signature]	DATE: 2/14/20 TIME: 1:23

GeoTesting EXPRESS

SOIL CHAIN OF CUSTODY & TEST REQUEST

GeoTesting Express, Inc.
125 Nagog Park
Acton, MA 01720
800 434 1062 Toll Free
978 635 0266 Fax

2358 Perimeter Park Drive, Suite 320
Atlanta, GA 30341
770 645 6575 Tel
770 645 6570 Fax

www.geotesting.com

CLIENT		INVOICE (complete if different from Client)	
Company: Haley & Aldrich, Inc.		Company:	
Address: 75 Washington Avenue, Suite 1A		Address:	
City, State, Zip: Portland, Maine 04101-2317		City, State, Zip:	
Contact: Bryan C. Steinert, P.E.	Phone: 207-482-4607	Contact:	Phone:
E-mail: bsteinert@haleyaldrich.com	Cell: 207-415-8322	E-mail:	Cell:

PROJECT		
Project Name: Route 9/I-395 Connector-Wilson Street Bridge	Client Project #: 132076-005	Purchase Order#:
Project Location: Brewer and Eddington, Maine	GTX Sales Order #:	Requested Turnaround:
On-site Contact: NA	E-mail: NA	Phone: NA

Boring ID	Sample ID	Depth	Atterberg Limits (ASTM D 4318)	USCS – Classification (ASTM D 2487)	Grain Size: ASTM D 422 ☐ Sieve Only ☑ Sieve & Hydrometer ☐	Density: ASTM D 2937 ☐ ASTM D 7263 ☐	Moisture Content (ASTM D 2216)	Organic Content (ASTM D 2974)	pH (ASTM D 4972)	Specific Gravity (ASTM D 854)	Electrical Resistivity (ASTM G 57)	Proctor Compaction: Standard – ASTM D 698 ☐ Modified – ASTM D 1557 ☐	California Bearing Ratio * (ASTM D 1883)	Direct Shear* (ASTM D 3080)	Triaxial Shear* : UU – ASTM D 2850 ☐ CU – ASTM D 4767 ☑ CD – ASTM D7181 ☐	Incremental Consolidation* (ASTM D 2435)	Permeability/ Hydraulic Conductivity* : Fixed Wall – ASTM D 2434 ☐ Flexible Wall – ASTM D 5084 ☐	Unconfined Compression (ASTM D 2166)	Other:	Other:
BB-BWS-301	3U	29'-31'	x												x	x				
BB-BWS-301	5D	19'-21'	x																	
BB-BWS-301	6D	24'-26'	x																	
BB-BWS-301	7D	34'-36'	x																	
BB-BWS-301	2D	5'-7'			x															
BB-BWS-301	3D	10'-12'			x															
BB-BWS-202	R3	79.7'-80.2'																*x		
BB-BWS-203	R1	45.3'-45.9'																*x		
BB-BWS-206	R1	79.8'-80.5'																*x		
BB-BWS-205	2D	2'-4'			x															
BB-BWS-204	3D	5'-7'			x															

*Specify Test Conditions (Undisturbed or Remolded, Density and moisture, Test Normal Loads, Test Confining Stresses, etc.):

*Unconfined Compression Tests with Elastic Modulus

AUTHORIZE BY SIGNING AND DATING:

SIGNATURE: bsteinert@haleyaldrich.com Digitally signed by bsteinert@haleyaldrich.com DN: cn=bsteinert@haleyaldrich.com Date: 2014.08.01 16:44:17 -04'00' PRINT NAME: Bryan Steinert DATE: 02/14/20

For GTX Use Only
Incoming Sample Inspection Performed ☐
Adverse conditions:

Relinquished By: [signature]	DATE: 2/14/2020	Received By: [signature]		DATE: 2/14/20	
	TIME: 11:05A			TIME: 11:15a	
Relinquished By: [signature]	DATE: 2/14/20	Received By: [signature]		DATE: 2/14/20	
	TIME: 1:30pm			TIME: 1:23	

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.

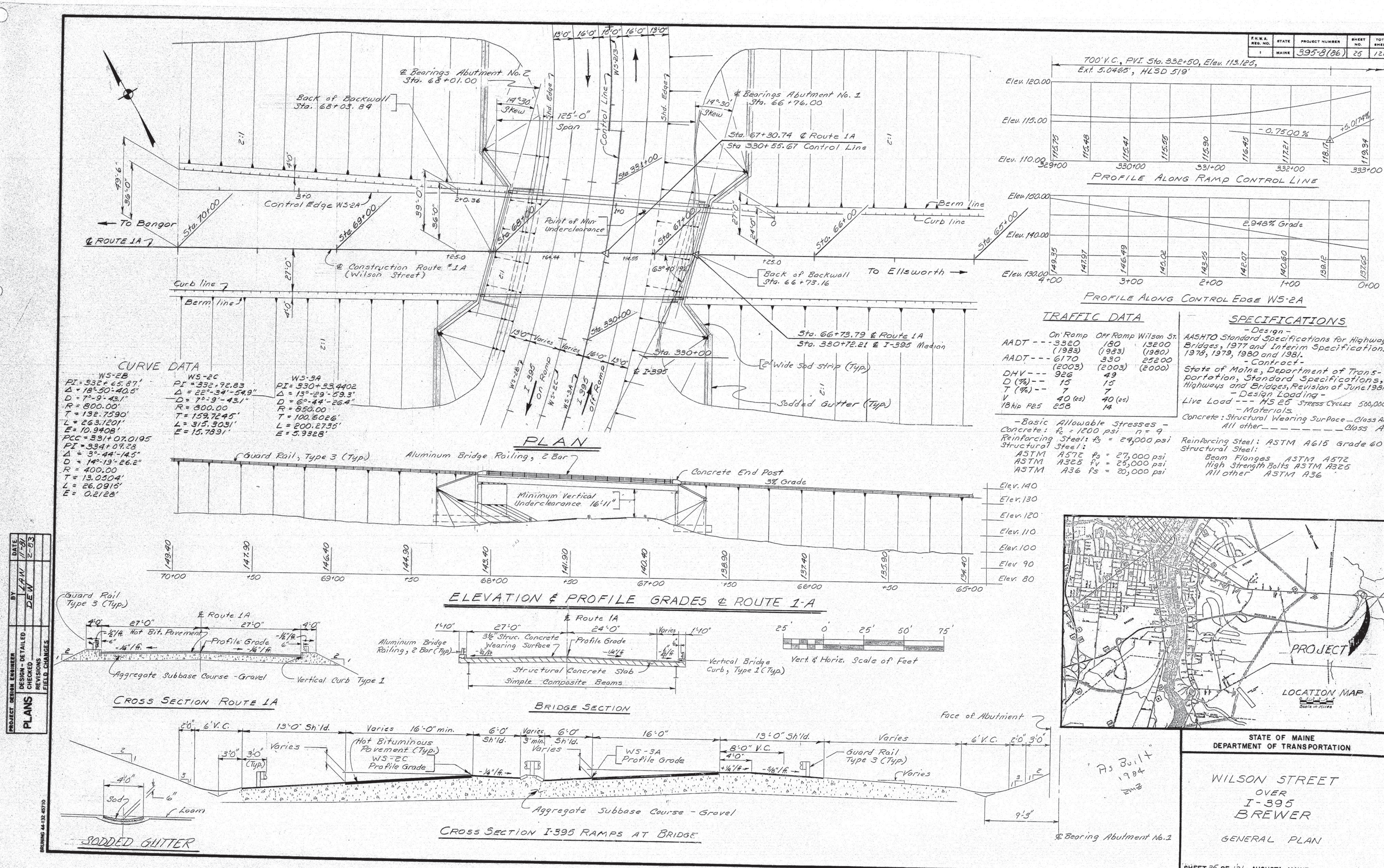
GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any 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

Symbol	Definition
A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
CAI	CERCHAR Abrasiveness Index
CIU	isotropically consolidated undrained triaxial shear test
CR	compression ratio for one dimensional consolidation
CSR	cyclic stress ratio
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	damping ratio
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
H_R	Rebound Hardness number
i	gradient
I_S	Uncorrected point load strength
$I_{S(50)}$	Size corrected point load strength index
H_A	Modified Taber Abrasion
H_T	Total hardness
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_r	Post cyclic undrained shear strength
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_s	shear wave velocity
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'_{vc}	Effective vertical consolidation 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

APPENDIX D

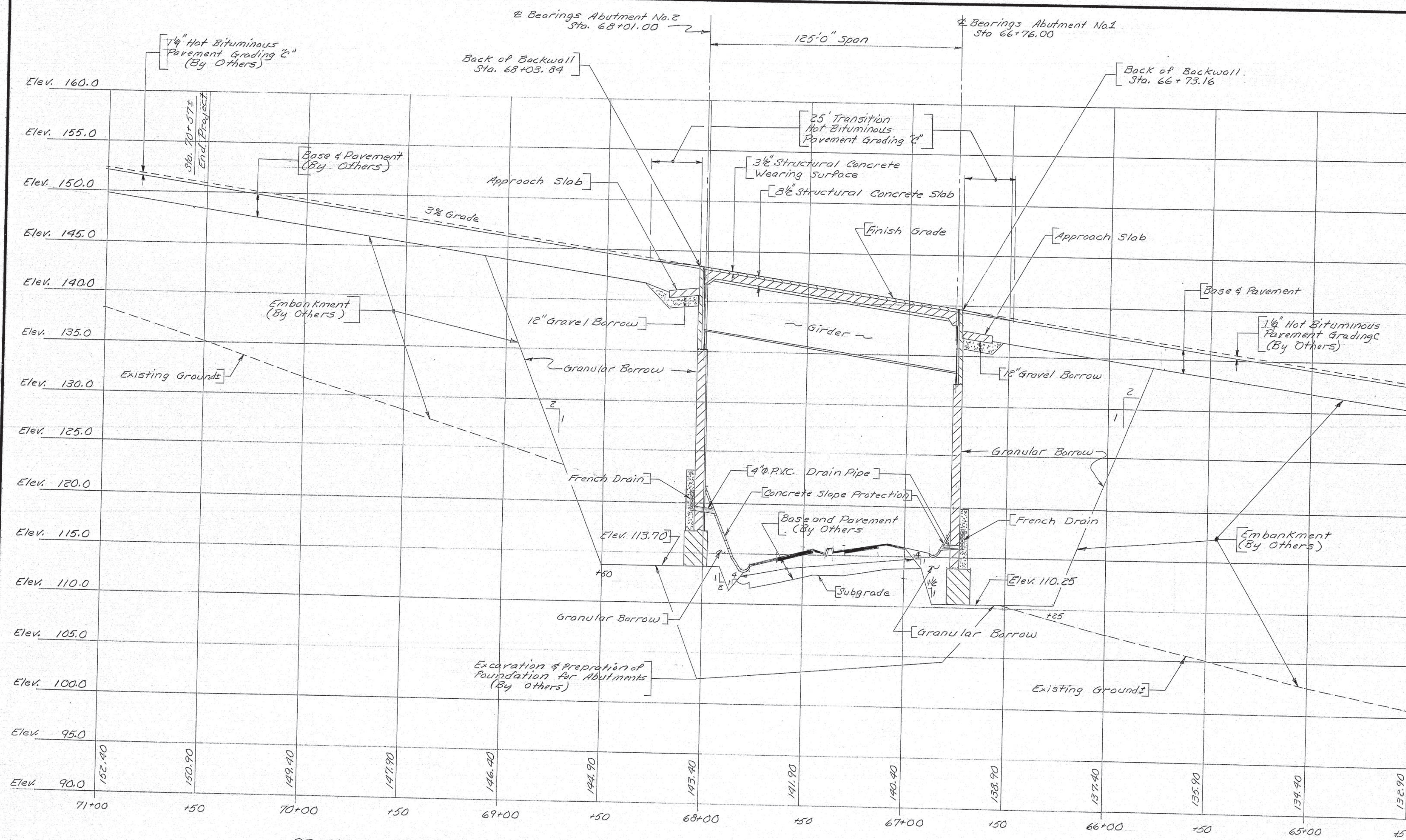
Historic Bridge Drawings and Subsurface Information



