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MEMORANDUM

7 May 2020
File No. 132076-005

TO: Maine Department of Transportation
Laura Krusinski, P.E.

FROM: Haley & Aldrich, Inc.
Erin A. Force, P.E., Wayne A. Chadbourne, P.E.

SUBJECT: Geotechnical Evaluations for Temporary Stockpile Area Adjacent to Wilson Street Bridge
Wilson Street (Route 1A) over Interstate 395, Bridge No. 1584
Interstate 395/Route 9 Connector Roadway Project
MaineDOT WIN 018915.00
Brewer, Maine

This memorandum provides the results of a recently completed supplemental subsurface exploration program and presents the results of geotechnical evaluations related to the proposed temporary stockpile area that is proposed to be located adjacent to Wilson Street Bridge in Brewer, Maine (see Figure 1, Project Locus). This work has been completed 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.

We presented the results of nearby subsurface explorations and summarized the soil, bedrock and groundwater conditions encountered at the Wilson Street Bridge in a preliminary geotechnical report dated 1 November 2019 and a final geotechnical design report dated 7 May 2020.

Introduction

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

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

BACKGROUND

Based on recent discussions with you, it is our understanding that MaineDOT would like to be able to allow the Contractor to temporarily stockpile embankment fill soils generated from excavation within the existing Wilson Street bridge eastern approach embankment in an area located adjacent to and north of the existing east approach embankment at the location shown on Figure 2, Site and Subsurface Exploration Location Plan. The temporary stockpile area is proposed to be approximately 120 by 300 ft in plan. The maximum fill height for stockpiled soils is proposed to be approximately 25 to 30 ft (maximum) above current site grades (El. 133) and the side slopes are proposed to be no steeper than 2H:1V.

As shown in the cross sections in Appendix D, the existing ground surface in Wilson Street ranges from approximately El. 117 to El. 123 adjacent to the proposed temporary stockpile area. The ground surface slopes down from the roadway at approximately 2H:1V to a bench at approximately El. 100, then slopes down again at approximately 2H:1V to the toe of the embankment at approximately El. 85. The temporary stockpile area is proposed to be located between the edge of Wilson Street to the edge of the bench.

Subsurface Exploration Program

SUPPLEMENTAL SUBSURFACE EXPLORATIONS

Haley & Aldrich completed a "supplemental" field investigation at the site to determine subsurface information at the proposed location of the temporary stockpile area. One test boring, designated BB-BWS-301, was drilled on 30 January 2020. The test boring location was laid out by MaineDOT. The "as-drilled" test boring location and ground surface elevation at the test boring location was 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 exploration is shown on the boring log included in Appendix A and the boring location is shown on Figure 2.

The test boring was drilled by S.W. Cole (SWC) of Bangor, Maine using a track-mounted Mobile Drill B-53 drill rig and was monitored by a Haley & Aldrich geologist. The test boring was drilled to a depth of approximately 42.1 ft below existing ground surface (BGS) using 4.0-in. (HW-size) inside diameter (ID) steel casing. Soil samples were collected 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., using a calibrated automatic hammer.

The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the test boring log. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval (blows per foot, bpf).

The drill rig was equipped with a calibrated automatic hammer. Based on the calibration information provided by SWC, a theoretical hammer efficiency factor of 0.977 was used for the automatic hammer.

The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (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.

The test boring was terminated on the top of likely bedrock. Bedrock was not cored but a roller bit was advanced approximately 1 ft below the last split spoon. The test boring was backfilled with drill cuttings upon completion of drilling, from the bottom of the test boring up to the existing ground surface.

All soil samples were collected and preserved in glass jars. 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.

Detailed descriptions of soil and bedrock encountered during drilling are provided on the test boring log included in Appendix A.

HISTORIC EXPLORATIONS

Multiple phases of explorations were conducted at the site by the Maine State Highway Commission in association with the design and construction of the existing bridge and approach embankments. A total of four "wash borings" were drilled in the vicinity of the proposed stockpile location.

- 1980 – one "wash boring", designated GP-63-80
- 1982 – three "wash borings", designated CB-13-82, CB-15-82, and CB-22-82

Test borings were drilled to depths ranging from approximately 21 to 42 ft BGS using 4-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 borings GP-63-80 and CB-15-82 were advanced approximately 5 to 10 ft into bedrock.

The locations of the historic explorations are shown on Figure 2 and logs providing information on subsurface conditions encountered in the "wash borings" are provided in Appendix B.

Generalized Subsurface Conditions

SOIL AND BEDROCK CONDITIONS

The subsurface conditions encountered in the proposed stockpile area consist of the following geologic units presented in order of increasing depth BGS: fill, marine deposit, glacial till, weathered bedrock and bedrock. A general description of each soil unit is provided below. Please note that the information presented below is limited to the conditions encountered in test boring BB-BWS-301, with additional strata thickness information included from historic borings where indicated.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
In-Situ Fill	14 (1 to 9)[1]	Medium stiff to stiff SILT (ML), varying amounts of fine to coarse sand and gravel
Marine Deposit	22 (11 to 24)[1]	Medium stiff to stiff Silty CLAY (CL), varying amounts of fine to coarse sand, trace organics
Glacial Till	5 (2 to 12)[1]	Medium dense to dense fine to coarse SAND (SM), some gravel, little silt. Loosely to well bonded.
Weathered Bedrock	< 1	A thin layer of weathered bedrock was encountered.
Bedrock	NA	The top of bedrock was encountered at a depth of approximately 41.1 ft BGS (El. 59.4), based on spoon refusal and rollerbit cuttings. (Historic borings encountered the top of bedrock at depths of 36 to 37 ft.)

[1] Thickness encountered in historic borings.

GROUNDWATER CONDITIONS

An observation well was not installed in the recently completed borehole. As a result, static groundwater levels in the vicinity of the proposed stockpile were not determined. However, a water level of 12 ft BGS (El. 88.5) was encountered during drilling. Please note that this water level may have been affected by drilling means/methods and may not be representative of actual static groundwater levels at the site.

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 and provided on the testing boring log included in Appendix A.

Laboratory Test Results

A geotechnical laboratory testing program was undertaken by Haley & Aldrich on representative soil samples collected from boring BB-BWS-301 to aid in soil classification and determination of engineering soil properties (i.e, strength and compressibility). 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 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	4	WC = 24% to 33%
Grain Size	ASTM D422	Existing Fill	2	AASHTO Classification: A-4 USCS Classification: ML
Atterberg Limits	ASTM D4318	Marine Deposit	11	31 < LL < 36 19 < PL < 21
One-Dimensional Consolidation[2]	ASTM D2435 (Method B)	Marine Deposit	1	Preconsolidation Pressure ($\sigma_{p'}$): 2,800 psf Overconsolidation Ratio (OCR): 1.2 Recompression Ratio (RR): 0.03 Virgin Compress. Ratio (CR): 0.16 Virgin Secondary Compression Ratio ($C_{\alpha\varepsilon}$): 0.0003 Recompression Coefficient of Consolidation (c_v):0.06 ft^2/day Virgin Coefficient of Consolidation (c_{vr}): 0.06 ft^2/day
Consolidated Undrained Triaxial	ASTM D4767	Marine Deposit	1	Undrained Shear Strength (S_u): 843 psf Strain at Failure: 9.2%

Notes:
[1] LL = Liquid Limit; PL = Plastic Limit; WC = Moisture Content; psf = pounds per square ft
[2] Consolidation test results are interpretations from laboratory test data.

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

Geotechnical Evaluations and Design Recommendations

GLOBAL EMBANKMENT STABILITY

As described above, MaineDOT would like to allow the Contractor to temporarily stockpile soils excavated from the eastern approach embankment of the existing Wilson Street bridge in a designated area adjacent to and north of the existing east approach embankment. The temporary stockpile is proposed to be up to 25 to 30 ft above existing site grades with side slopes no steeper than approximately 2H:1V. Fill placement on and adjacent to the existing embankment changes the loading conditions on the embankment, and could cause a global stability failure. As a result, a series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 7.0 to evaluate the likelihood of embankment global stability failures.

Static and seismic stability evaluations were conducted transverse to the existing approach roadway east of the existing Wilson Street bridge. A typical soil profile was developed based on the subsurface conditions encountered in the recently completed and historic explorations. Soil strength properties

were determined based on empirical correlations with soil type and SPT N-values and our experience. In addition, a 250 psf live load surcharge was assumed to act at the crest of the embankment in Wilson Street. Soil properties used in our stability evaluations are summarized below.

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Proposed Stockpile Fill	120	30	0
In-Situ Fill	120	30	0
Marine Deposit (Upper)	115	0	1,500
Marine Deposit (Lower)	115	0	800
Glacial Till	130	38	0

The calculated minimum global factors of safety for the stockpile conditions described above are provided in the table below:

	Static Factor of Safety	Seismic Factor of Safety
Stockpile Area	1.4	1.2

Please note that the factors of safety reported above are for deep-seated failure surfaces passing into the marine clay deposit below the in-situ fill soils. There were also shallow, surficial failure surfaces with a factor of safety between 1.2 and 1.3 passing through the stockpile fill only.

The factor of safety for pseudo-static seismic load cases were calculated using a horizontal acceleration coefficient, k_h, of 0.05g (i.e., one half of the acceleration coefficient, As). The acceleration coefficient was determined based on the geographic site location and "Site Class D" designation based on boring BB-BWS-301 and 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). A value of As/2 was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2; the reduction from As 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 permanent 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). Therefore, the calculated factors of safety are acceptable under both static and seismic conditions. Based on the conditions encountered in the borings and results of our evaluations it is our

opinion that placement of the proposed stockpile will not cause a deep-seated global stability failure of the existing embankment.

However, due to the marginal factors of safety for shallow failure surfaces, it may be necessary for the Contractor to more systematically place and compact the material along the outer edges of the stockpile to maintain stable surficial slopes and prevent shallow instabilities and raveling of the stockpiled soils. Additionally, silt fencing and haybales should be installed at the toe of the embankment to limit siltation/migration of stockpiled soils into adjacent wetlands caused by surficial raveling. Also if the temporary soils are to remain in place for an appreciable amount of time, it may make sense to require the Contractor to place a 4 to 6-in. thick layer of mulch on the top and sideslopes of the stockpile for dust control.

EMBANKMENT SETTLEMENT

Because of the subsurface condition present at the site, stockpiling of soils on and adjacent to the existing embankment will cause settlement of the existing embankment. Based on the recent test boring completed within the footprint of the proposed stockpile, existing fill soils overlay compressible marine clay deposits. Accordingly, we have evaluated settlements due to the proposed stockpile that will be placed during construction of the new Wilson Street bridge.