F.H.W.A. REG. NO. 1 | STATE MAINE | PROJECT NUMBER 395-8(86) | SHEET NO. 25 | TOTAL SHEETS 121
PLAN
℄ Bearings Abutment No. 2 Sta. 68+01.00
Back of Backwall Sta. 68+03.84
℄ Bearings Abutment No. 1 Sta. 66+76.00
125'-0" Span
Control Line
Sta. 67+30.74 ℄ Route 1A
Sta. 330+55.67 Control Line
Control Edge WS-2A
To Bangor
℄ ROUTE 1A
℄ Construction Route #1A (Wilson Street)
Point of Min. Underclearance
Back of Backwall Sta. 66+73.16
To Ellsworth
Curb line
Berm line
Sta. 66+73.79 ℄ Route 1A
Sta. 330+72.21 ℄ I-395 Median
2' Wide Sod strip (Typ)
Sodded Gutter (Typ)
I-395 on Ramp
I-395 off Ramp
CURVE DATA
WS-2B
PI = 332+65.87'
Δ = 18°-50'-40.5"
D = 7°-9'-43.1"
R = 800.00'
T = 132.7590'
L = 263.1201'
E = 10.9408'
PCC = 331+07.0195
PI = 334+09.28
Δ = 3°-44'-14.5"
D = 14°-19'-26.2"
R = 400.00
T = 13.0504'
L = 26.0916'
E = 0.2128'
WS-2C
PI = 332+92.83
Δ = 22°-34'-54.9"
D = 7°-9'-43.1"
R = 800.00
T = 159.7245'
L = 315.3031'
E = 15.7891'
WS-3A
PI = 330+33.4402
Δ = 13°-29'-59.3"
D = 6°-44'-26.4"
R = 850.00
T = 100.6026'
L = 200.2735'
E = 5.9328'
700' V.C., PVI Sta. 332+50, Elev. 113.125, Ext. 5.0465', HLSD 519'
PROFILE ALONG RAMP CONTROL LINE
-0.7500%
+5.0174%
2.948% Grade
PROFILE ALONG CONTROL EDGE WS-2A
TRAFFIC DATA
On Ramp | Off Ramp | Wilson St.
AADT --- 3320 (1983) | 180 (1983) | 13200 (1980)
AADT --- 6170 (2003) | 330 (2003) | 25200 (2000)
DHV --- 926 | 49
D (%) -- 15 | 15
T (%) -- 7 | 7
V 40 (es) | 40 (es)
18 kip P.S 258 | 14
-Basic Allowable Stresses-
Concrete: f'c = 1200 psi n = 9
Reinforcing Steel: fs = 24,000 psi
Structural Steel:
ASTM A572 fs = 27,000 psi
ASTM A325 Fv = 25,000 psi
ASTM A36 Fs = 20,000 psi
SPECIFICATIONS
-Design-
AASHTO Standard Specifications for Highway Bridges, 1977 and Interim Specifications 1978, 1979, 1980 and 1981.
-Contract-
State of Maine, Department of Transportation, Standard Specifications, Highways and Bridges, Revision of June 1981.
-Design Loading-
Live Load --- HS 25 Stress Cycles 500,000
-Materials-
Concrete: Structural Wearing Surface — Class AA
All other — Class A
Reinforcing Steel: ASTM A615 Grade 60
Structural Steel:
Beam Flanges ASTM A572
High Strength Bolts ASTM A325
All other ASTM A36
Guard Rail, Type 3 (Typ)
Aluminum Bridge Railing, 2 Bar
Concrete End Post
3% Grade
Minimum Vertical Underclearance 16'-11"
ELEVATION & PROFILE GRADES ℄ ROUTE 1-A
Guard Rail Type 3 (Typ)
Hot Bit. Pavement
Profile Grade
Aggregate Subbase Course - Gravel
Vertical Curb Type 1
CROSS SECTION ROUTE 1A
3½" Struc. Concrete Wearing Surface
Structural Concrete Slab
Simple Composite Beams
Vertical Bridge Curb, Type 1 (Typ)
BRIDGE SECTION
Vert. & Horiz. Scale of Feet
LOCATION MAP
PROJECT
Face of Abutment
Hot Bituminous Pavement (Typ)
WS-2C Profile Grade
WS-3A Profile Grade
Aggregate Subbase Course - Gravel
CROSS SECTION I-395 RAMPS AT BRIDGE
SODDED GUTTER
℄ Bearing Abutment No. 1
"As Built" 1984
STATE OF MAINE DEPARTMENT OF TRANSPORTATION
WILSON STREET OVER I-395 BREWER
GENERAL PLAN
SHEET 25 OF 121 AUGUSTA, MAINE
PLANS: DESIGN-DETAILED LAW 11-81; CHECKED DEW 2-83

182-58

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-B (86)	26	121



PROFILE & CONSTRUCTION ~ WILSON STREET ROUTE #1A

PROJECT DESIGN ENGINEER	BY	DATE
PLANS DESIGN - DETAILED	S.O.T. R.V.N.	11-82
CHECKED	DEW	2-83
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710

"As Built" 1984 RMB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

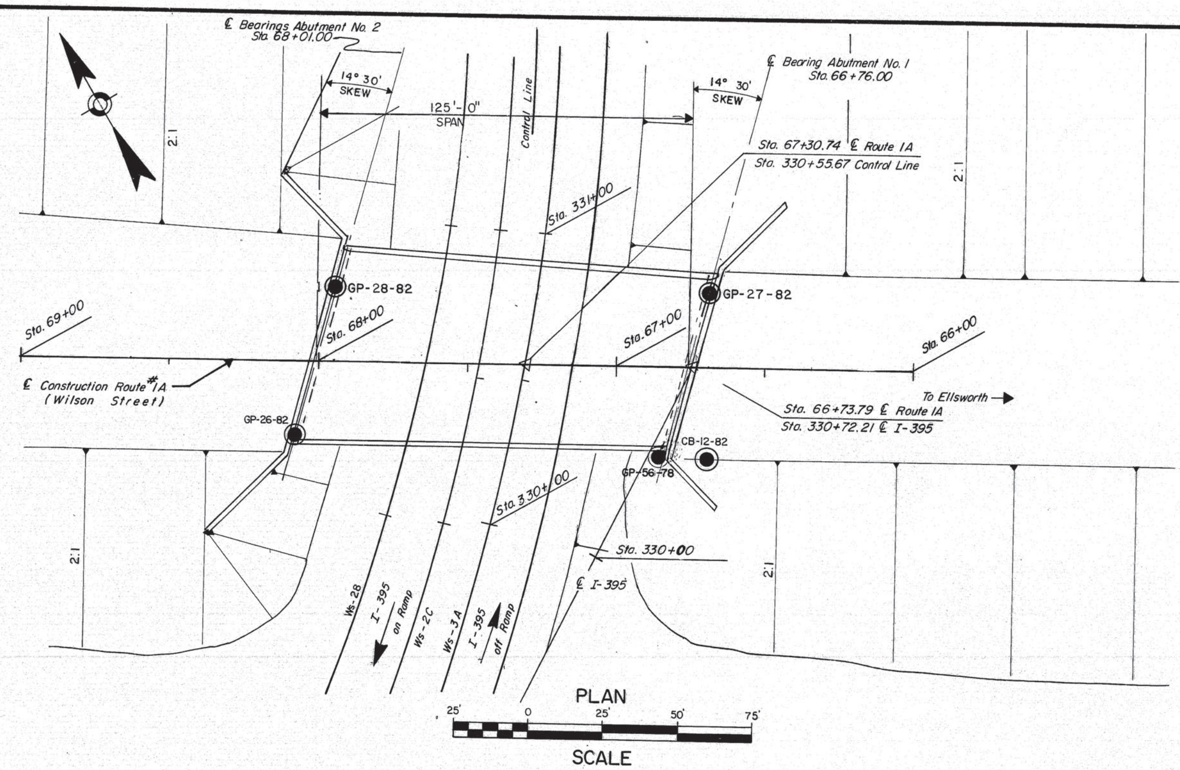
WILSON STREET
OVER
I-395
BREWER

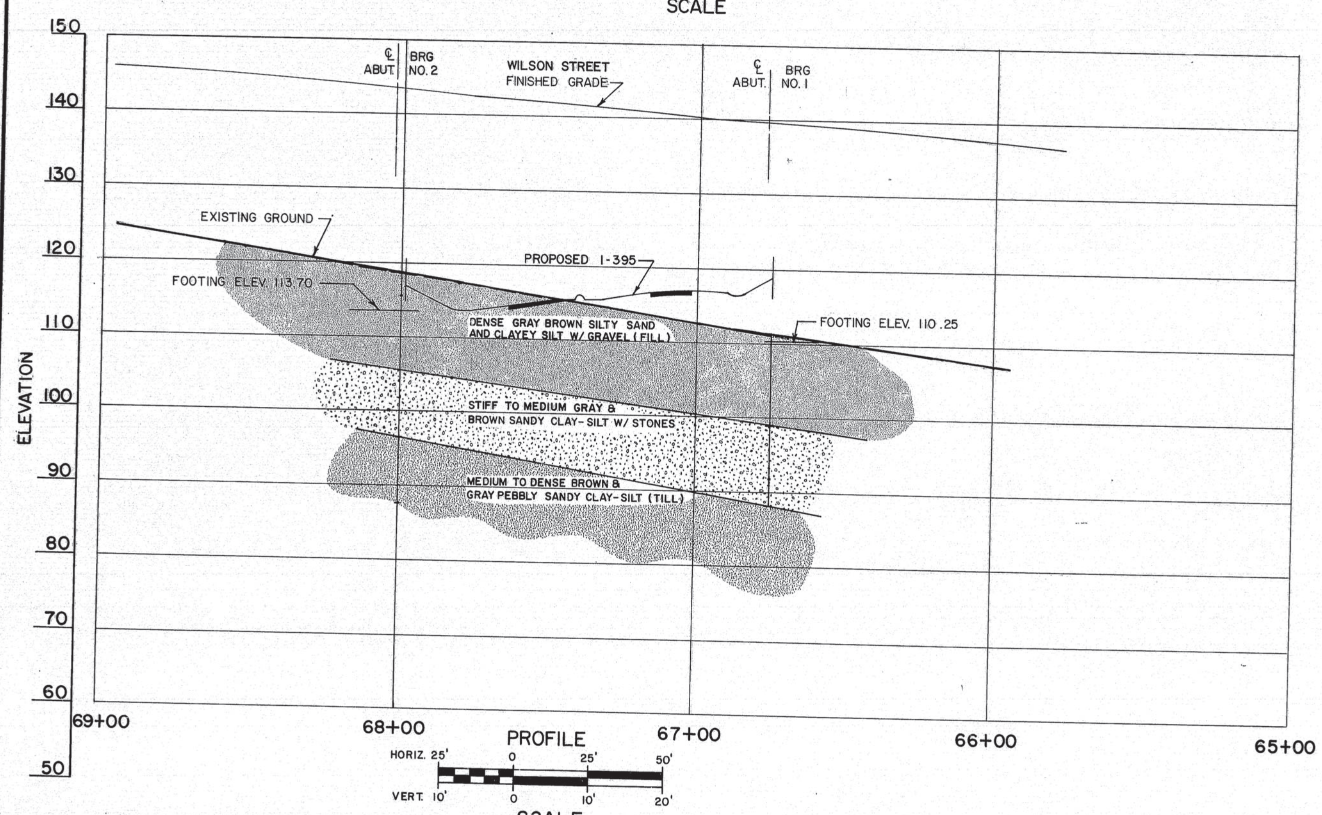
PROFILE & CONSTRUCTION LIMITS

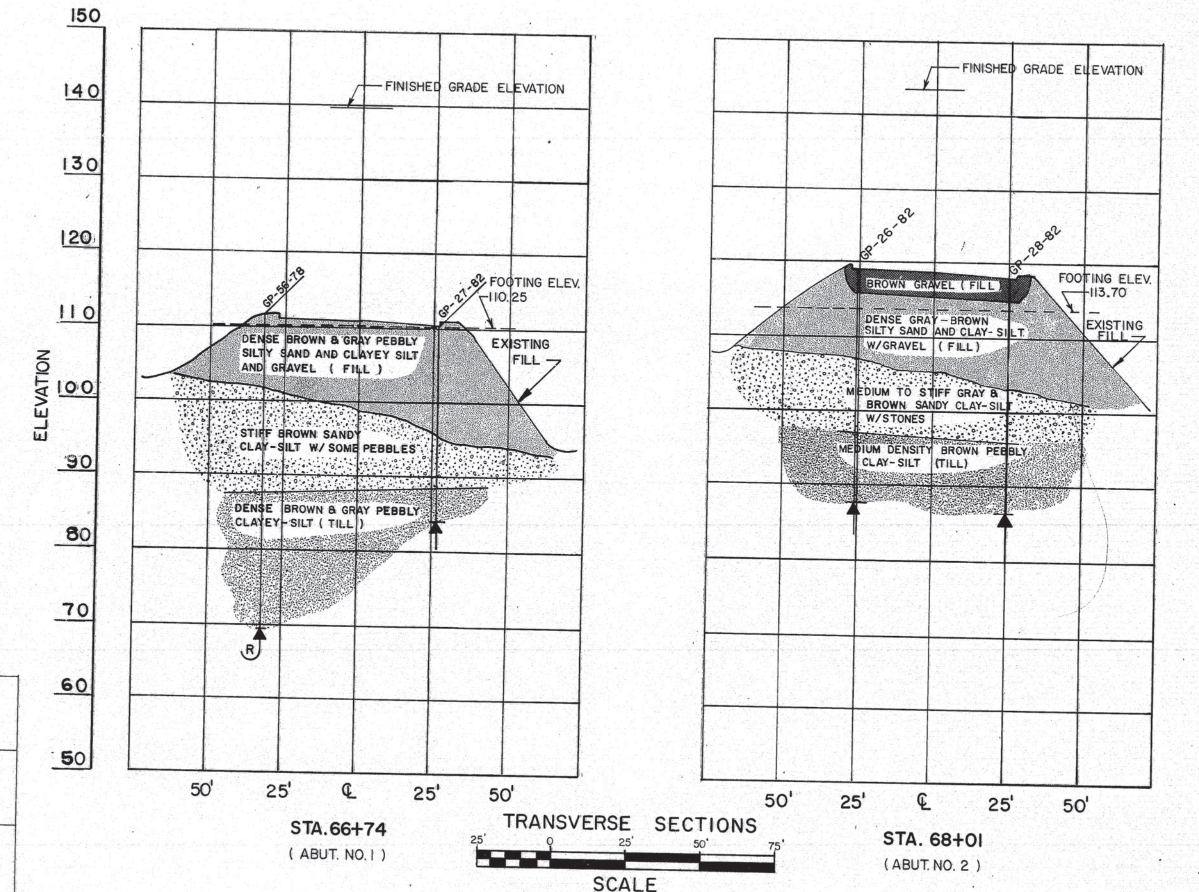
SHEET 26 OF 121 AUGUSTA, MAINE

182-59

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (86)	27	121







PROJECT DESIGN ENGINEER	BY	DATE
PLANS		
DESIGN-DETAILED		
CHECKED		
REVISIONS		
FIELD CHANGES		

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILSON STREET
OVER
I-395
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY
FOUNDATION SURVEY

SHEET 27 OF 121 AUGUSTA, MAINE

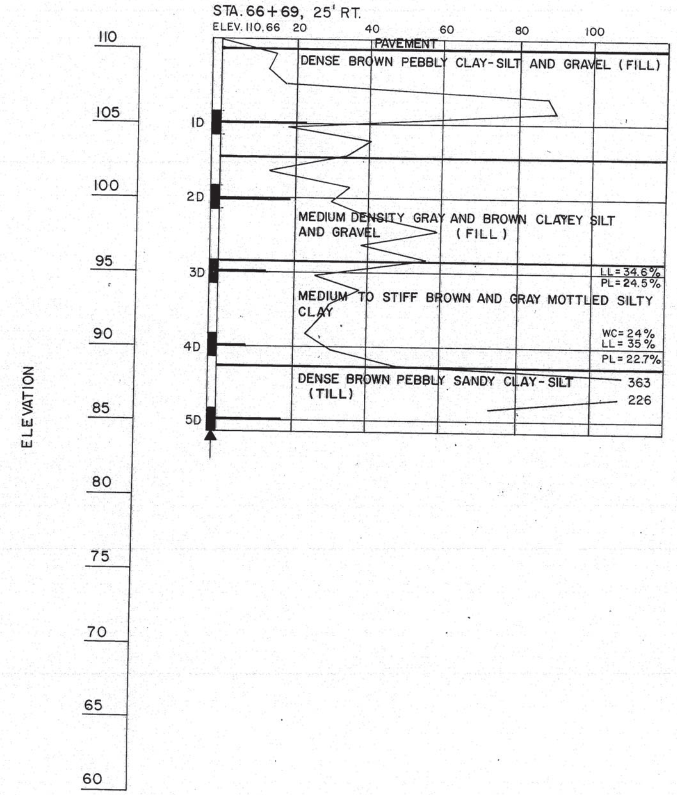
182-60

BRUNING 44-132 45710

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (66)	28	121

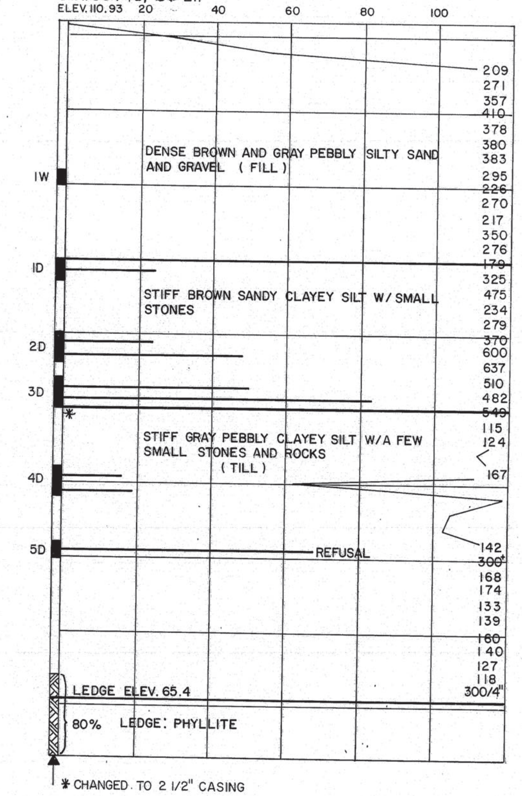
BORING GP-27-82

STA. 66+69, 25' RT.



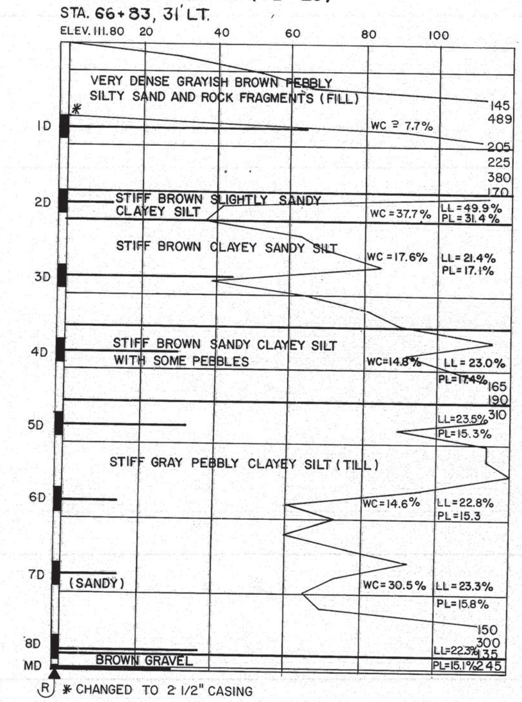
BORING CB-12-82

STA. 66+70, 30' LT.



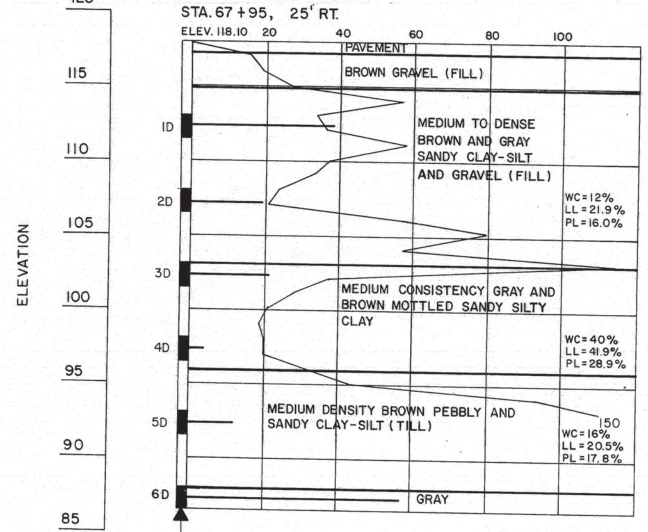
BORING GP-56-78 (TB-23)

STA. 66+83, 31' LT.



BORING GP-28-82

STA. 67+95, 25' RT.



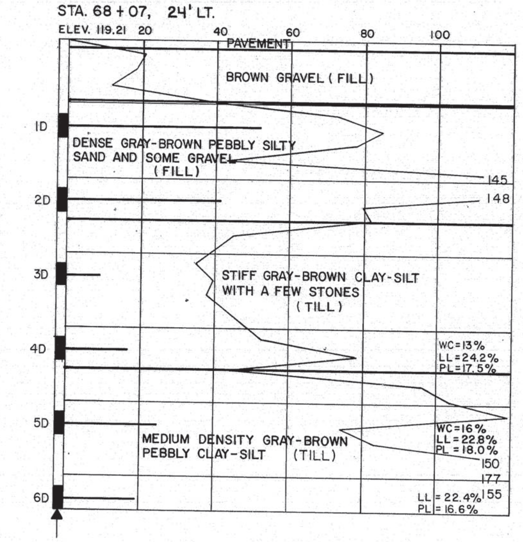
BORING NOTES

All samples and vanes are made ahead of casing

- Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blows
- Location of sample or sample attempt
- Number and type of dry sample
- ID — S & H Sampler # 1290's
- IW — Wash sample and number
- MD — Unsuccessful sample attempt and type of sampler
- Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow
- Bottom of boring (may not be bottom of soil strata)
- (R) Refusal of drill rods or casing (may not be ledge)
- 71% Locations cored by diamond bit and percent recovery of rock

BORING GP-26-82

STA. 68+07, 24' LT.



PROJECT DESIGN ENGINEER	BY	DATE
PLANS — DESIGN-DETAILED		
CHECKED		
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

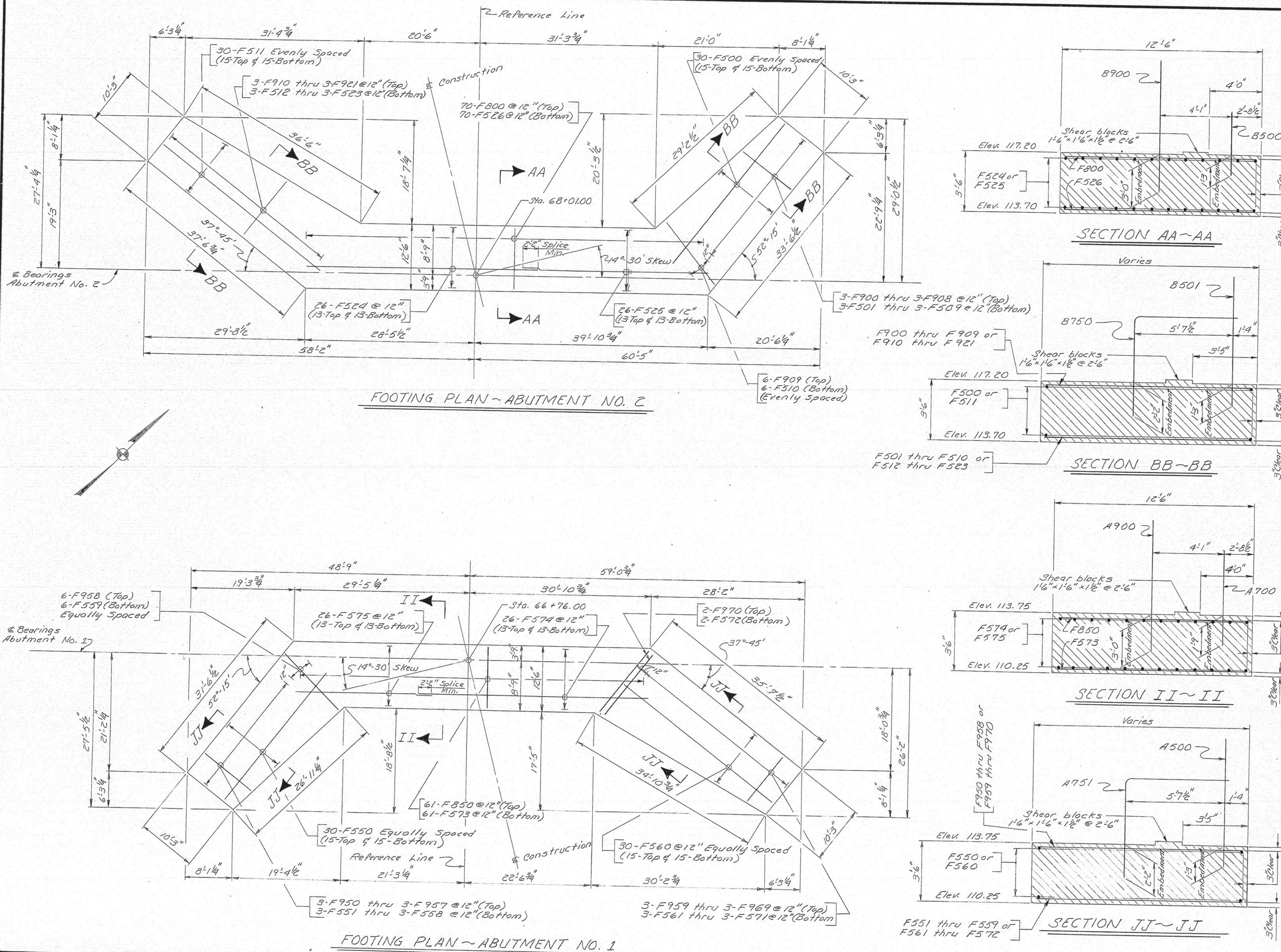
WILSON ST.
OVER
I-395
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY

BORING DETAILS

SHEET 28 OF 121 AUGUSTA, MAINE

182-61

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(86)	29	121



ABUTMENT NOTES

1 — Reinforcing steel shall have 2 inches cover unless otherwise indicated.

2 — Protective coating for concrete surfaces shall be applied to the following areas:
Top of concrete curbs.
Top of abutment backwalls and 1' below top of backwalls on the back side.

3 — Place 4" diameter drains in breastwall and wings at 20 feet maximum spacing. Exact location to be determined by the Engineer in the field.

4 — Maximum calculated footing pressures are:
Abutments 5.0 Tons/square foot
Wings 4.0 Tons/square foot.

PROJECT DESIGN ENGINEER	BY	DATE
PLANS DESIGN - DETAILED	G.D.T. R.V.N.	6-82
CHECKED	DEW L.A.W.	2-83
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