The computer analyses software, Settle3D Version 4, developed by RocScience, Inc., was used to evaluate settlements due to the proposed stockpile. The following marine deposit compressibility parameters and assumptions were used in the settlement analyses:

- Subsurface conditions were based on test boring BB-BWS-301.
- Settlement was evaluated assuming the stockpile could be in place for a period of up to 1 year.
- The following marine deposit compressibility parameters were selected based on laboratory consolidation results:
  - Preconsolidation Pressure (σ_p') = 2.8 ksf
  - Overconsolidation Ratio (OCR) = 1.2
  - Initial Void Ratio (e_0) = 0.89
  - Coefficient of Compression (C_c) = 0.29
  - Virgin Compression Ratio (CR): 0.16
  - Coefficient of Recompression (C_r) = 0.05
  - Recompression Ratio (RR): 0.03
  - Coefficient of Consolidation (c_v): 0.06 ft^2/day

Based on the above assumptions and the results of our evaluations, we estimate magnitudes of total settlement as summarized below.

Time after Stockpile Placement (months)	At Center of Stockpile		
	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)
6	4.5	2.5	7.0
9	4.5	3.0	7.5
12	4.5	3.5	8.0

Time after Stockpile Placement (months)	At Northern Edge of Wilson Street Shoulder			At Centerline of Wilson Street		
	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)
6	0.5	0.5	1.0	<1/4	<1/4	1/4
9	0.5	0.5	1.0	<1/4	<1/4	1/4
12	0.5	0.5	1.0	<1/4	1/4	1/4

Please note that the majority of the anticipated settlement comes from elastic compression of the embankment fill, which will occur relatively quickly during placement of stockpiled soils. Based on the results of the evaluations, it is our opinion that the settlement caused by placement of the proposed stockpile soils will be tolerable given that the magnitude of anticipated settlement at the northern edge of the Wilson Street shoulder will be limited to 1 in. and settlement at the centerline of Wilson Street will generally be less than 1/4 in. We anticipate that this magnitude of settlement will be acceptable to MaineDOT.

Closure

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

Enclosures:

- Figure 1 – Project Locus
- Figure 2 – Site and Subsurface Exploration Location Plan
- Appendix A – Test Boring Log
- Appendix B – Historic Test Boring Location Plan and Logs
- Appendix C – Laboratory Test Results
- Appendix D – Calculations

Figures

MAP SOURCE: ESRI

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

HALEY ALDRICH

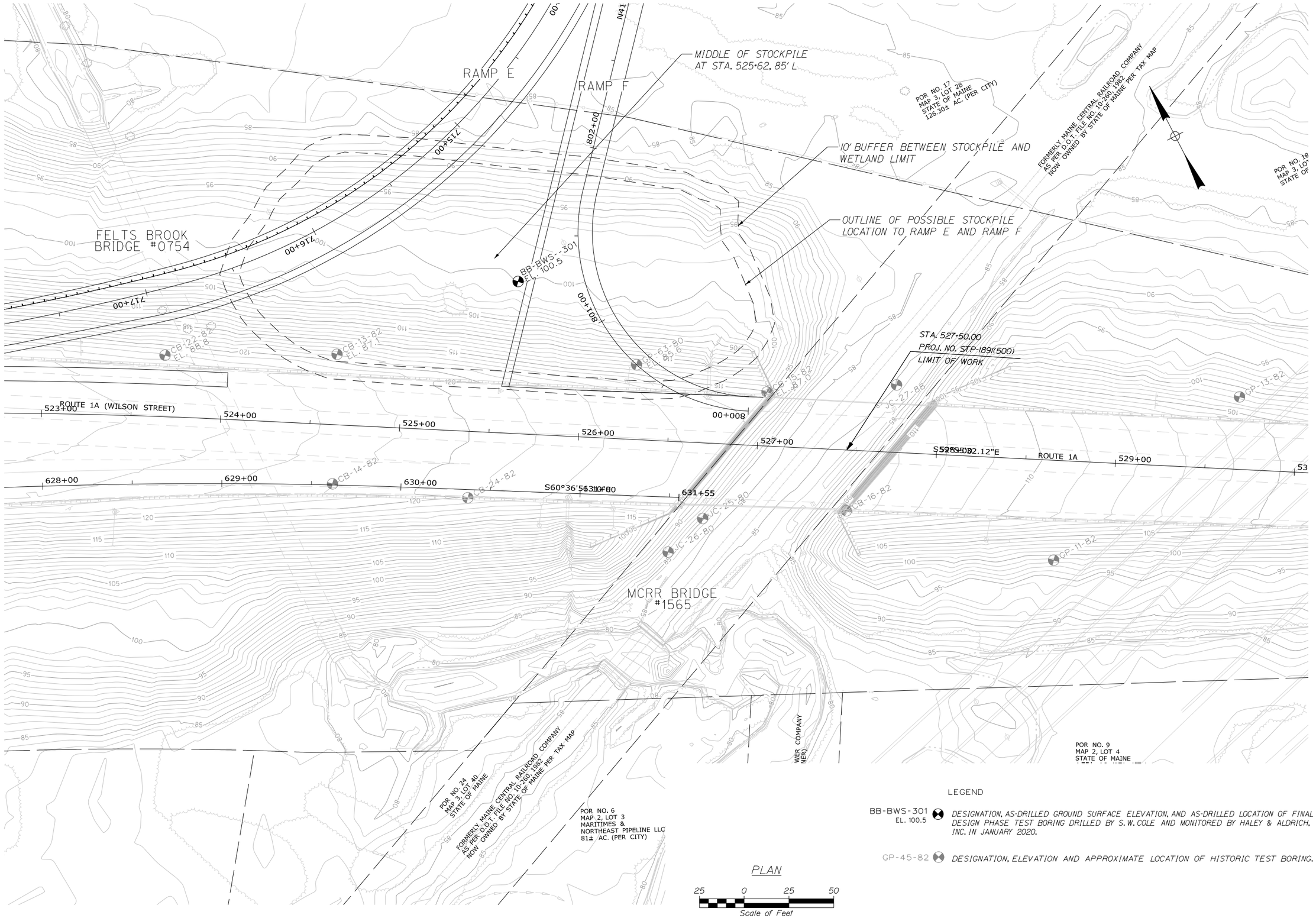
TEMPORARY STOCKPILE AREA
WILSON STREET (ROUTE 1A) OVER INTERSTATE 395
BRIDGE NO. 1584
MAINEDOT WIN 018915.00
BREWER, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
MAY 2020

FIGURE 1

132076-003_1_LOCUS.PDF



PLAN

25 0 25 50

Scale of Feet

LEGEND

BB-BWS-301 EL. 100.5 — DESIGNATION, AS-DRILLED GROUND SURFACE ELEVATION, AND AS-DRILLED LOCATION OF FINAL DESIGN PHASE TEST BORING DRILLED BY S.W. COLE AND MONITORED BY HALEY & ALDRICH, INC. IN JANUARY 2020.

GP-45-82 — DESIGNATION, ELEVATION AND APPROXIMATE LOCATION OF HISTORIC TEST BORING.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(5000)

WIN 018915.00
BRIDGE NO.

HIGHWAY PLANS

		BY	DATE
PROJ. MANAGER	----------	----------	----------
DESIGN-DETAILED	E. FORCE	K. POST	4-2-20
CHECKED-REVIEWED	----------	----------	----------
DESIGN2-DETAILED2	----------	----------	----------
DESIGN3-DETAILED3	----------	----------	----------
REVISIONS 1	----------	----------	----------
REVISIONS 2	----------	----------	----------
REVISIONS 3	----------	----------	----------
REVISIONS 4	FIELD CHANGES	----------	----------

SIGNATURE

P.E. NUMBER

DATE

BREWER - EDDINGTON
I-395/ROUTE 9 CONNECTOR

SITE AND SUBSURFACE
EXPLORATION LOCATION PLAN

SHEET NUMBER

2

OF 2

Date:4/10/2020

Username:

Filename: ...\008 Wilson Street RampStockpile.dgn Division:

Appendix A

Test Boring Log

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 ☐		

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	0.0 - 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		11.7	
	4D	24/22	12.0 - 14.0	3/3/3/3	6	10	46	88.5		Brown, fine to coarse SAND -FILL-(SM)	
							48			12.0 Note: Water encountered at 12.0 ft.	
15	1U	24/24	14.0 - 16.0				53	86.8		Brown to grey, wet, medium stiff, SILT, some fine to coarse sand, little gravel -FILL-(ML)	
							50			13.7 Note: 1U - Little fine to medium sand within CLAY matrix observed in top and bottom of tube sample.	
	2U	24/19	16.0 - 18.0				66			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)	
							57				
							87				
20	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
							137				
							183				
							209				
							193				
25	6D	24/24	24.0 - 26.0	2/3/4/4	7	11	OPEN			Similar to 5D above	G#545428 LL=35, PL=20

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.

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 ☐		

Definitions:
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HSA = Hollow Stem Auger
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WOH = Weight of 140 lb. Hammer
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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
										Note: Grey, Silty CLAY in wash water at 29.0 ft.	
								71.5		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)	G#545429
	V3		34.0 - 35.0	Su=790/235 psf						-MARINE DEPOSIT-(CL)	LL=31, PL=19
35	V4		35.0 - 35.8	Su=650/235 psf						55x110 mm vane raw torque readings:	PI=12, WC=33
								64.7		V3: 17/5 ft lbs	
										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	
40										-GLACIAL TILL-(SM)	
								59.7		40.8	
								59.4		Note: Probable weathered bedrock based on drill action.	
										-PROBABLE WEATHERED BEDROCK-	
								58.4		41.1	
										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	
45										**Bottom of Exploration at 42.1 feet below ground surface.**	
50											