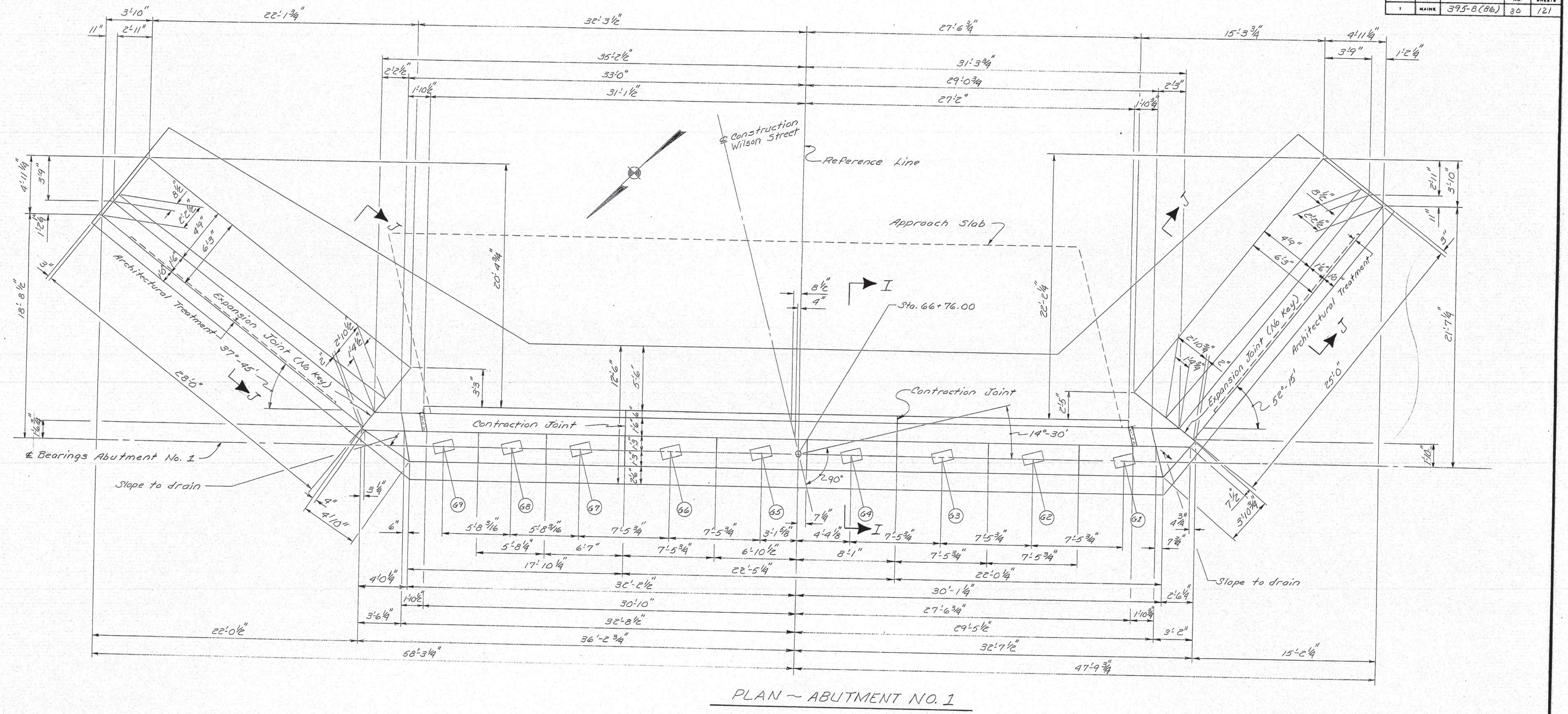
WILSON STREET
OVER
I-395
BREWER

ABUTMENT FOOTINGS

182-62

SHEET 29 OF 121 AUGUSTA, MAINE

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(86)	30	121



PLAN ~ ABUTMENT NO. 1

PROJECT DESIGN ENGINEER	BY	DATE
PLANS DESIGN-DETAILED		
CHECKED		
REVISIONS		
FIELD CHANGES		

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

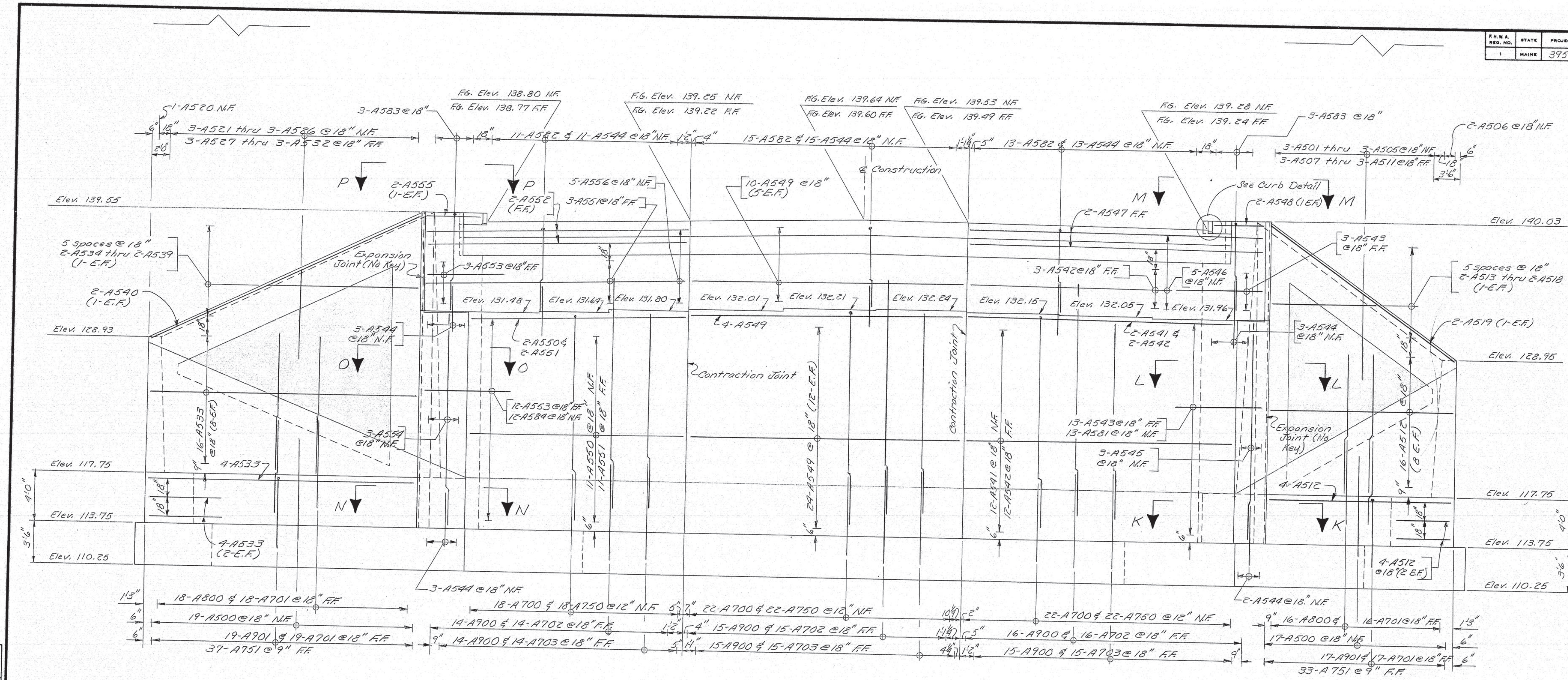
WILSON STREET
OVER
I-395
BREWER

PLAN ~ ABUTMENT NO. 1

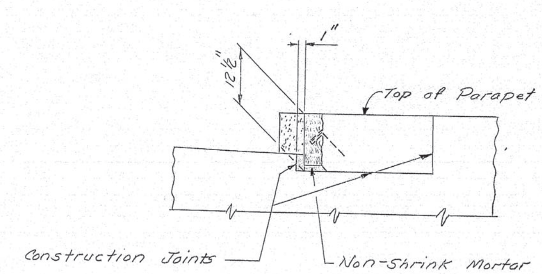
182-63

SHEET 30 OF 121 AUGUSTA, MAINE

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(86)	31	121



ELEVATION ABUTMENT NO. 1



CURB DETAIL
For details not shown see BD 126-81, sheet #23.

~LEGEND~
N.F. = Near Face
F.F. = Far Face
E.F. = Each Face

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILSON STREET
OVER
I-395
BREWER

ELEVATION ~ ABUTMENT NO. 1

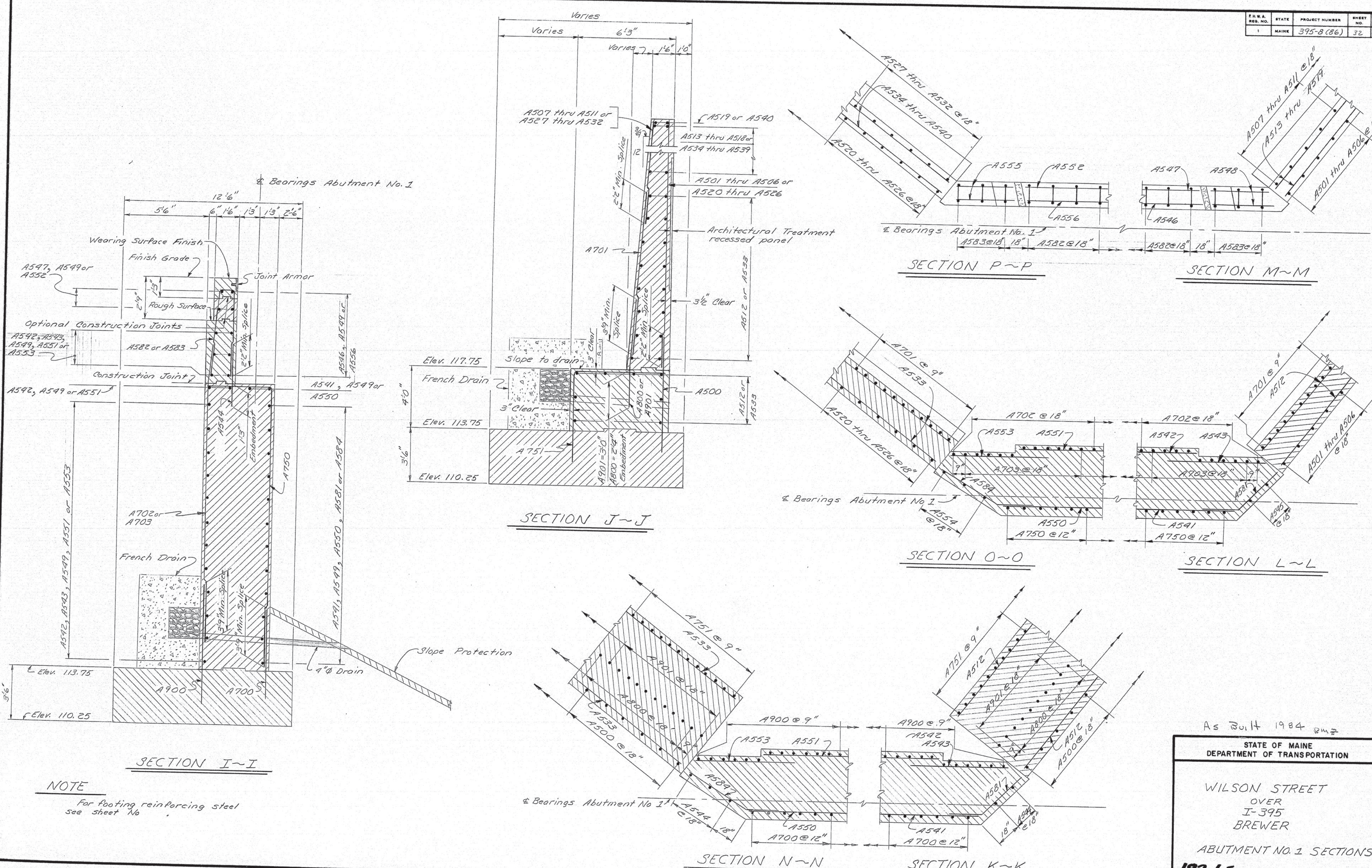
182-64

SHEET 31 OF 121 AUGUSTA, MAINE

PROJECT DESIGN ENGINEER		BY		DATE
PLANS	DESIGN - DETAILED	G.O.T.	R.V.N.	7-82
	CHECKED	DEW	LAW	2-83
	REVISIONS			
	FIELD CHANGES			

BRUNING 44-132 45710

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (86)	32	121



NOTE

For footing reinforcing steel see sheet No

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILSON STREET
OVER
I-395
BREWER

ABUTMENT NO. 1 SECTIONS

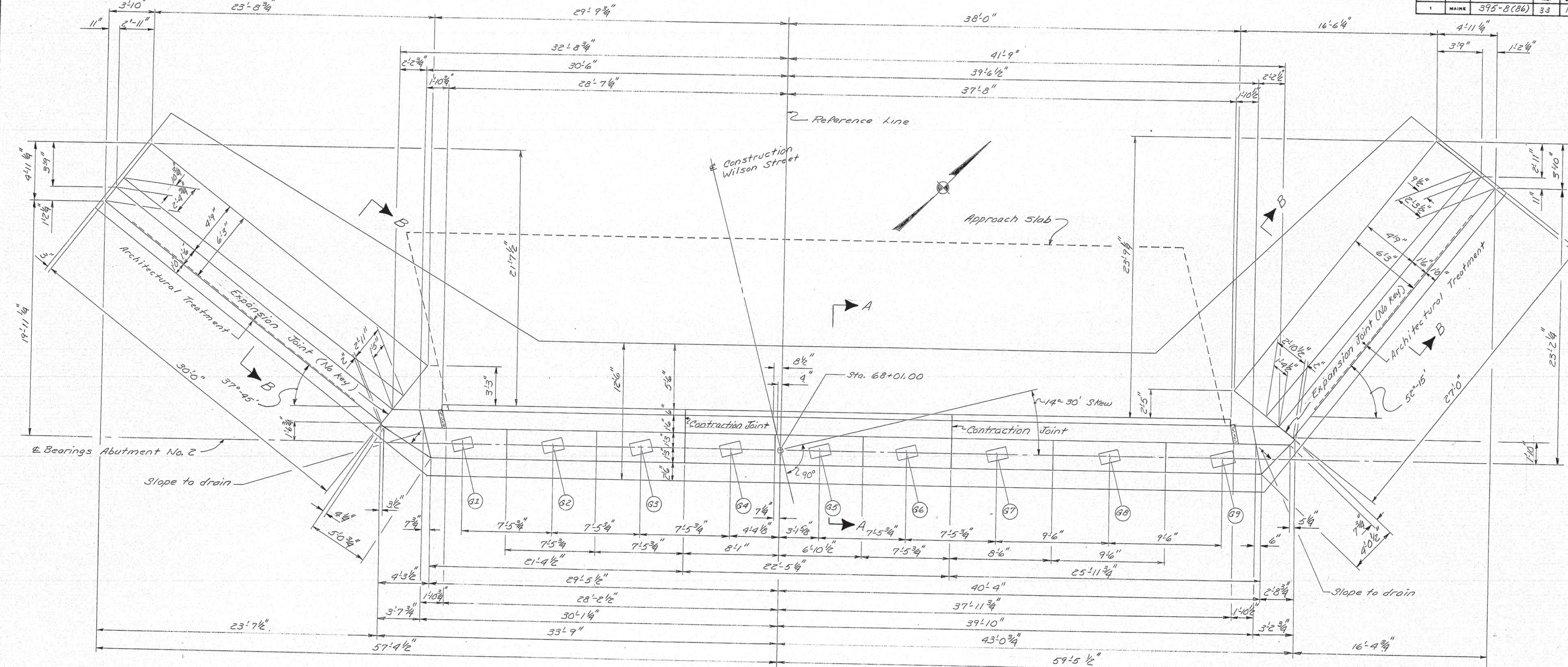
182-65

SHEET 32 OF 121 AUGUSTA, MAINE

PROJECT DESIGN ENGINEER	BY	DATE
PLANS DESIGN-DETAILED	G.O.I. / R.V.N.	6-82
CHECKED	DEW / L.A.W.	2-83
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(86)	33	121



PLAN ~ ABUTMENT NO. 2

PROJECT DESIGN ENGINEER	BY	DATE
PLANS — DESIGN - DETAILED	G.O.T. R.K.N.	5-82
CHECKED	D.E.W. L.A.W.	2-83
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

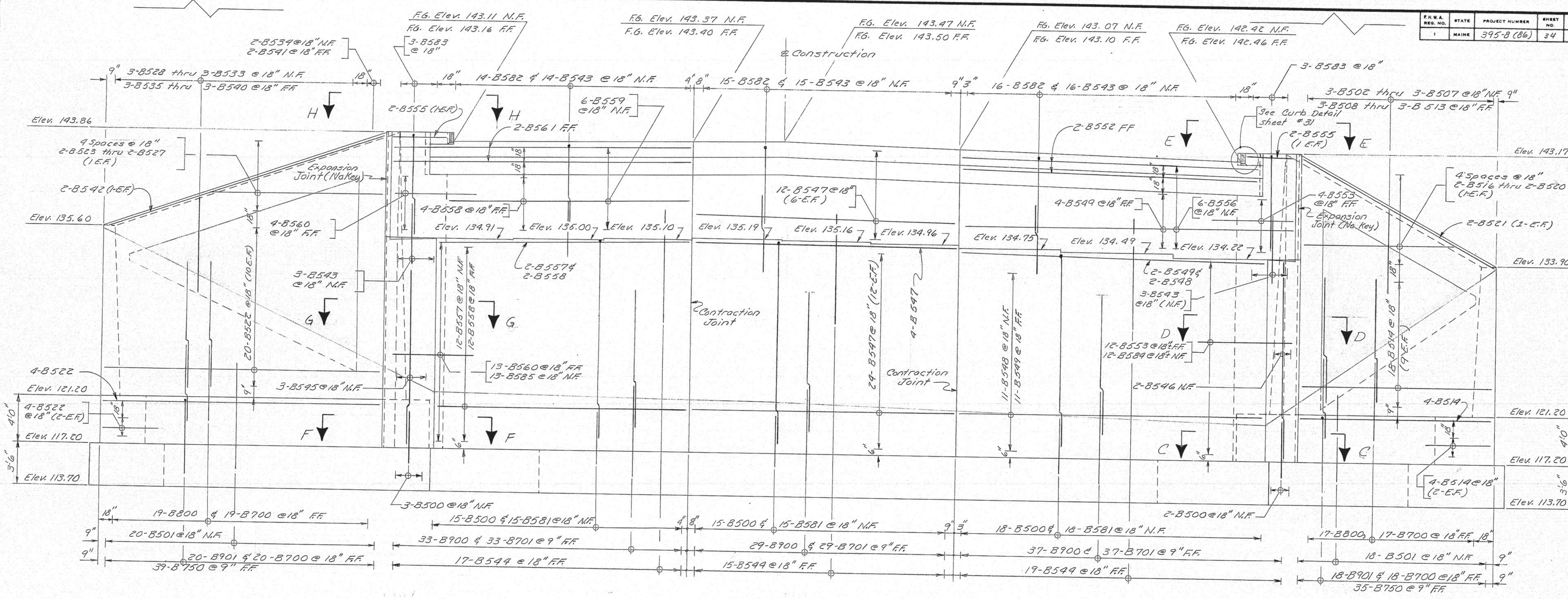
WILSON STREET
OVER
I-395
BREWER

PLAN ~ ABUTMENT NO. 2

182-66

SHEET 33 OF 121 AUGUSTA, MAINE

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (86)	34	121



ELEVATION ABUTMENT NO. 2

SEE PROJECT RECORDS for settlement this Abut during Const.

~ LEGEND ~

N.F. = Near Face
F.F. = Far Face
E.F. = Each Face

PROJECT DESIGN ENGINEER		BY	DATE
PLANS	DESIGN - DETAILED	G.O.T. R.V.N.	6-82
	CHECKED	DEW L.A.W.	2-83
	REVISIONS		
	FIELD CHANGES		

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

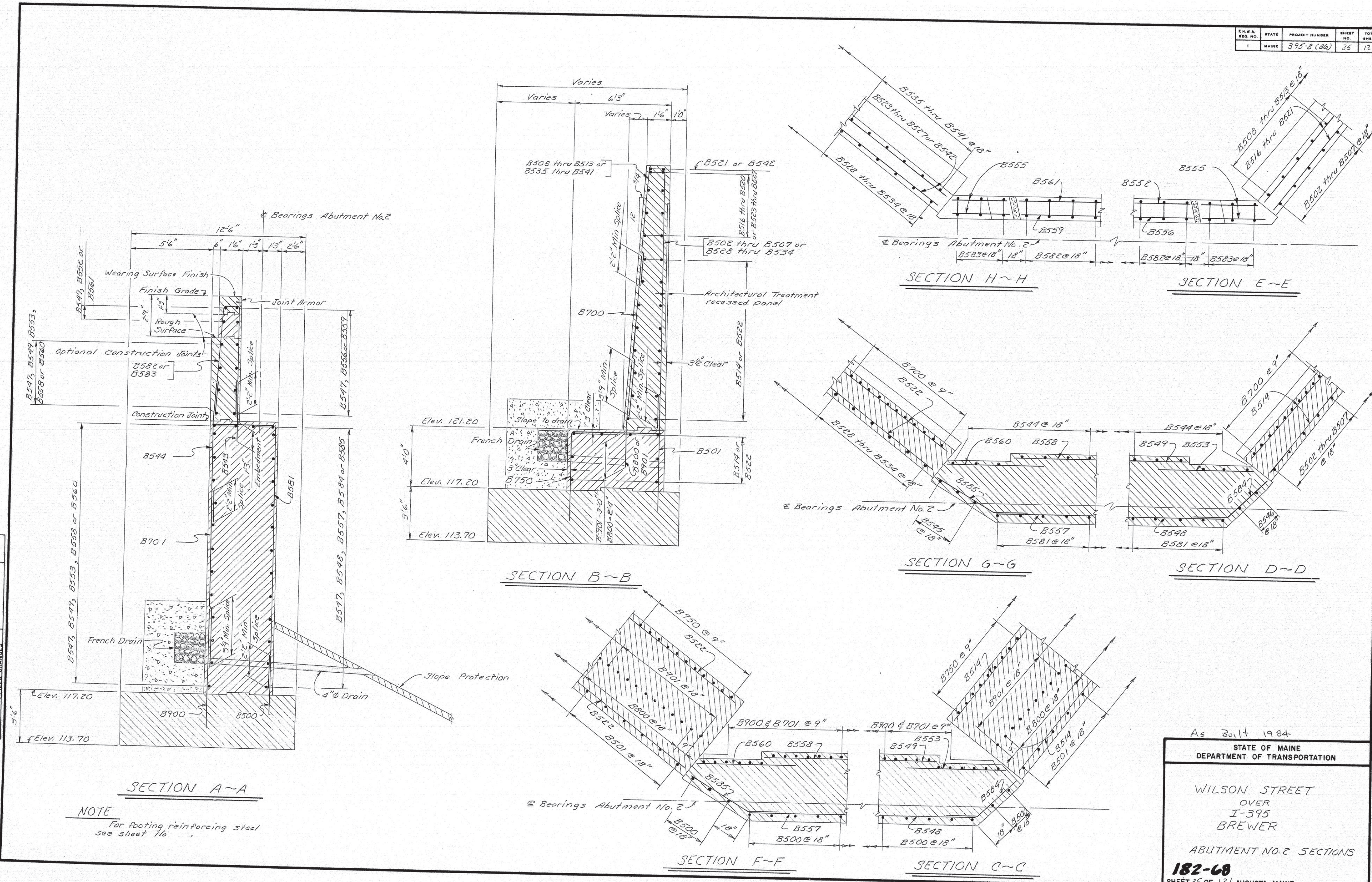
WILSON STREET
OVER
I-395
BREWER

ELEVATION~ABUTMENT NO. 2

182-67

SHEET 34 OF 121 AUGUSTA, MAINE

BRUNING 44-132 45710

F.H.W.A. REG. NO. 1 | STATE MAINE | PROJECT NUMBER 395-8 (86) | SHEET NO. 35 | TOTAL SHEETS 121
SECTION A~A
SECTION B~B
SECTION H~H
SECTION E~E
SECTION G~G
SECTION D~D
SECTION F~F
SECTION C~C
℄ Bearings Abutment No.2
Wearing Surface Finish
Finish Grade
Joint Armor
Rough Surface
Optional Construction Joints
Construction Joint
French Drain
Slope Protection
4"⌀ Drain
Elev. 117.20
Elev. 113.70
Elev. 121.20
Slope to drain
Architectural Treatment recessed panel
NOTE
For footing reinforcing steel see sheet No.
PROJECT DESIGN ENGINEER
PLANS
DESIGN - DETAILED: G.O.T. R.V.N. 6-82
CHECKED: DEW L.R.W. 2-83
REVISIONS
FIELD CHANGES
As Built 1984
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
WILSON STREET
OVER
I-395
BREWER
ABUTMENT NO.2 SECTIONS
182-68
SHEET 35 OF 121 AUGUSTA, MAINE

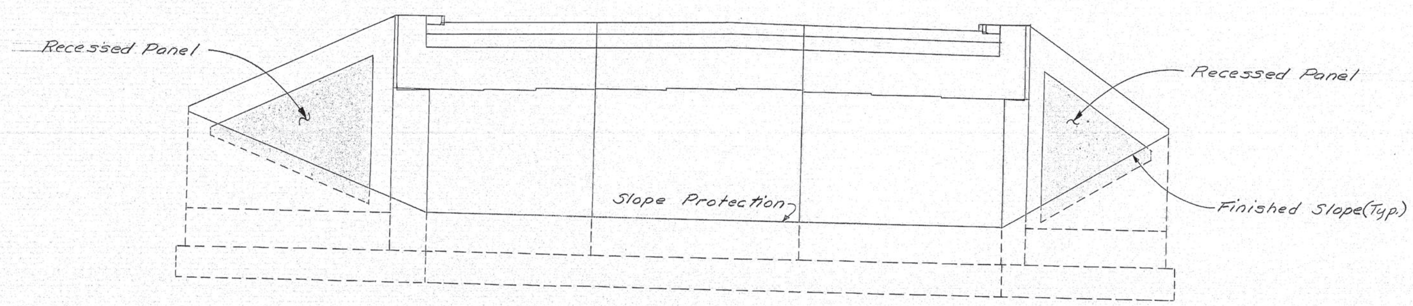
F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (86)	36	121

GENERAL NOTES

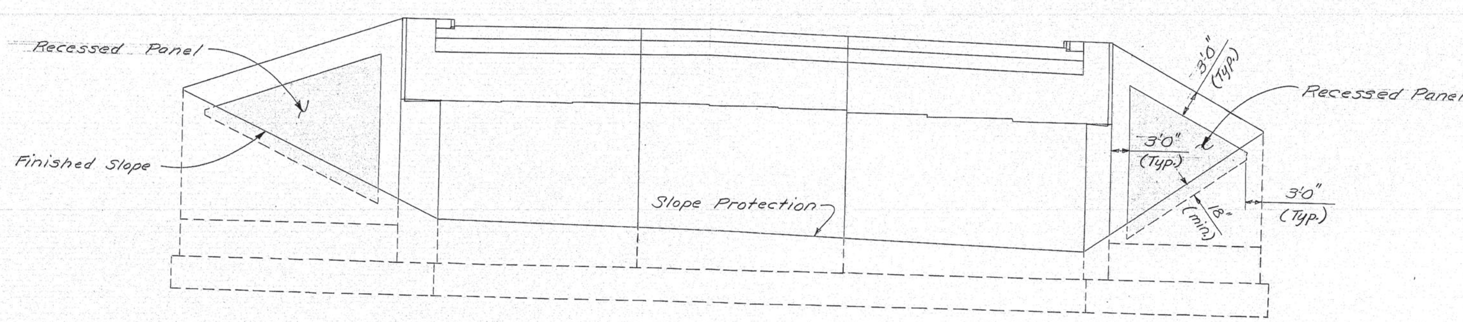
1----The recessed panels shall be carried to a minimum depth of 18 inches below the finish ground.

2----Special care shall be exercised so that form joints at the exposed face of concrete shall be tight.

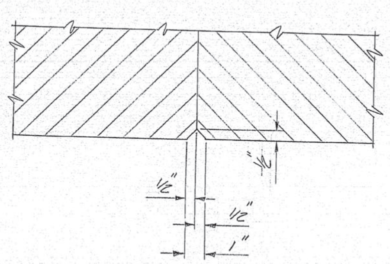
3----No deduction in the concrete pay volume shall be made for the recess of the architectural treatment.



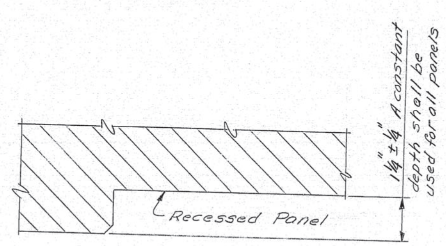
ELEVATION ABUTMENT NO. 1



ELEVATION ABUTMENT NO. 2



TYPICAL JOINT SECTION



TYPICAL PANEL DETAIL

PROJECT DESIGN ENGINEER	BY		DATE
PLANS — DESIGN - DETAILED	G.O.T.	R.V.N.	10-82
CHECKED	DEW		2-83
REVISIONS			
FIELD CHANGES			