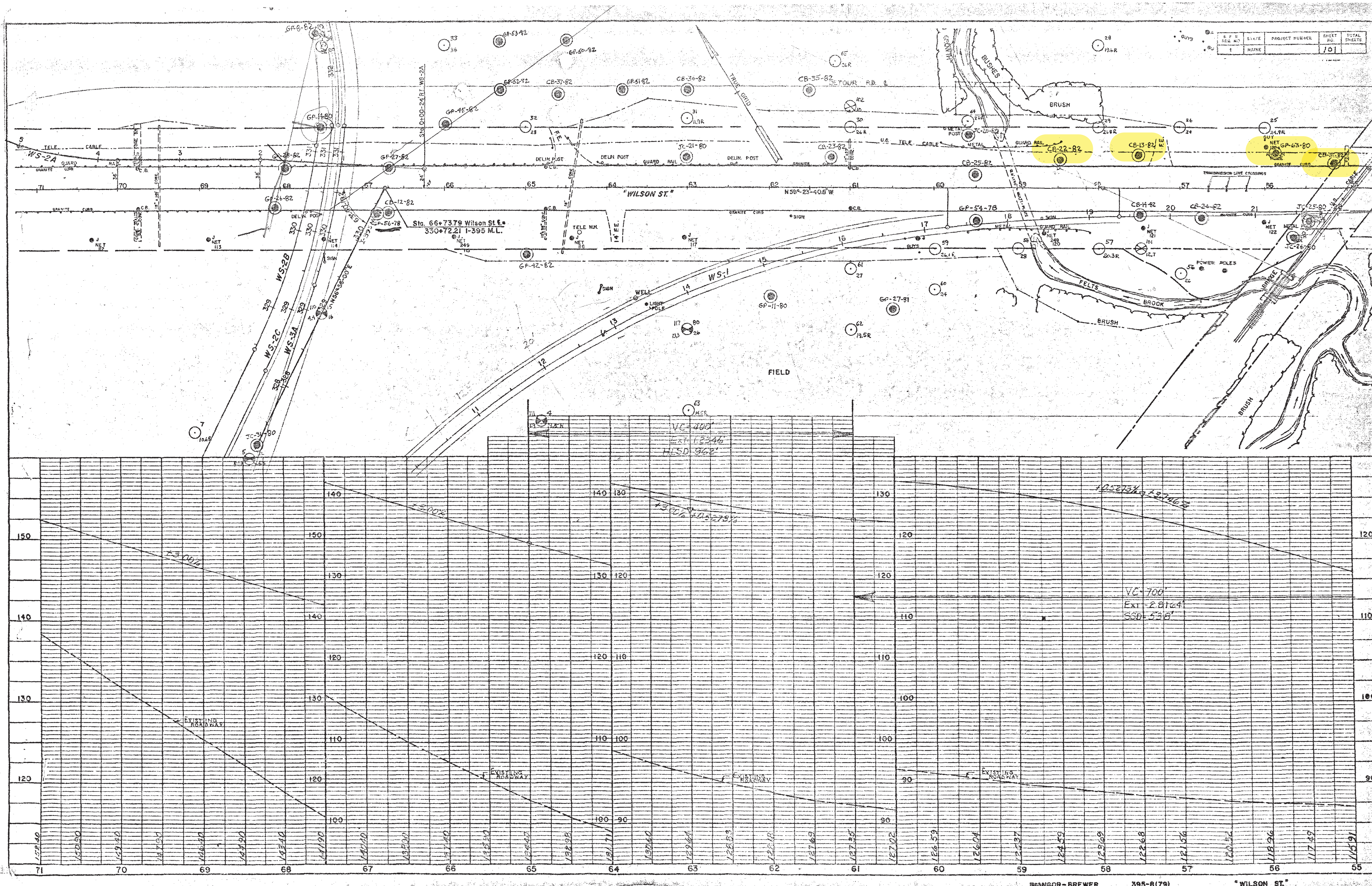
Remarks:

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

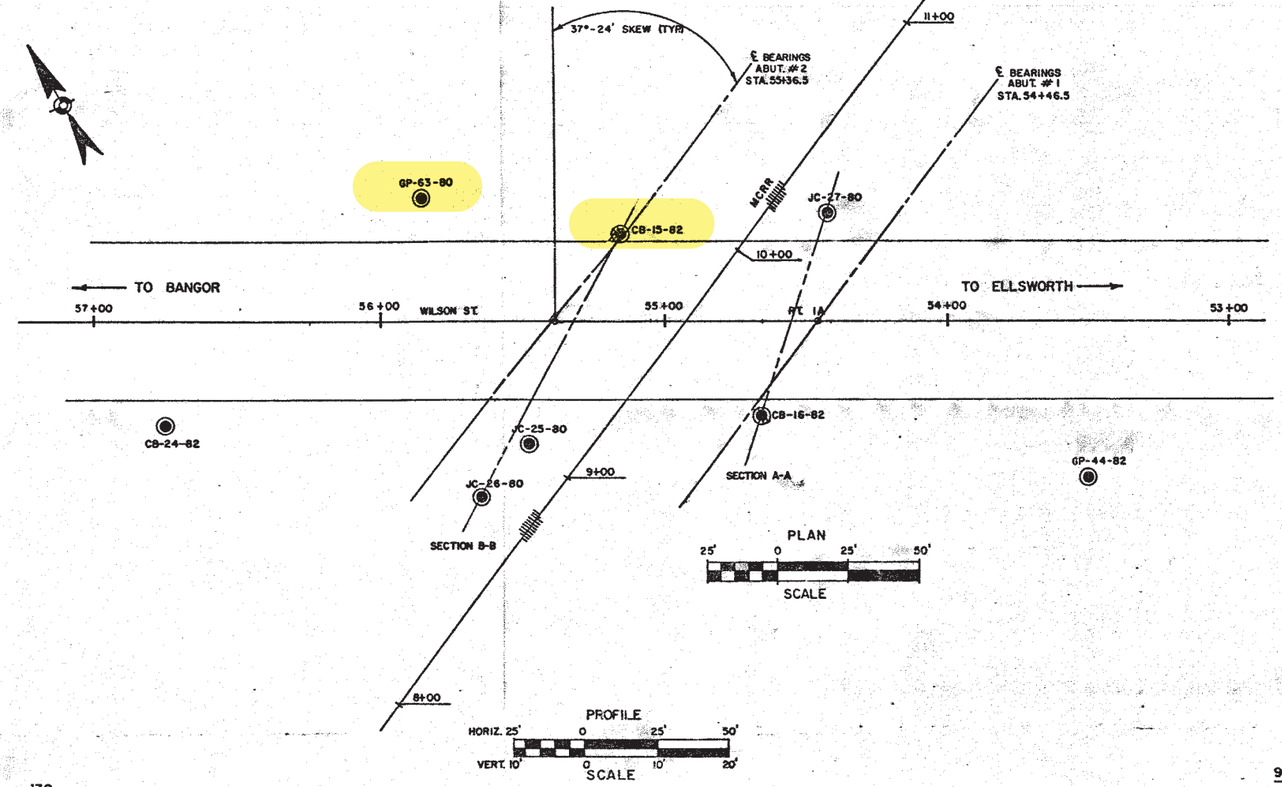
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

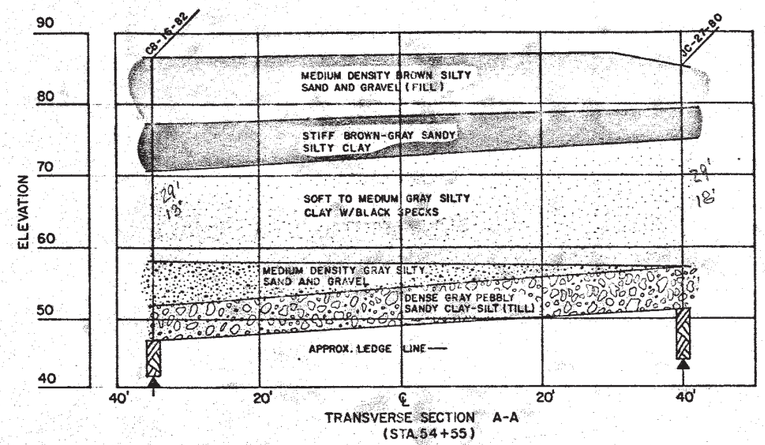
Appendix B

Historic Test Boring Location Plan and Logs

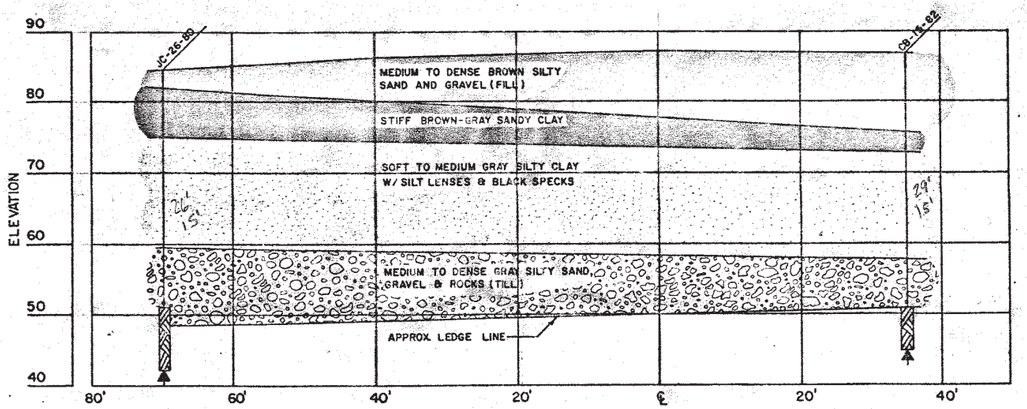

"WILSON ST."
N 58°-23-40.6 W
Sta. 66+73.79 Wilson St. E
330+72.21 I-395 M.L.
WS-2A
WS-2B
WS-2C
WS-3A
WS-1
FIELD
FELTS BROOK
BRUSH
BRIDGE
DETOUR RD. 1
TRUE GRID
TELE. CABLE
GUARD RAIL
GRANITE CURB
DELIN. POST
POWER POLES
WELL
LIGHT POLE
SIGN
CB-22-82
CB-13-82
GP-63-80
GP-54-78
GP-27-82
GP-11-80
GP-27-81
GP-42-82
VC-400'
VC-700'
+3.00%
+0.5275%
EXISTING ROADWAY
BANGOR-BREWER
395-6(79)
"WILSON ST."
MAINE
101

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	395-8(79)	15	





TRANSVERSE SECTION A-A
(STA. 54+55)



TRANSVERSE SECTION B-B
(STA. 55 + 30)

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
MCRR
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY

FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE

F.A.P. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE		14	

334
333
332
331
330
329
RAMP WS 2/3
RAMP WS 2B
RAMP WS 2C
RAMP WS 3A
2:1
3:1
70' BERM @ ELEV. 100
95' BERM @ ELEV 110
45' BERM @ ELEV 100
35' BERM @ ELEV 100
40'
14 RAMP WS-1
FELTS BROOK
RELOCATED PIPE
STA 58+15
28° SKEW
EXCAVATION LIMIT
MCRR
30' BERM ELEV. 95
20'
10'
50'
55'
WILSON ST. RTE. 1A
RR BRIDGE
68 67 66 65 64 63 62 61 60 59 58 57 56 55 54 53 52
17 18 19 20 21 15 16 12 13 11

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

BRIPING 44 132 45710

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WILSON ST.
OVER
MCRR
IN THE TOWN OF
BREWER
PENOBSCOT COUNTY
BERM AND EXCAVATION LIMITS

SHEET OF AUGUSTA, MAINE

SML-201(9-62)

BORING GP-63-80 STATION 55+85 43'RT.