BRUNING 44-132 45710

As Built 1984

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

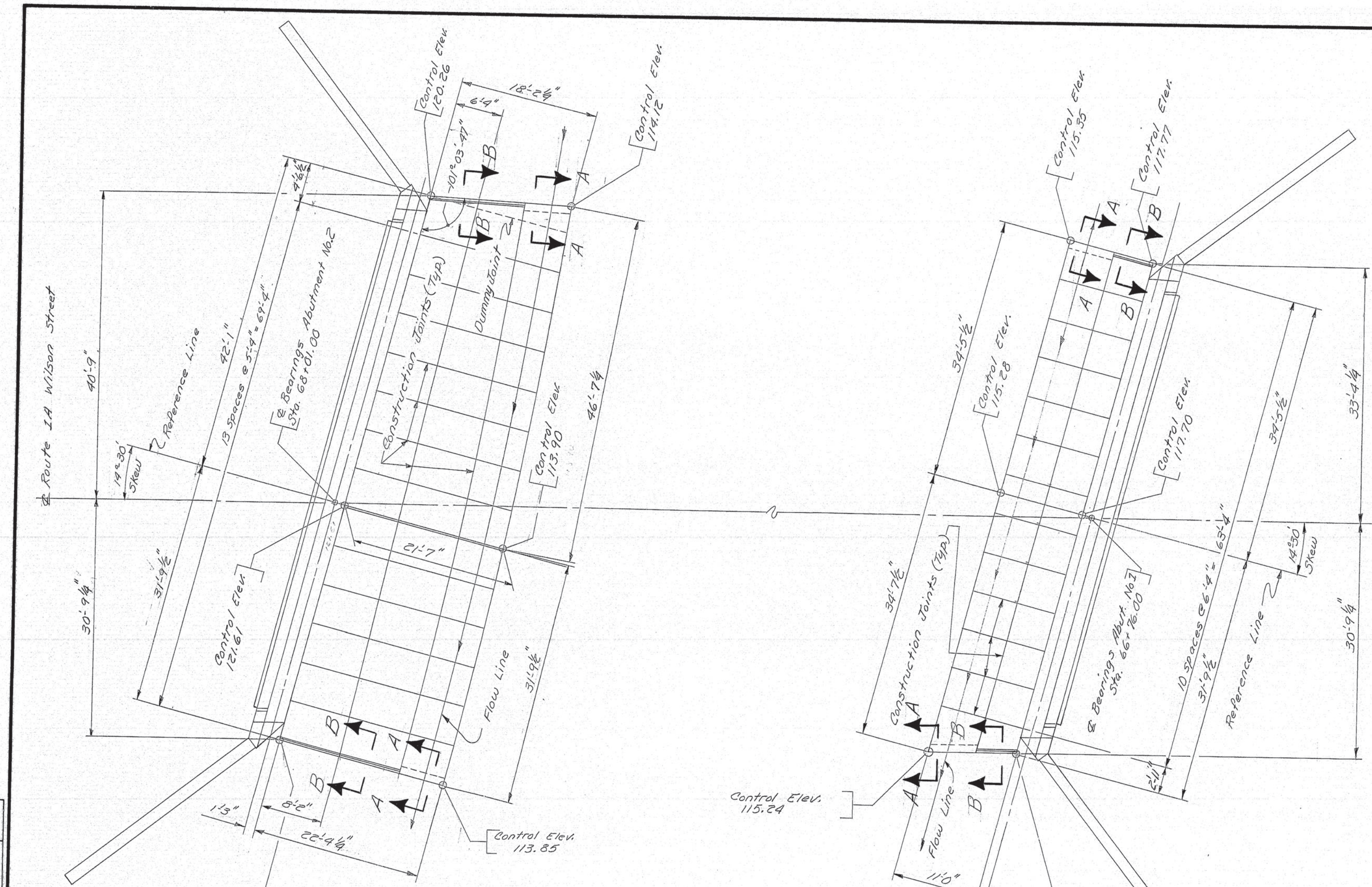
WILSON STREET
OVER
I-395
BREWER

ARCHITECTURAL TREATMENT

182-69

SHEET 36 OF 121 AUGUSTA, MAINE

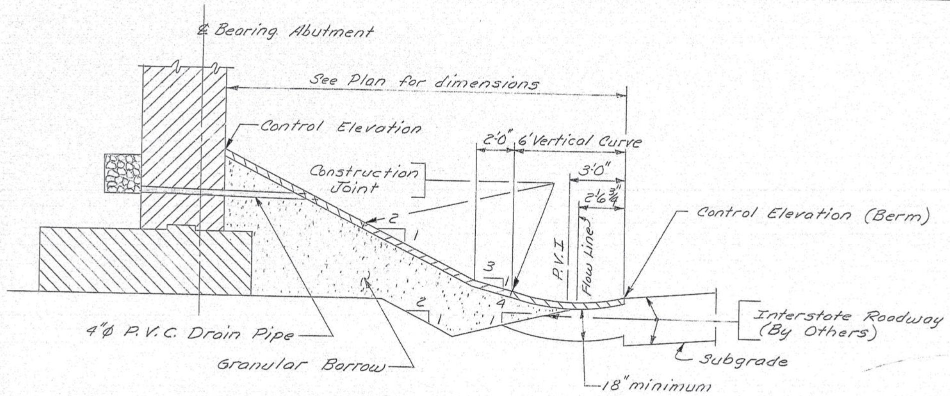
F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(86)	37	121



CONCRETE SLOPE PROTECTION ~ PLAN

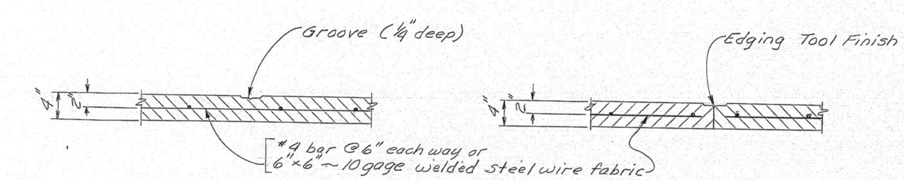
CONCRETE SLOPE PROTECTION NOTES

1. Steel mesh shall not pass through any construction joint.
2. Break the bond in construction joints by a method approved by the Engineer.
3. Portland Cement Concrete for slope protection shall be class Y.



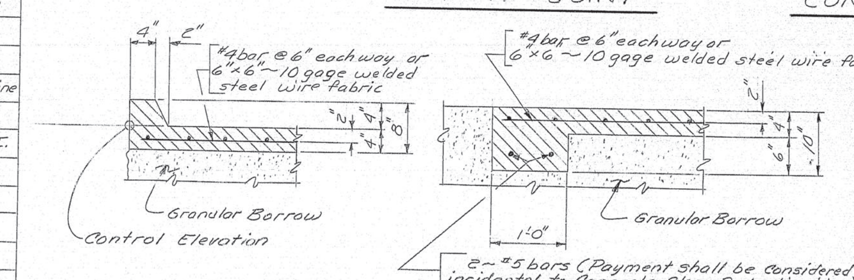
TYPICAL SECTION
(All dimensions normal to abutment)

Ditch Section Elevations Relative To Berm at Every Foot		
Distance	Elevation	Remarks
0'	0.0'	Berm
1'	-0.2'	—
2'	-0.31'	—
2.57'	-0.32'	Flow Line
3'	-0.31'	—
3'	-0.75'	P.V.I.
4'	-0.22'	—
5'	-0.03'	—
6'	+0.25'	P.V.T.
7'	+0.583'	—
8'	+0.916'	—



DUMMY JOINT

CONSTRUCTION JOINT



SECTION B~B

SECTION A~A

As Built 1984

PROJECT DESIGN ENGINEER	BY	DATE
PLANS DESIGN - DETAILED	G.D.T.	10-82
CHECKED	R.V.N.	12-83
REVISIONS	DEW	
FIELD CHANGES		

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

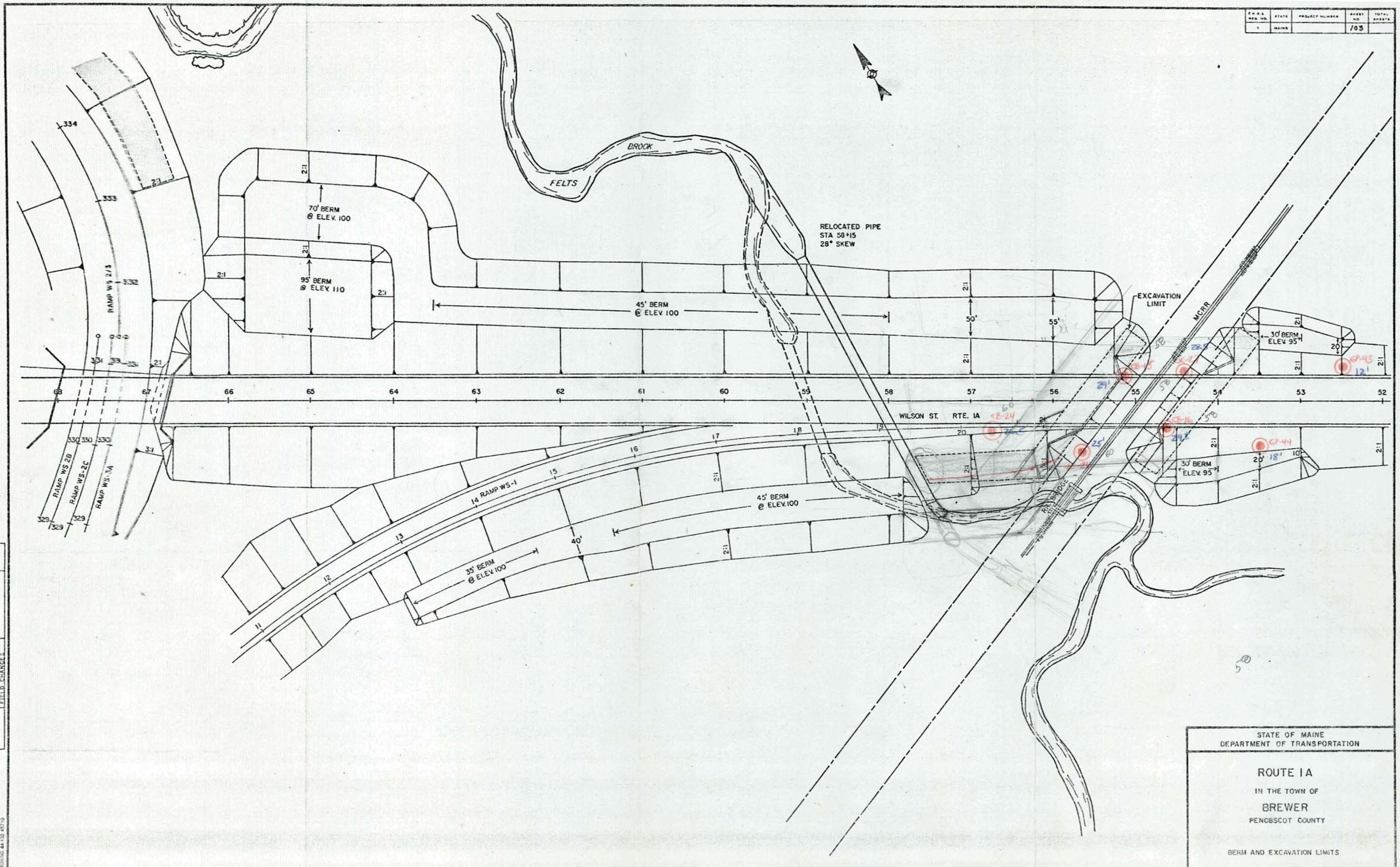
WILSON STREET
OVER
I-395
BREWER

CONCRETE SLOPE PROTECTION

182-70

SHEET 37 OF 121 AUGUSTA, MAINE

BRUNING 44-132 45710



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

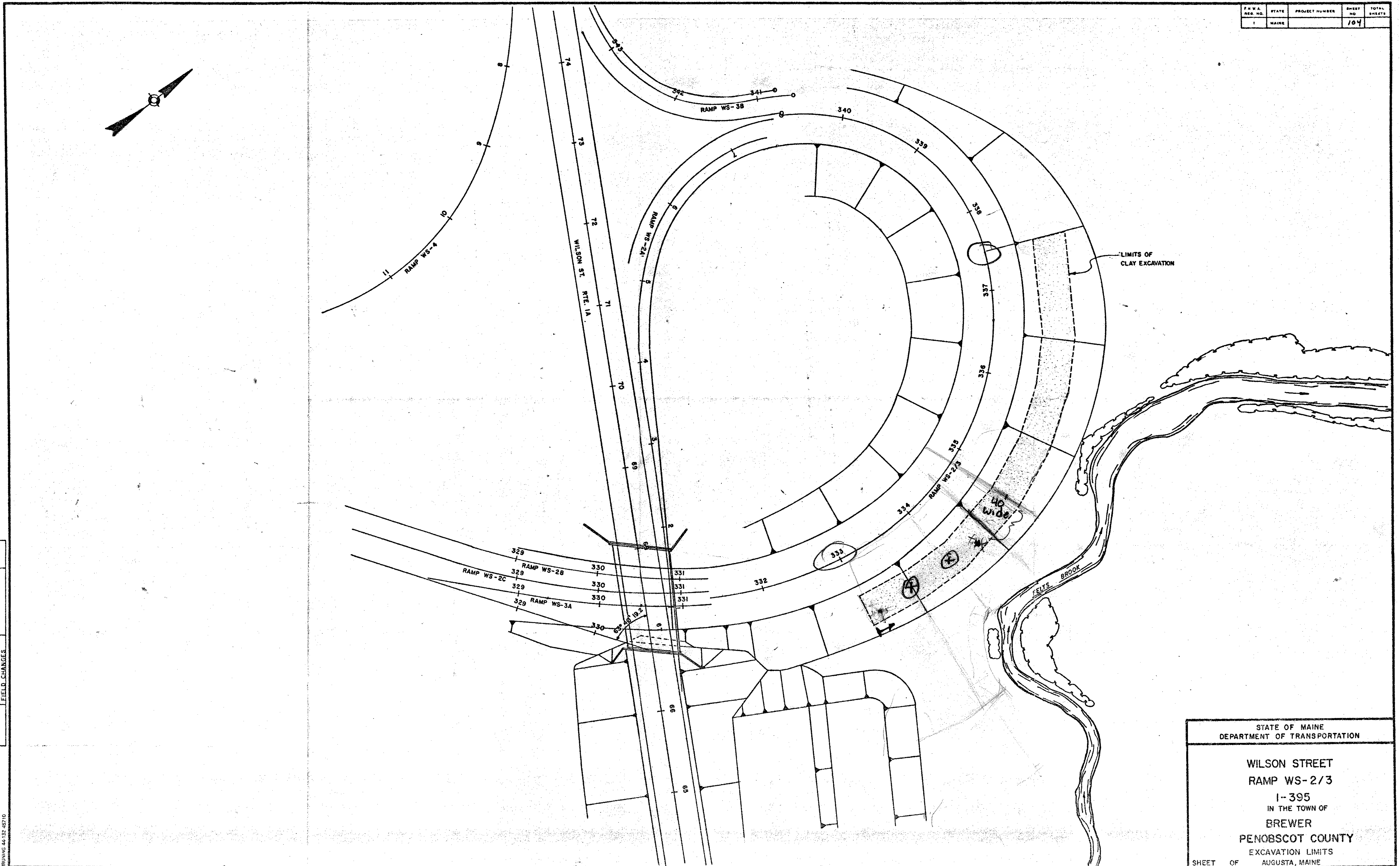
ROUTE 1A
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY

BERM AND EXCAVATION LIMITS

SHEET OF AUGUSTA, MAINE

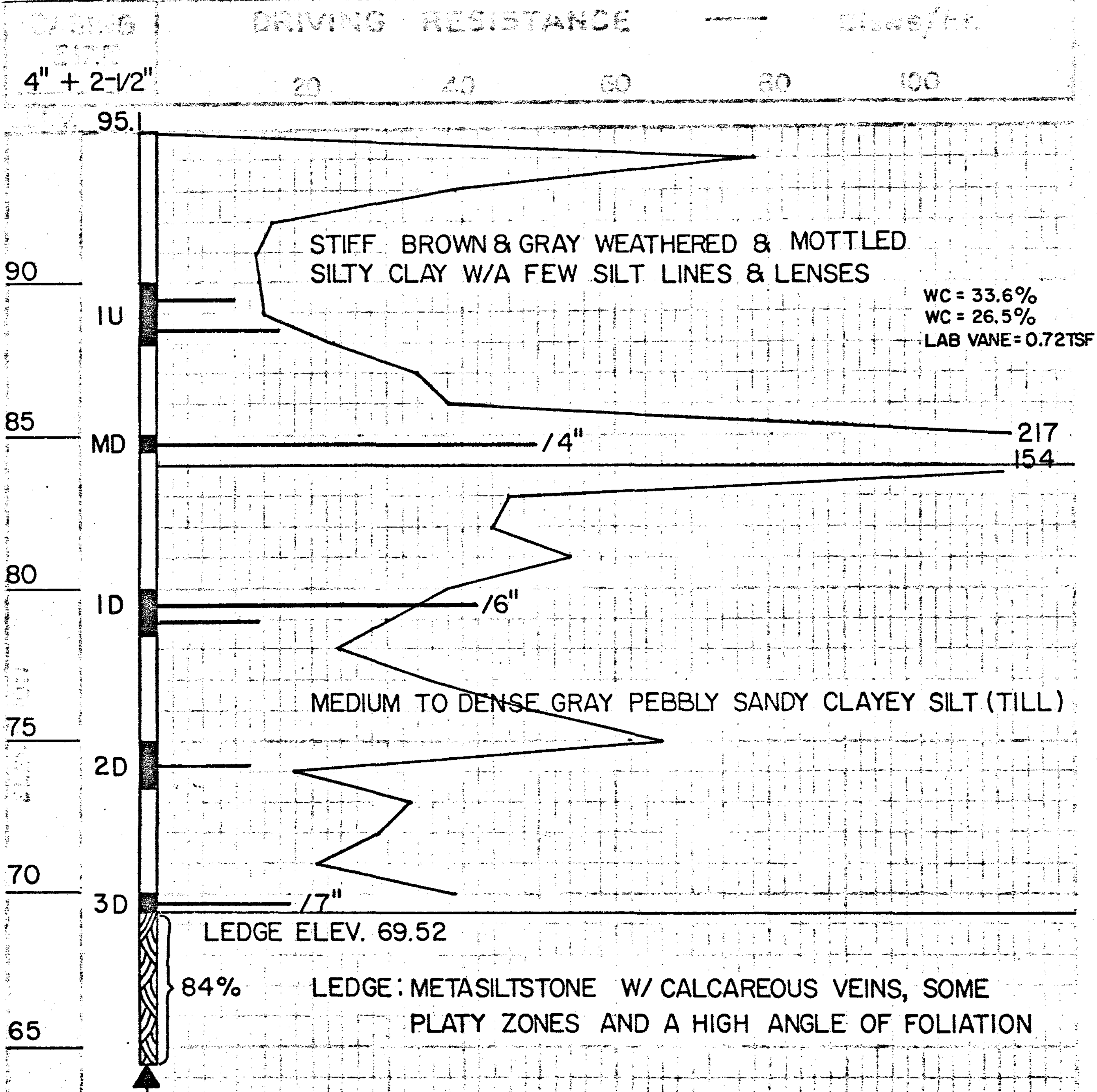
F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE		103	

PLANS	PROJECT DESIGN ENGINEER	BY	DATE
	DESIGN - DETAILED		
	CHECKED		
	REVISIONS		
	FIELD CHANGES		



F.H.W.A. REG. NO.
STATE
PROJECT NUMBER
SHEET NO.
TOTAL SHEETS
1
MAINE
104
RAMP WS-3B
RAMP WS-4
RAMP WS-2A
WILSON ST. RTE. 1A
RAMP WS-2/3
RAMP WS-2B
RAMP WS-2C
RAMP WS-3A
LIMITS OF CLAY EXCAVATION
40' wide
FELTS BROOK
PROJECT DESIGN ENGINEER
PLANS
DESIGN-DETAILED
CHECKED
REVISIONS
FIELD CHANGES
BY
DATE
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
WILSON STREET
RAMP WS-2/3
I-395
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY
EXCAVATION LIMITS
SHEET OF
AUGUSTA, MAINE
BRUNING 44-132 45710

BORING GP-14-80 STATION 67+50 75' RT



MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING GP-14-80

BREWER

395-8(79)

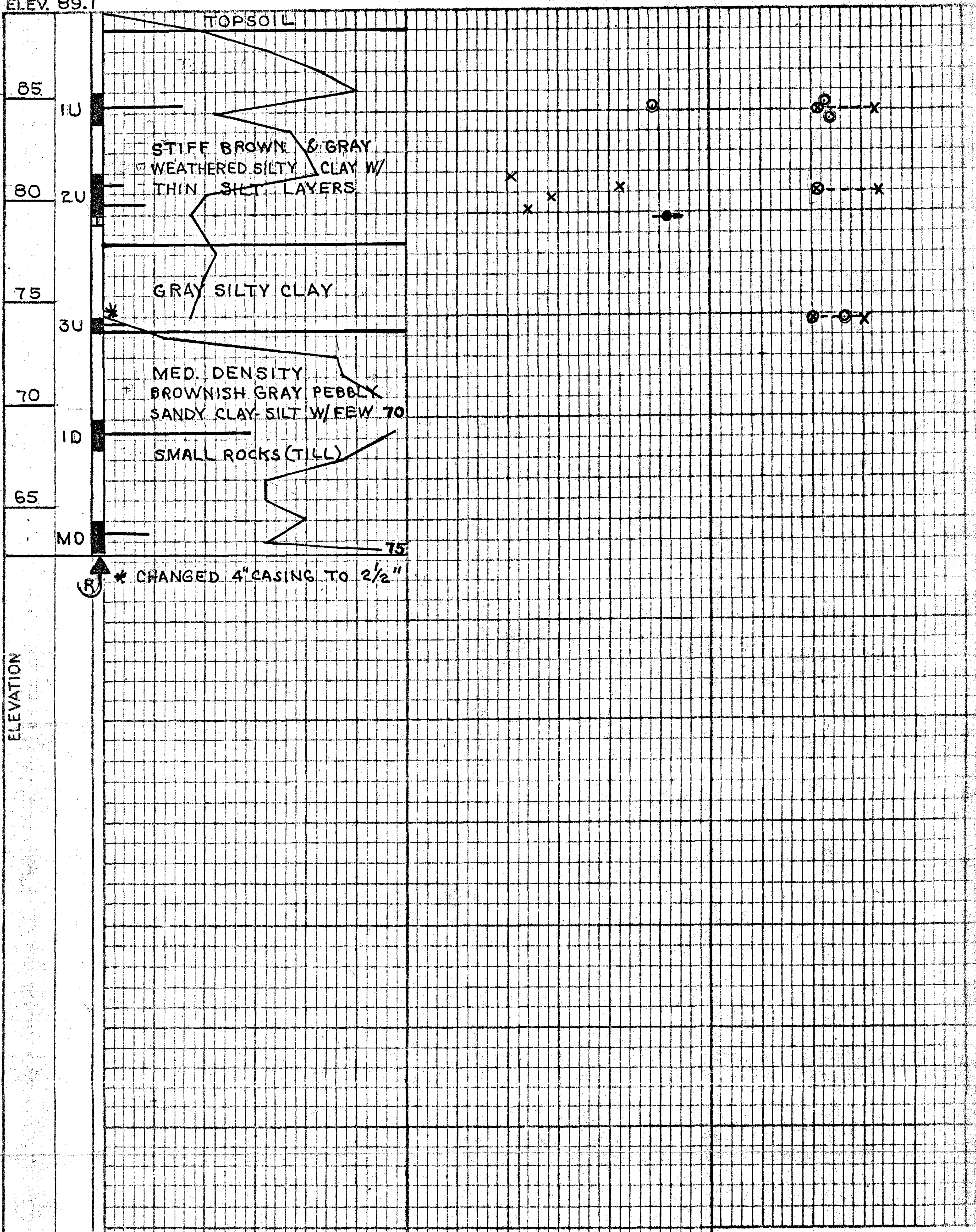
DATE: APRIL 1980

SML-201(9-62)

BORING GP-45-82   STATION 66+00 79' RT.

CASING SIZE 4" & 2 1/2"	DRIVING RESISTANCE Blows/Ft. 20 40	VANE SHEAR STRENGTH Tons/Sq.Ft. 0.4 0.8	WATER CONTENT Percent 20 40

ELEV. 89.1



MAINE STATE HIGHWAY COMMISS[ION]
SOILS LABORATORY

DETAILED SOIL STRATIFICAT[ION] & CONSISTENCY DATA

BORING GP-45-82

BREWER

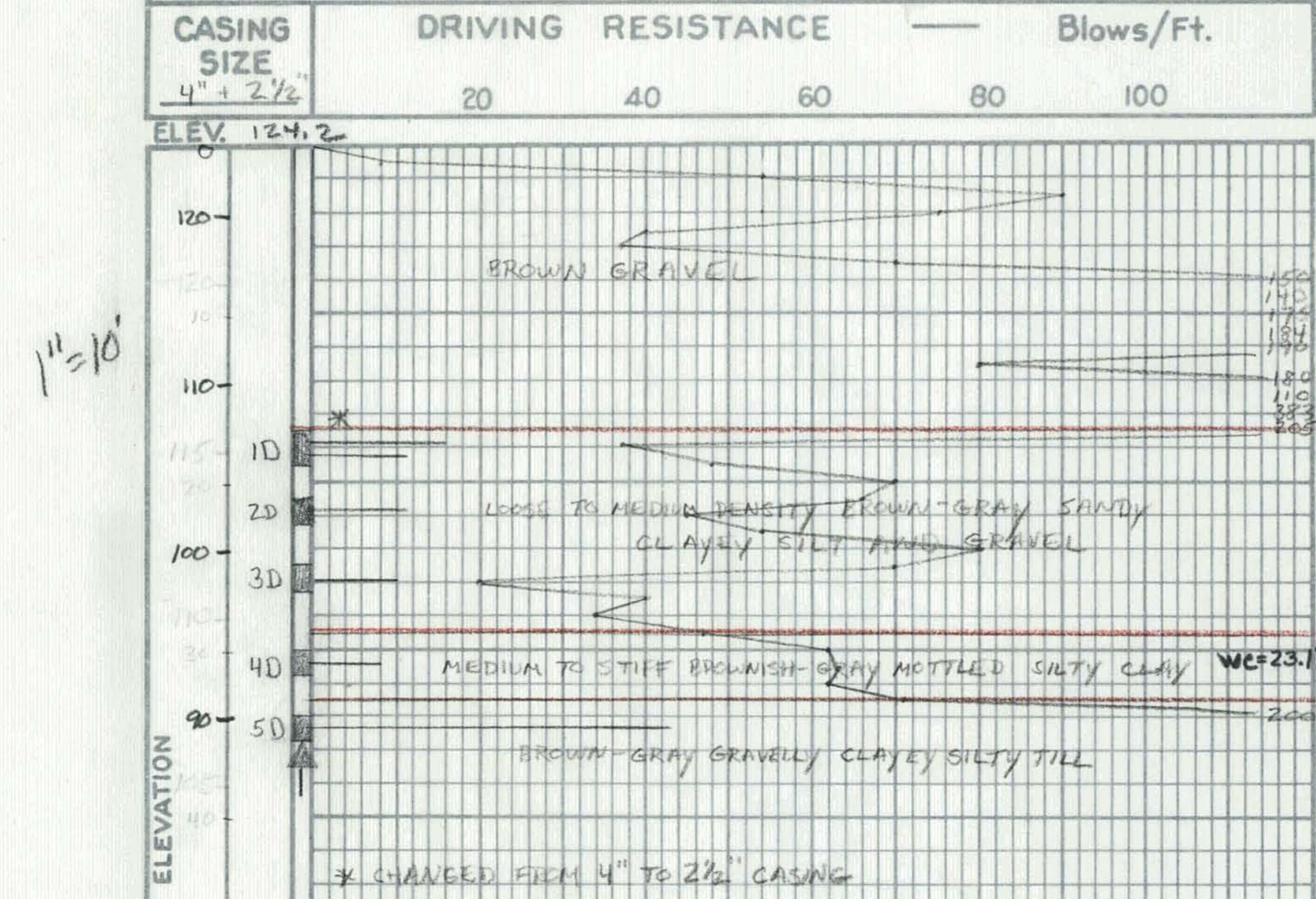
395-8 (79)

DATE: AUGUST 1982

18

Sheet No.