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

ELEV. 85.6

BROWN GRAVEL (FILL)

MED. CONSISTENCY GRAY SANDY SILTY CLAY & BROWN ORGANIC SILT OVER STIFF SANDY SILTY CLAY W/SMALL SAND LENSES

DENSE GRAY SILTY SANDY GRAVEL (TILL)

LEDGE ELEV. = 48.7

LEDGE: METASILTSTONE W/CALCAREOUS ZONES PHYLLITE ZONES & IMPURE SILICEOUS METASILTSTONE (HIGH ANGLE OF FOLIATION)

96%

* TOOK SAMPLES 3U & 4U INSIDE CASING, PULLED CASING & MOVED OVER & RESTARTED BORING

ELEVATION			
80	1U	H	72
75	2U	H	72
70	3U	H	64
	5U	H	
65	4U	H	72
	6U	H	
60	1D		64
			92
			222
55	2D	1/4"	212
			69
			170
			342
50			500
45			

IGN LOSS 4.6
IGN LOSS 10.0
IGN LOSS 29.2
IGN LOSS 30.7

106
161

MAINE STATE HIGHWAY COMMISSION
SOILS LABORATORY

DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING GP-63-80

BREWER
395-8(79)

DATE: APRIL 1981

SHEET NO. 61

SWL-20(3)-82)

BORING CB-13-82 STATION 57+50 40'RT.

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

ELEV. 87.1

85

80

75

70

65

ELEVATION

MD

MED. DENSITY BROWN SAND & GRAVEL (FILL)

160
161
117

IU

STIFF GRAY-BRN MOTTLED SILTY CLAY W/ SILT LENSES

H

MED CONS. GRAY SILTY CLAY W/ FAINT BLACK SPECKS

ID

MED. DENSITY GRAY PEB. SILTY SAND

110

MAINE STATE HIGHWAY COMMISSION
SOILS LABORATORY

DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING CB-13-82

BREWER
395-8(79)

DATE: MARCH 1982

BORING CB-15-82 STATION 55+15 30' RT.

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

ELEV. 87.0

85

MEDIUM TO DENSE BROWN SILTY SAND & GRAVEL (FILL) — 91, 88, 82, 60

80 — ID

75 — MU

STIFF GRAY SANDY CLAY & GRAVEL

70 — IU

MEDIUM GRAY SILTY CLAY W/ SILT LENSES & BLACK SPECKS — H

65 — 2U — H

60 — MU — H

(SANDY) — 88, 76

55 — 2D — 380

DENSE GRAY SILTY SAND & ROCKS

47%

50

LEDGE ELEV. 51.0

LEDGE: METASILTSTONE W/ HIGH ANGLE FOLIATION

83%

45

ELEVATION

MAINE STATE HIGHWAY COMMISSION
SOILS LABORATORY

DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING CB-15-82

BREWER
395-8(79)
DATE: MARCH 1982

SHEET NO. 35

SML-201(9-62)

BORING CB-22-82 STATION 58+48 34' RT.

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

ELEV. 88.8

BROWN SILTY SAND & GRAVEL & CLAY LAYERS (FILL)

142
167
180
63

STIFF BROWN SILTY CLAY

MED. CONSISTENCY GRAY SILTY CLAY

79

GRAY SAND & GRAVEL

ID

IU

85
80
75
70

ELEVATION

MAINE STATE HIGHWAY COMMISSION
SOILS LABORATORY

DETAILED SOIL STRATIFICATION
&
CONSISTENCY DATA
BORING CB-22-82

BREWER
395-8 (79)

DATE: MAY 1982

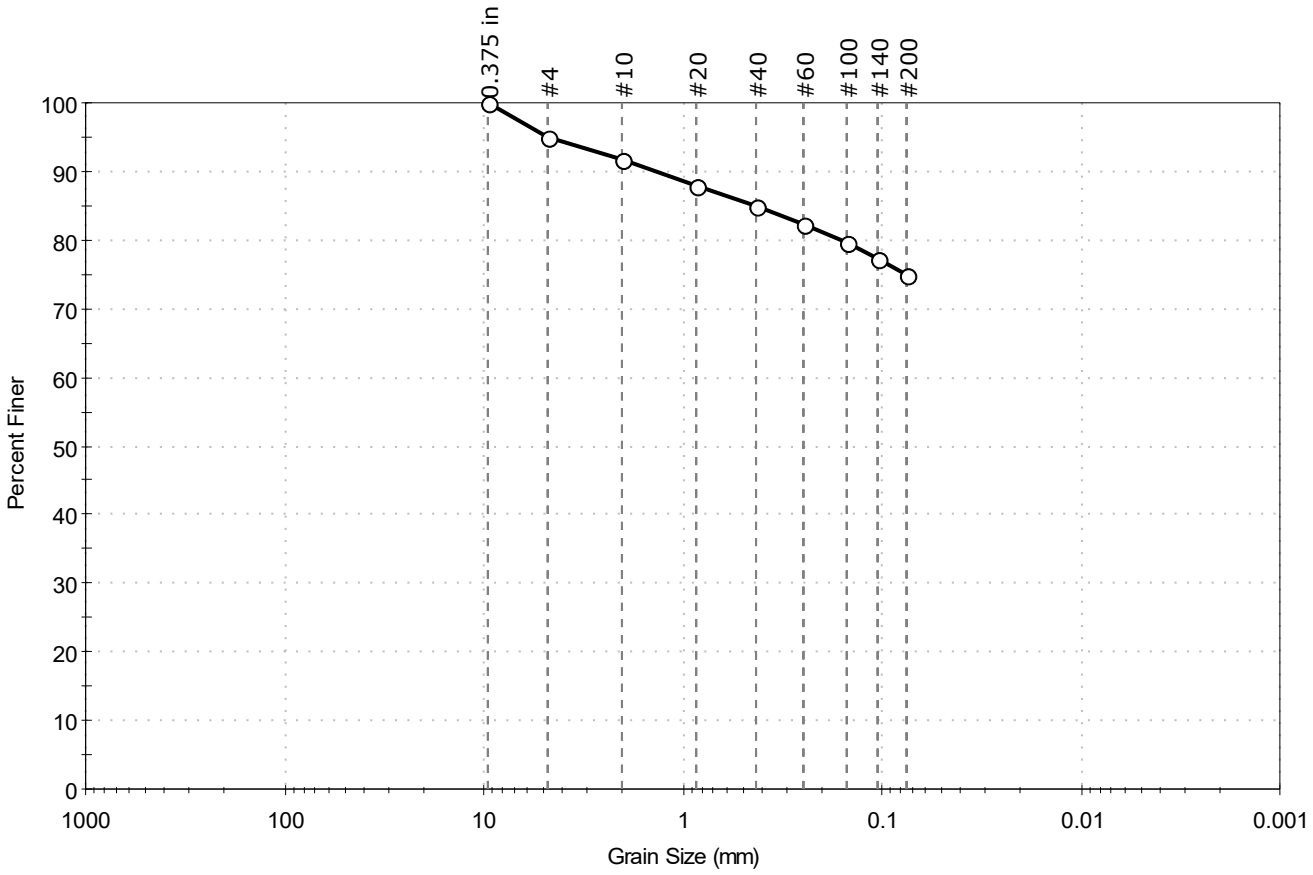
SHEET NO. 37

Appendix C

Laboratory Test Results

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

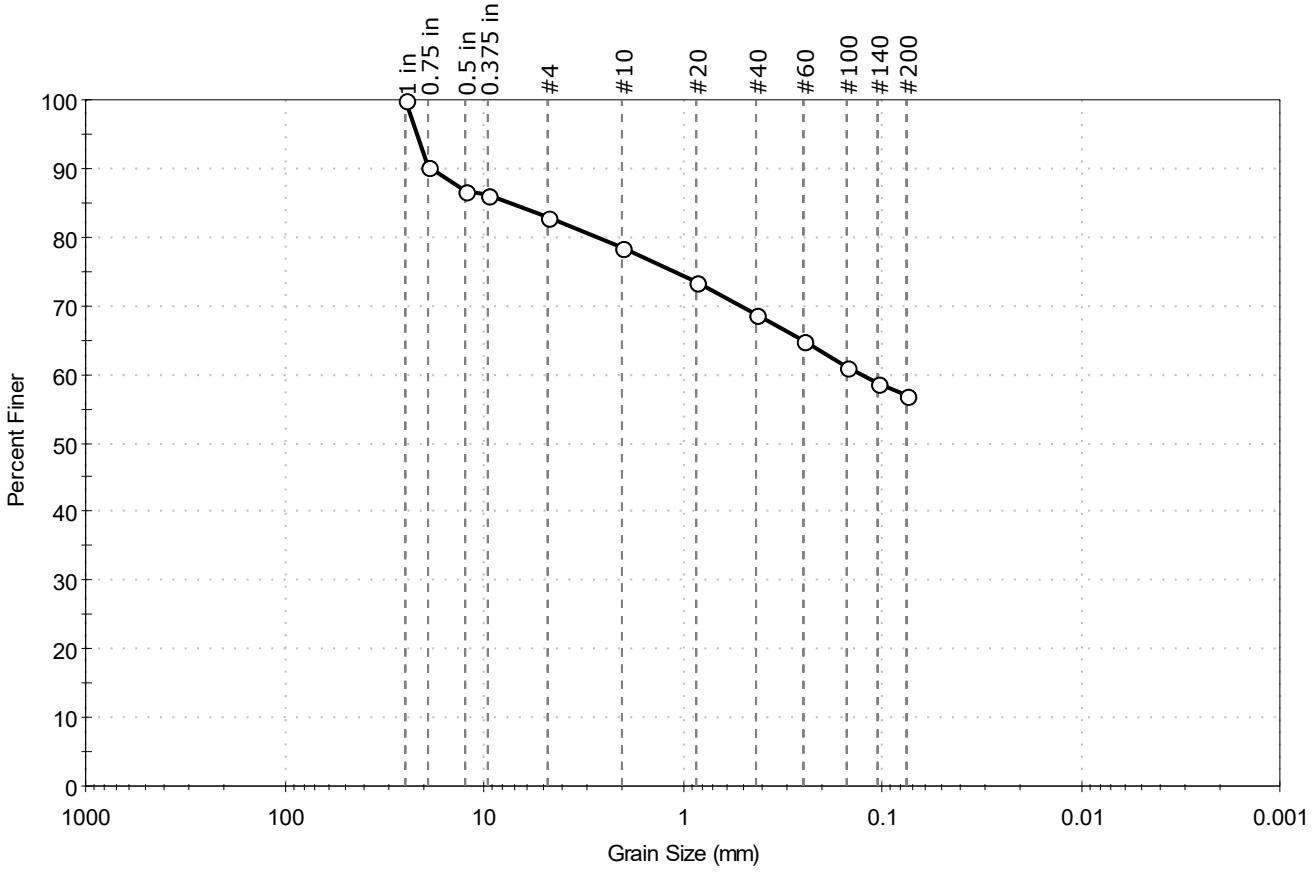
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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:	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))