BORING GP-43-84 STATION ____________



MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING GP-43-84

DATE:

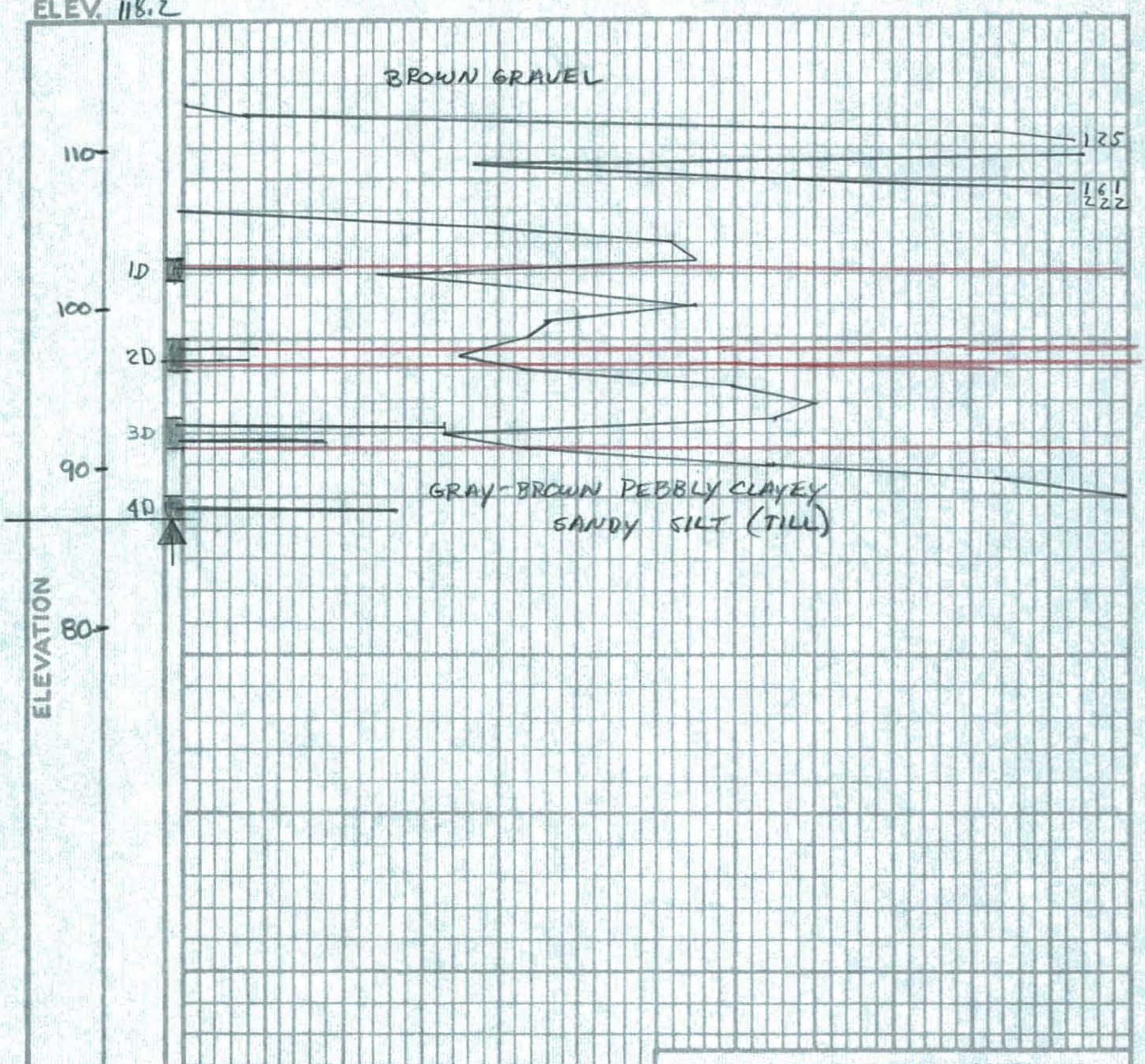
SML-202 (8-72)

Sheet No.

BORING GP-44-84 STATION

CASING SIZE 4" & 2½"	DRIVING RESISTANCE — Blows/Ft.
	20 40 60 80 100

ELEV. 118.2



MAINE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING GP-44-84

Brewer

DATE:

SML-202(8-72)

		6-13	6-21	From 5-17	From plan	6/29	From 5/17	Total
A2	P1	135.07	135.06	1/2	—	135.05	5/8	
	P2	134.87	.85	1/2	3/4	.84	5/8	7/8"
	P3	134.05	.03	1 1/4	2 1/4	.01	1 1/2	2 1/2"
	P4	—	132.98	2	—	Hitby dozer		
A1	P5	131.90	131.89	1/4	7/8"	131.88	3/8	1"
	P6	131.42	131.40	3/4	1"	131.37	1 1/8"	1 3/8"

Shims
1/2
7 3/4
8
9 1 1/8

SAMPLE NO.	LS NO.	STATION - OFFSET DEPTH	SHEET NO.	W.C.	LL NO.	LL	PL	PI	LI NO.	% IGN. LOSS	LpH NO.	pH	LP NO.	LBR NO.
GP43-84 4D					260	37.8	23.2	14.6						
GP44-84 2D					261	37.1	23.4	13.7						

ORIGINAL to SIEVE LAB ________ (DATE) 1ST COPY to SOILS MECH. LAB ________ (DATE) 2ND COPY to LAB FILE ________ (DATE) 3RD COPY to DRAFTING ROOM ________ (DATE)

GEOLOGIST SAMPLES

PROJECT NO. 395-8(79) TOWN(S) BREWER DATE ________

SML-211 (1/79)

MDOT MATERIALS & RESEARCH DATE 6/19

TO: Pete Thanks.

FROM : F. M. BOYCE ENGR. OF MATERIALS & RESEARCH

____ FOR YOUR INFORMATION ____ NOTE AND RETURN

____ FOR YOUR APPROVAL ____ DISCUSS WITH ME

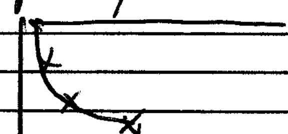
____ FOR YOUR COMMENTS, return ____ PLEASE ANSWER COPY TO ME

____ FOR YOUR SIGNATURE ____ PLEASE PROVIDE DATA FOR REPLY

____ FOR YOUR FILES ____ AS YOU REQUESTED

PLEASE RETURN ____ YES ____ NO BY DATE ____

COMMENTS: You should provide some settlement plots/time to give an



idea the rate of change

M & R-10 (3/82)

June 12, 1984

Larry Roberts — Design

Pete Coughlan — M & R Soils

Brewer - Wilson Street Over I-395

Several weeks ago it came to our attention that Abutment #2 of this bridge was undergoing some settlement. From observation and survey elevations of the bearing pads, the effects of this settlement can be seen most notably on the easterly half of the abutment. After reviewing the subsurface soil conditions in this area, we decided to do two additional borings adjacent to the footing and attempt to locate any areas of compressible material. The locations of the borings are shown on the accompanying plan and these borings encountered similar conditions to the 1982 washborings. That is, no distinct zone of compressible gray silty clay could be identified. The only potentially compressible material is the typical stiff gray and brown sandy clay-silt material that is located below the old fill embankment. This appears to range between 4 feet and 10 feet in thickness under the entire abutment length.

Using the maximum abutment deadloads and the embankment loads, the theoretical stress increases were calculated and applied to several corresponding pressure-void ratio curves from tests on stiff clay samples. In the first case, calculations were made for the conditions which existed over this past winter - namely, the nearly complete abutment sitting as a deadload at Elevation 113.7. The maximum toe pressure of 1.4 TSF produces a stress increase of 0.5± TSF within the stiff clay layer. Theoretically, this creates a consolidation of the clay layer of less than 1 inch. This corresponds roughly with the measured change in elevation of the easterly three bearing pads.

The second case includes the backfill material constructed to finished grade and this yields a stress increase of 0.75± TSF within the clay layer. This produces a consolidation of the clay layer of approximately 1 inch.

And the third case represents the completed structure and approach fills. The maximum toe load combined with the embankment load produces a total stress increase of 2.5± TSF in the stiff clay layer. This theoretically yields a total consolidation amount of 3± inches.

Thus, it is fair to say that a total settlement of approximately 3 inches can be anticipated for this abutment. As of May 22, 1984, it appeared that only the east wing and breastwall section were settling. Survey elevations of pads #1 through #6 were relatively unchanged between last November and May 22nd and pads #7, #8 and #9 showed amounts of 1/2 inch, 3/4 inch, and 1 1/8 inches, respectively. In the following two weeks after May 22nd, more settlement was recorded including some movement on the westerly end. As of June 5, 1984 settlement of 3/4 inch was measured on the west end (near pad #1) and the east end had increased to 1 7/8 inches (near pad #9). Thus, it can be seen that the east end has experienced a greater total settlement.

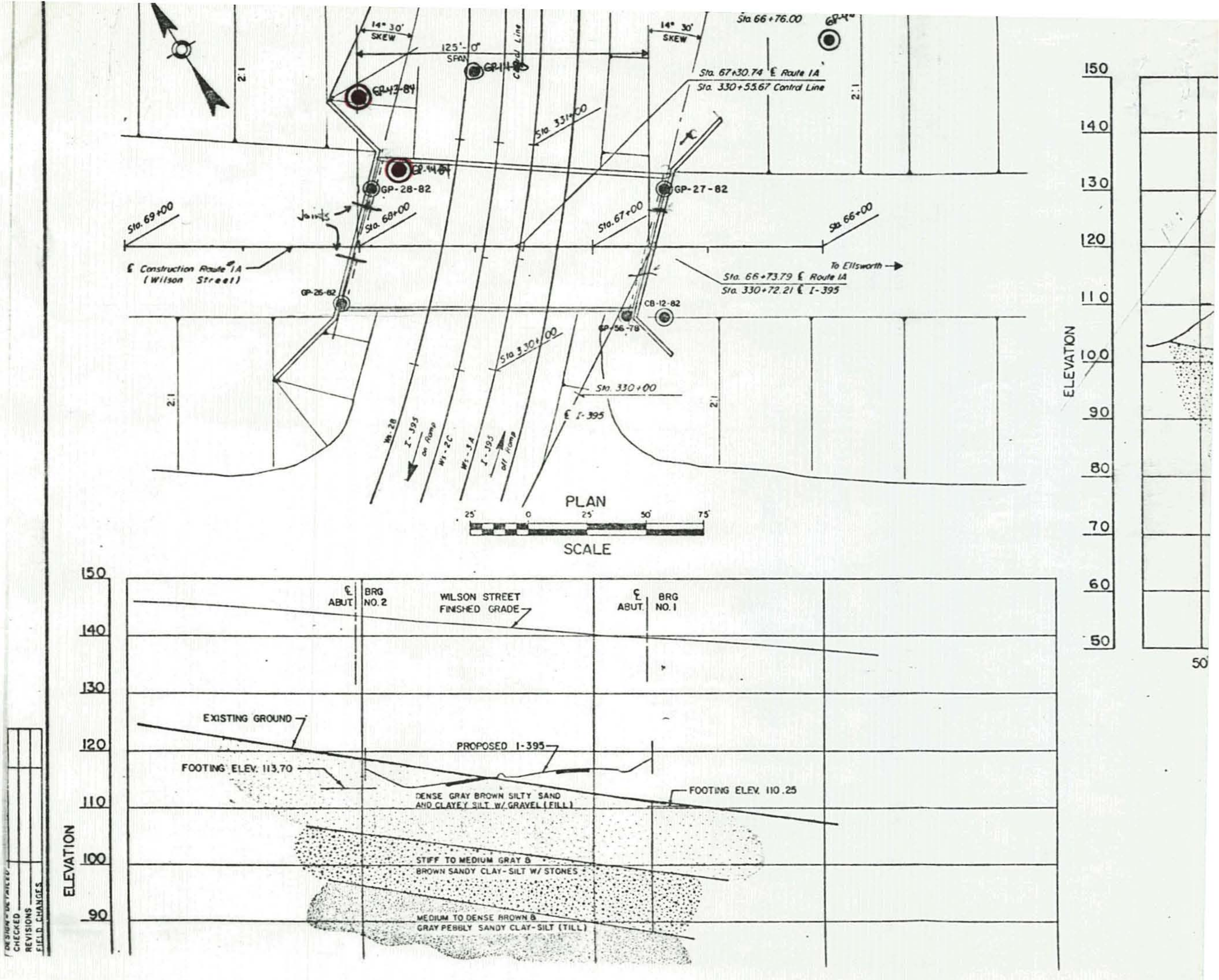
Calculations were also made to determine the average rate of consolidation of the clay layer. Because the clay thickness is relatively small and the coefficient of consolidation (Cv) is approximately 100 ft^2/year, then an average of 95 percent consolidation is expected within 4 months of loading. This estimate is rather approximate however, it does indicate that the rate of consolidation is rapid and the effect on the abutment will be short term.

In regards to Abutment #1, it appears that similar conditions are becoming apparent. As of May 22, 1984 more uniform elevation differences have been measured - generally 1/2± inch since November, 1983 - across the entire abutment length. As of June 5, 1984 total settlement of the west end measures 1 1/8 inches and that on the east end measured 1 1/4 inches.

Overall, it appears that consolidation of the underlying stiff, weathered silty clay is producing settlement of these two abutments as they are loaded and backfilled. Estimated total settlement is 3 inches and this should occur within a period of several months. Monitoring of the abutments' movements will continue as long as possible over the construction period.

nms

cc: Boyce
Morgan
Shailer
Zimmerman



14° 30' SKEW
125'-0" SPAN
14° 30' SKEW
Sta. 66+76.00
GP-43-84
GP-11-84
GP-44-84
GP-28-82
GP-27-82
GP-26-82
GP-56-78
CB-12-82
Control Line
Sta. 67+30.74 ℄ Route 1A
Sta. 330+55.67 Control Line
Sta. 331+00
Sta. 69+00
Sta. 68+00
Sta. 67+00
Sta. 66+00
Joints
℄ Construction Route #1A (Wilson Street)
To Ellsworth →
Sta. 66+73.79 ℄ Route 1A
Sta. 330+72.21 ℄ I-395
Sta. 330+00
Sta. 330+00
℄ I-395
I-395 on Ramp
I-395 off Ramp
2:1
PLAN
25' 0 25' 50' 75'
SCALE
℄ BRG ABUT. NO. 2
WILSON STREET FINISHED GRADE
℄ BRG ABUT. NO. 1
EXISTING GROUND
PROPOSED I-395
FOOTING ELEV. 113.70
FOOTING ELEV. 110.25
DENSE GRAY BROWN SILTY SAND AND CLAYEY SILT W/ GRAVEL (FILL)
STIFF TO MEDIUM GRAY & BROWN SANDY CLAY-SILT W/ STONES
MEDIUM TO DENSE BROWN & GRAY PEBBLY SANDY CLAY-SILT (TILL)
ELEVATION
150
140
130
120
110
100
90
80
70
60
50
50'
DESIGN - DETAILED
CHECKED
REVISIONS
FIELD CHANGES