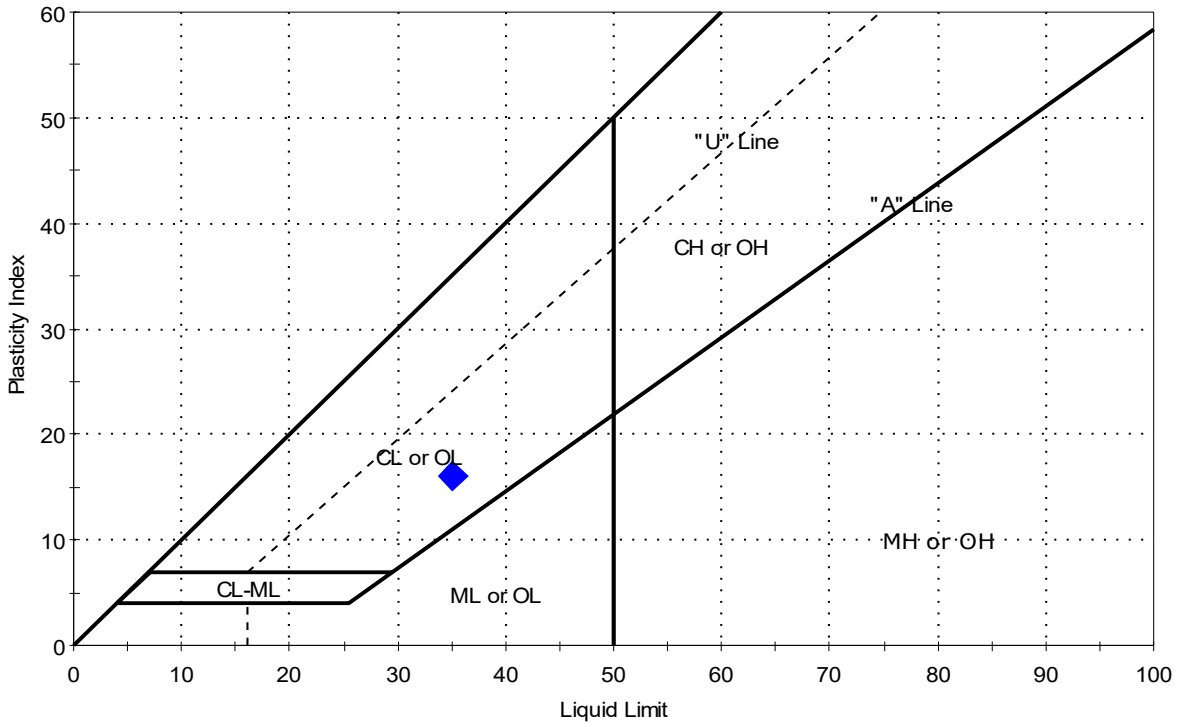
Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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	B-BWS-30	29-31 ft	32	35	19	16	0.8	

Sample Prepared using the WET method

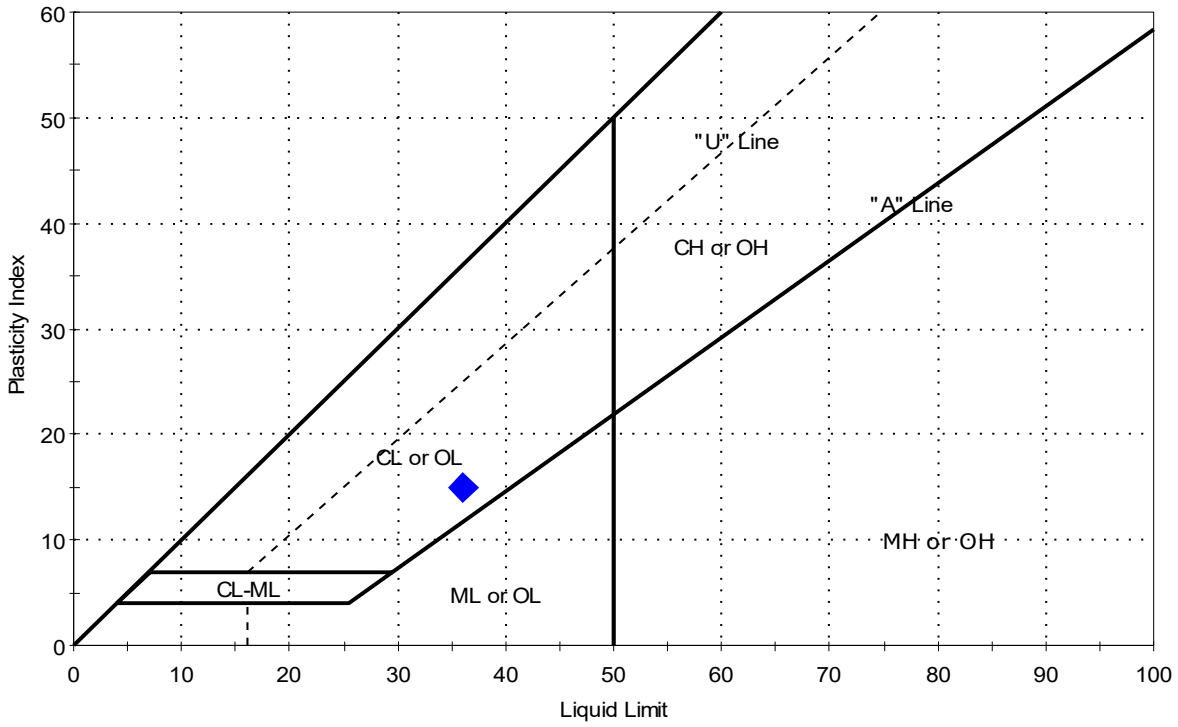
Dry Strength: VERY HIGH

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	B-BWS-30	19-21 ft	24	36	21	15	0.2	

Sample Prepared using the WET method

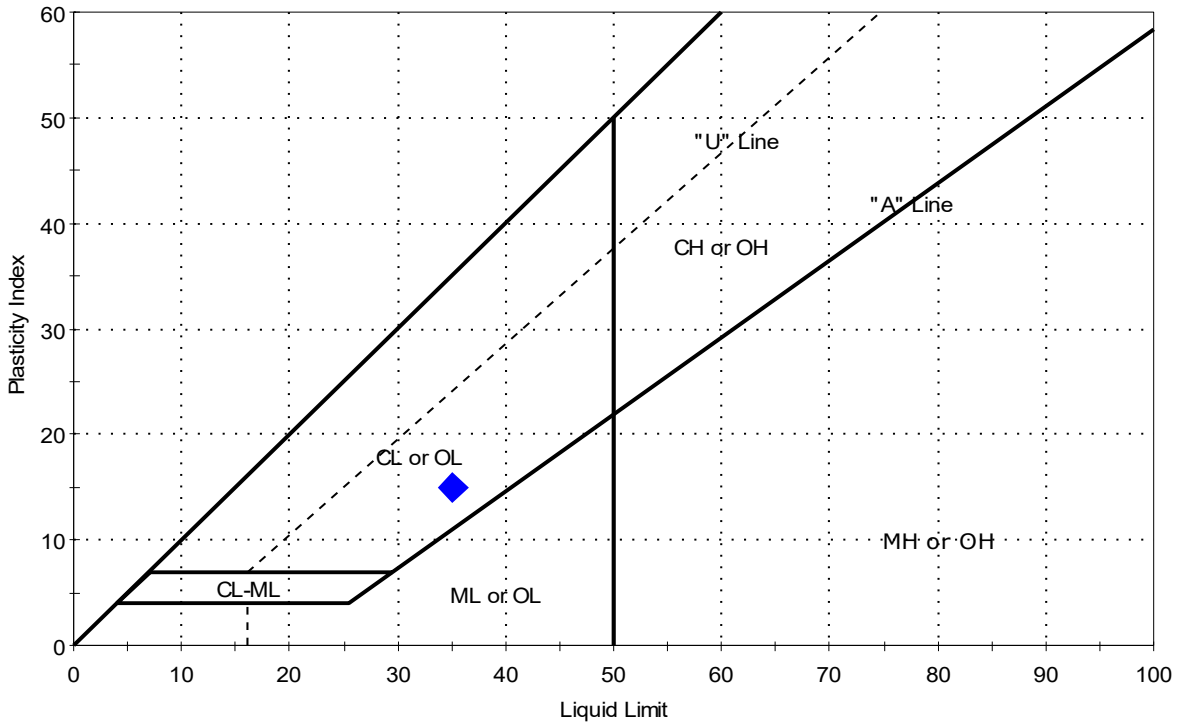
Dry Strength: VERY HIGH

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	B-BWS-30	24-26 ft	29	35	20	15	0.6	

Sample Prepared using the WET method

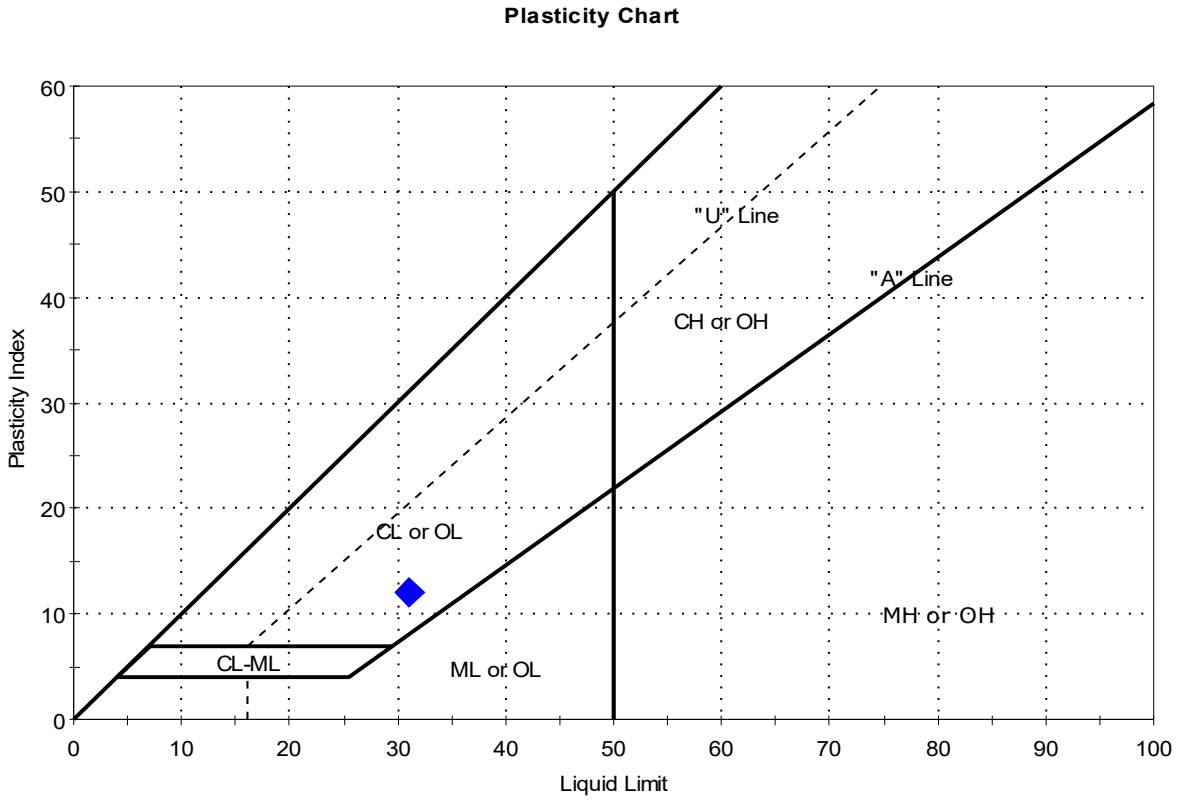
Dry Strength: VERY HIGH

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:	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	B-BWS-30	34-36 ft	33	31	19	12	1.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

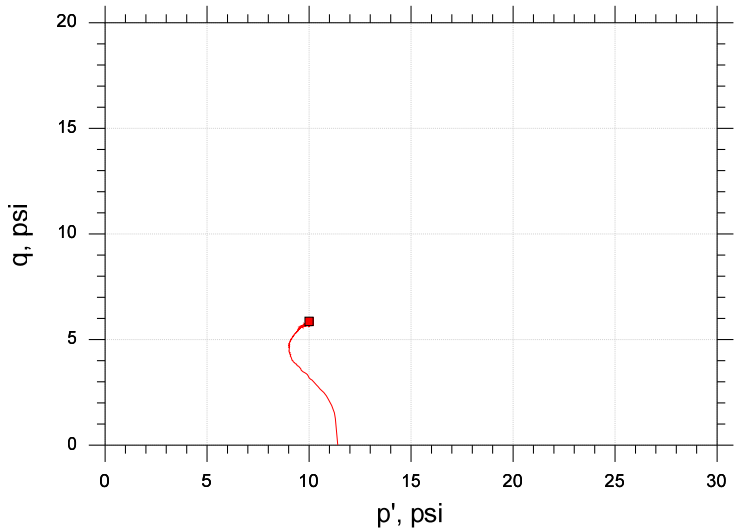
Dilatancy: SLOW

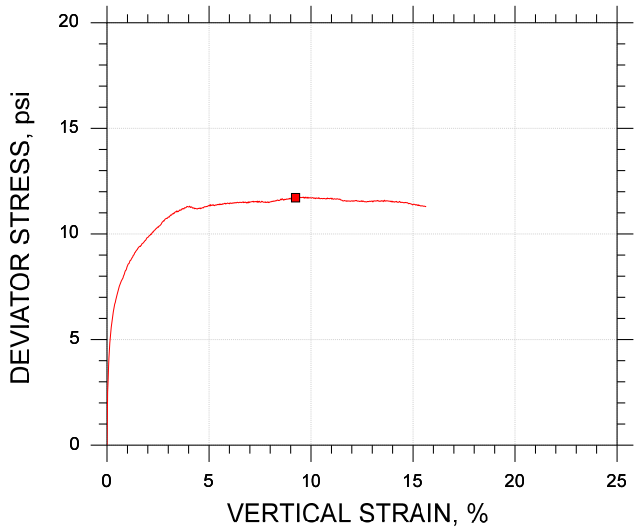
Toughness: LOW

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

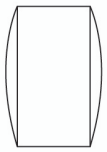
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





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			

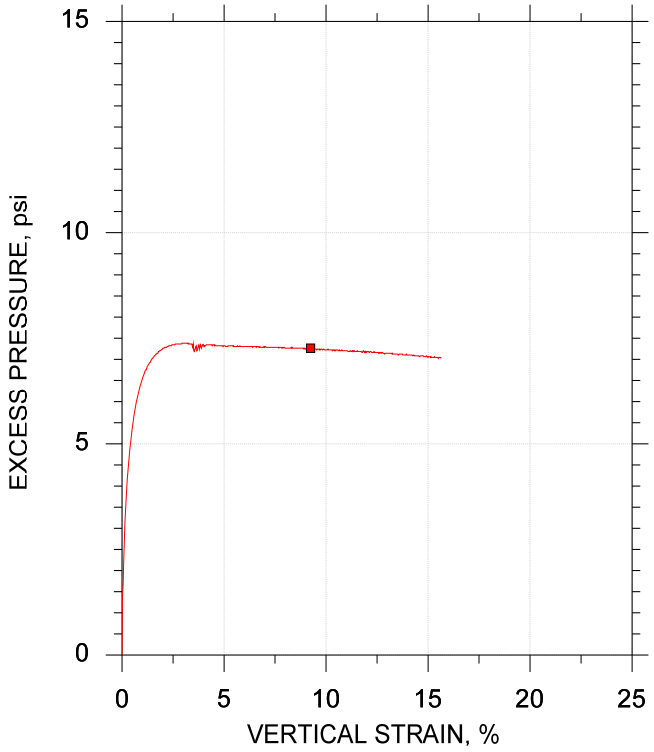
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should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

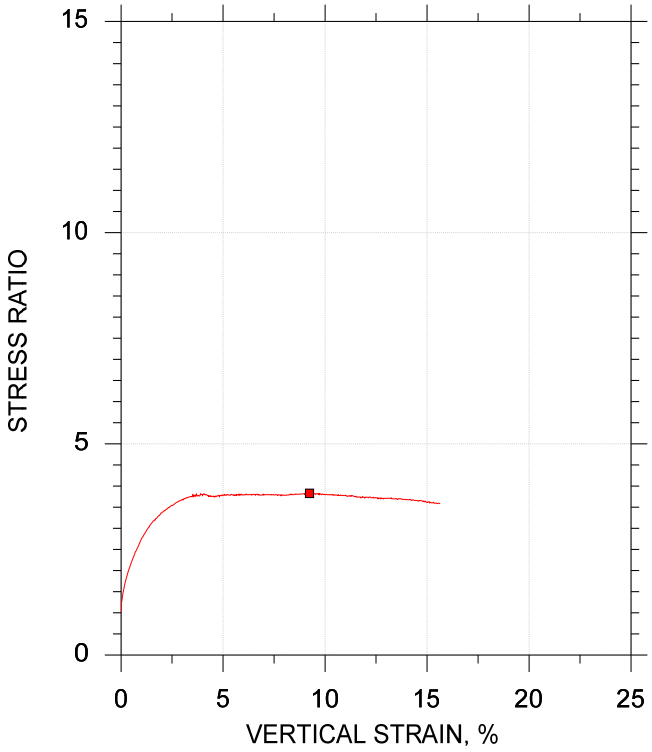


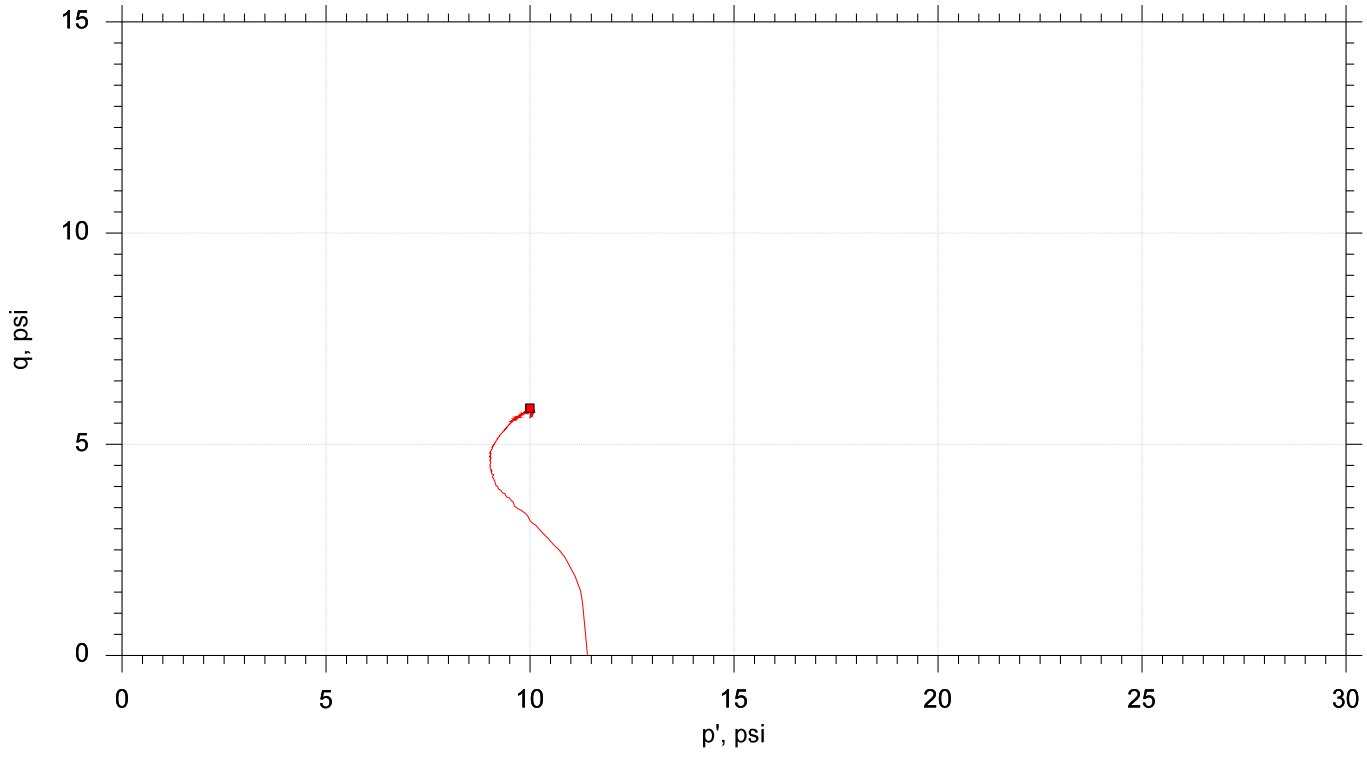
Remarks:

System RR

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767







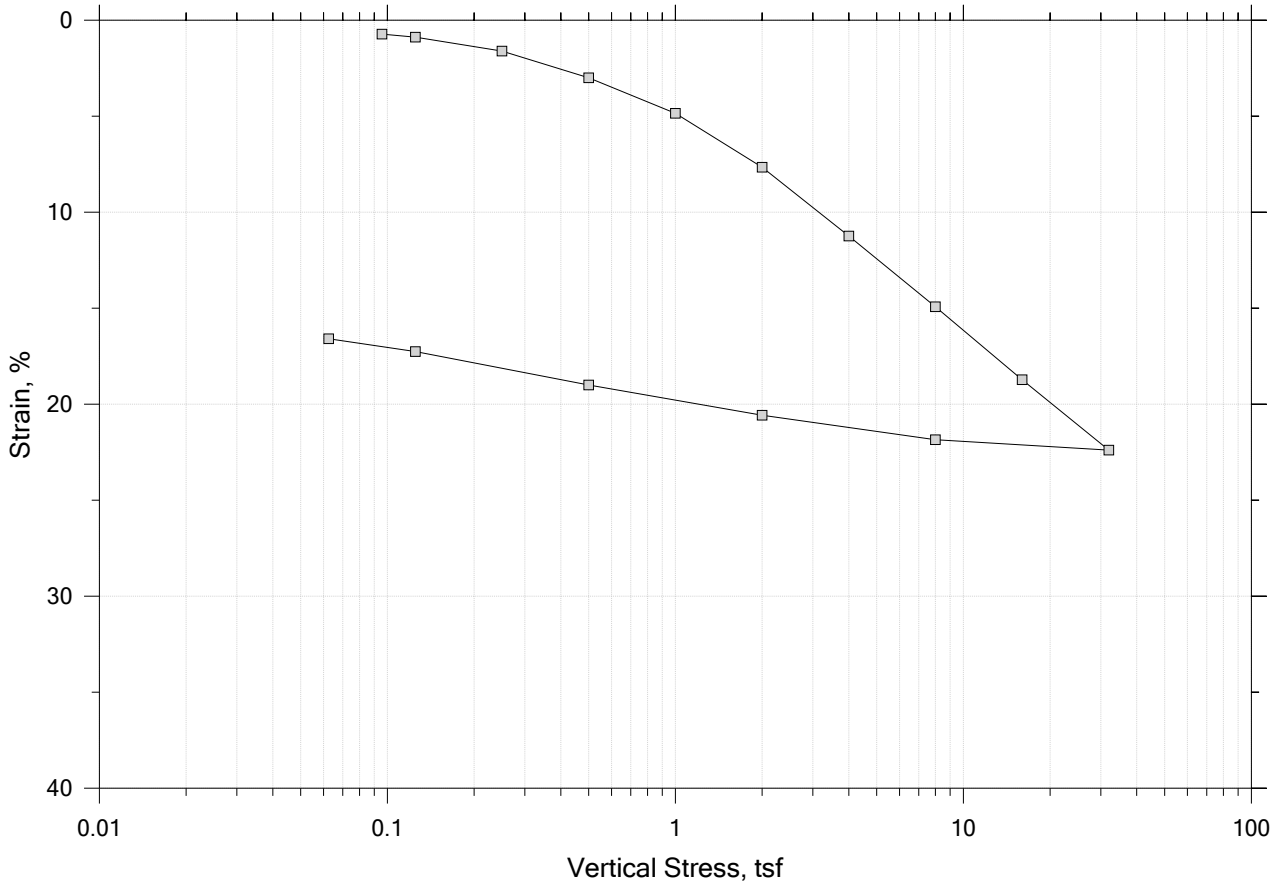
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	3U	CU-1-1	29-31	md	02/18/20	njh	---	311345-CU-1-1n.dat

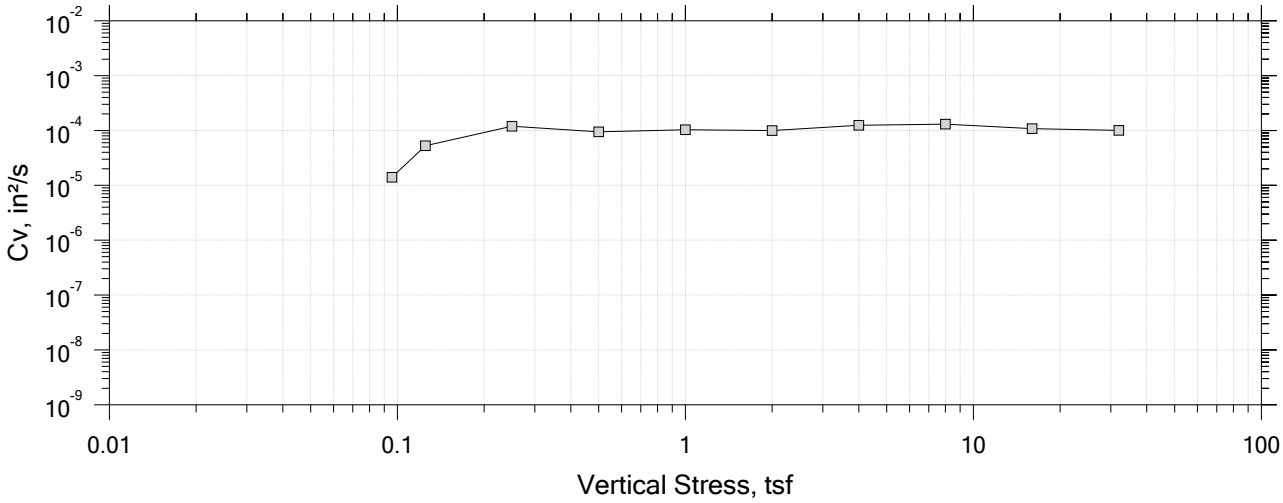
GeoTesting EXPRESS

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





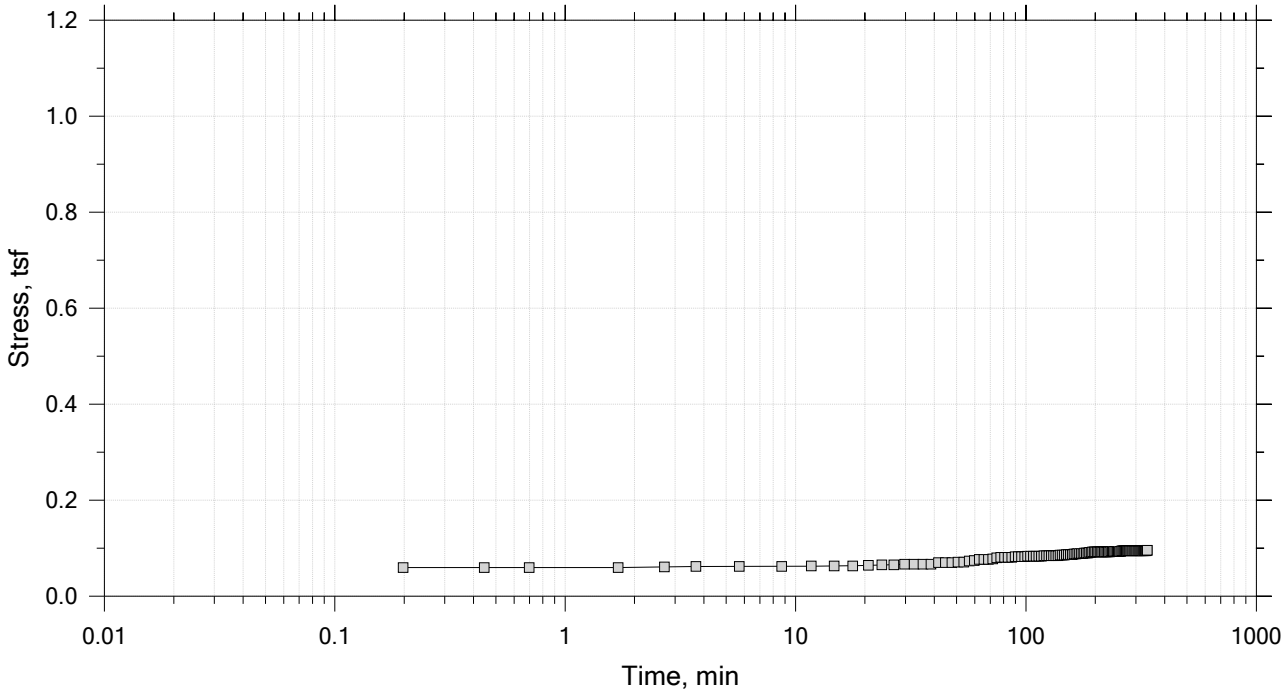
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

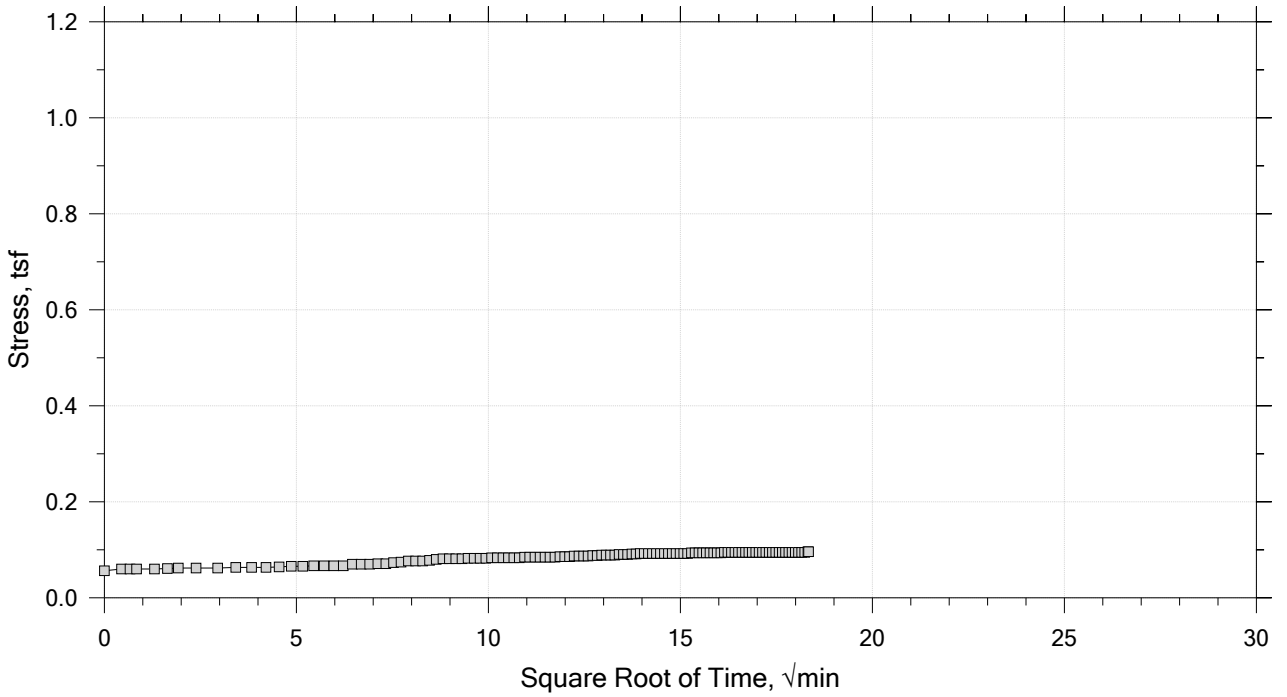
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 15

Constant Volume Step

Stress: 0.0958 tsf

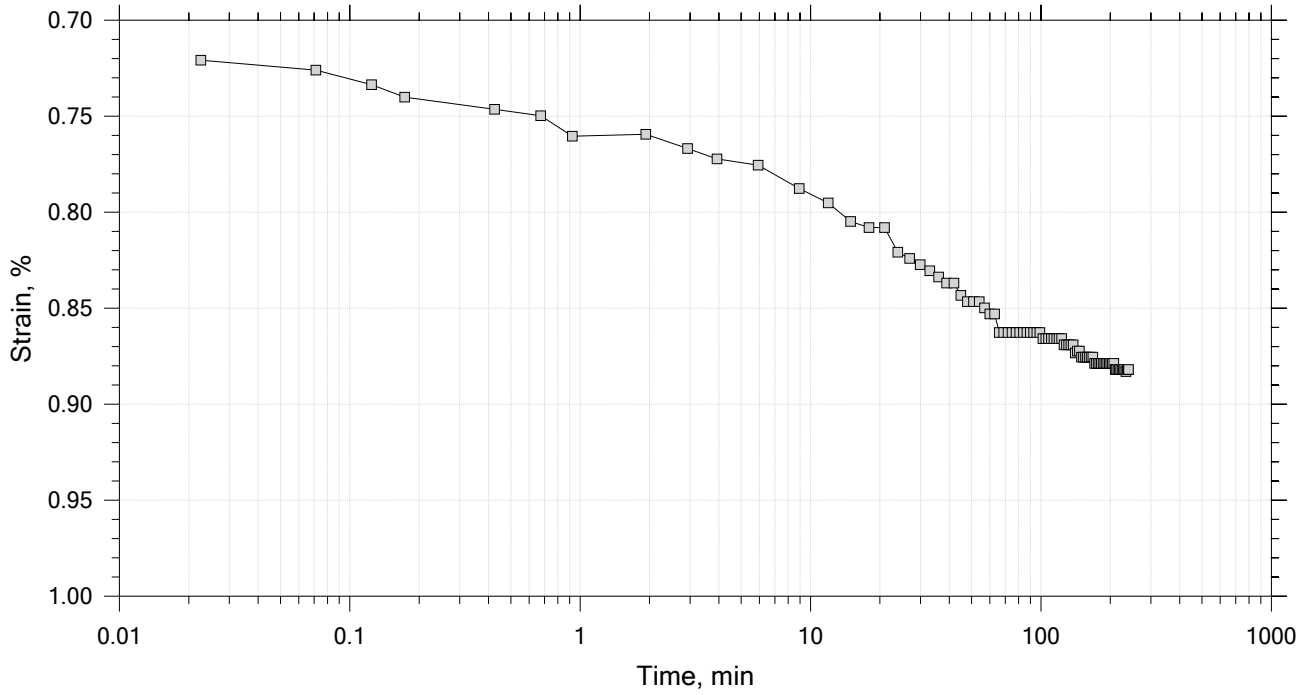


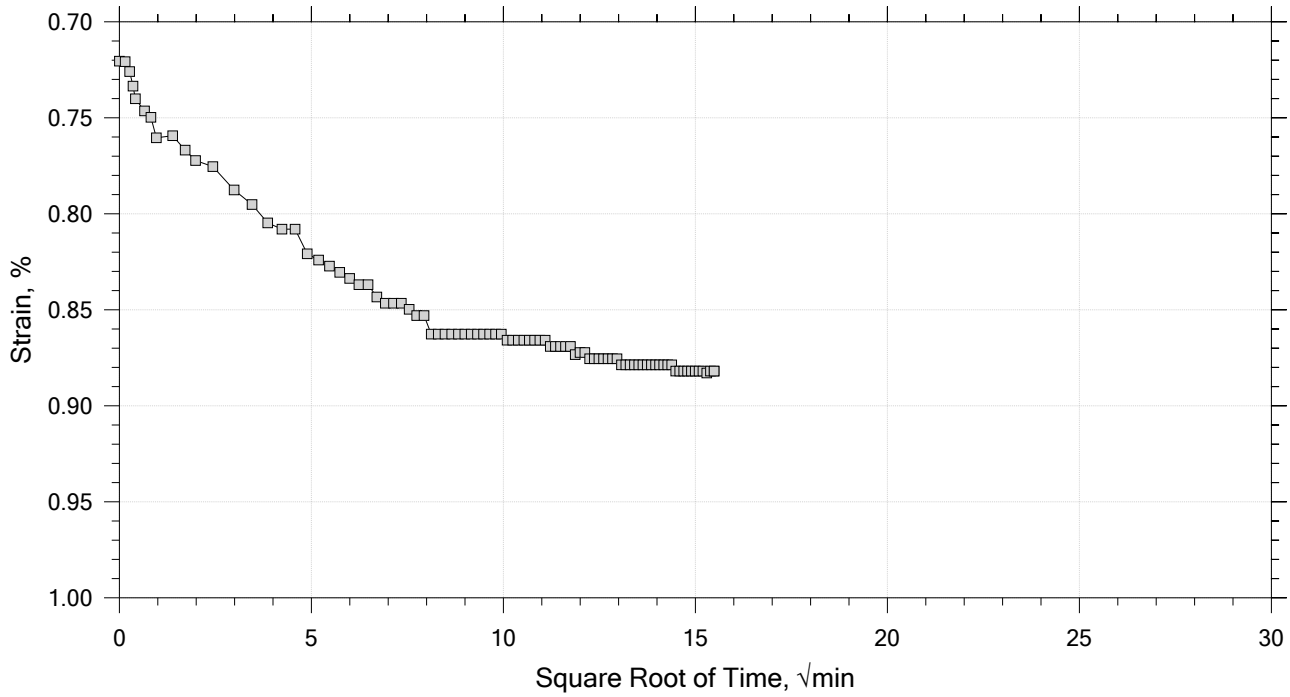


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

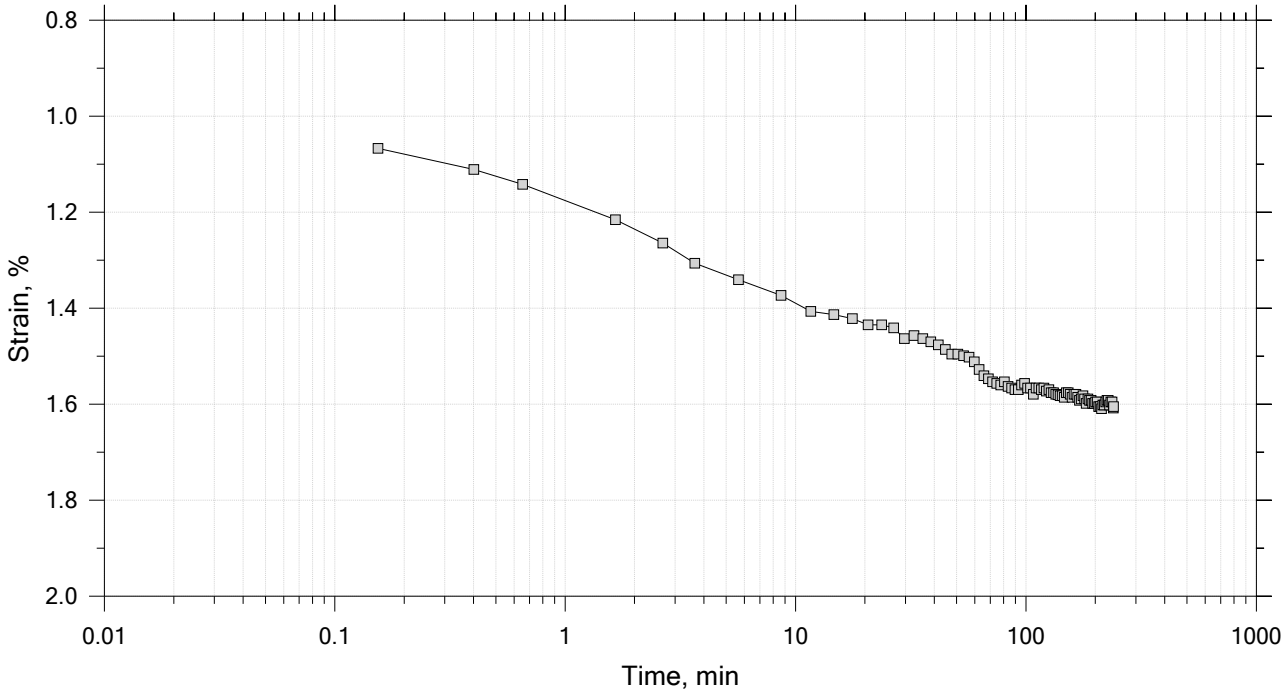


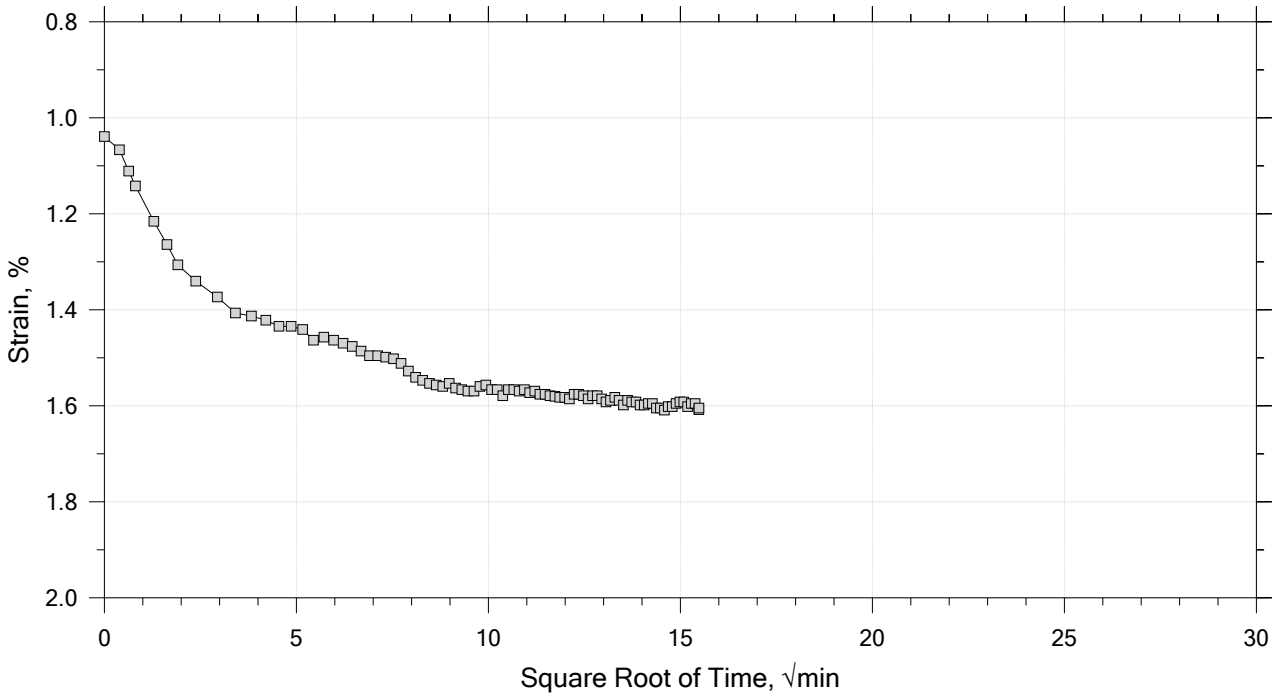


Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
Test No.: IP-1	Sample Type: tube	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B,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





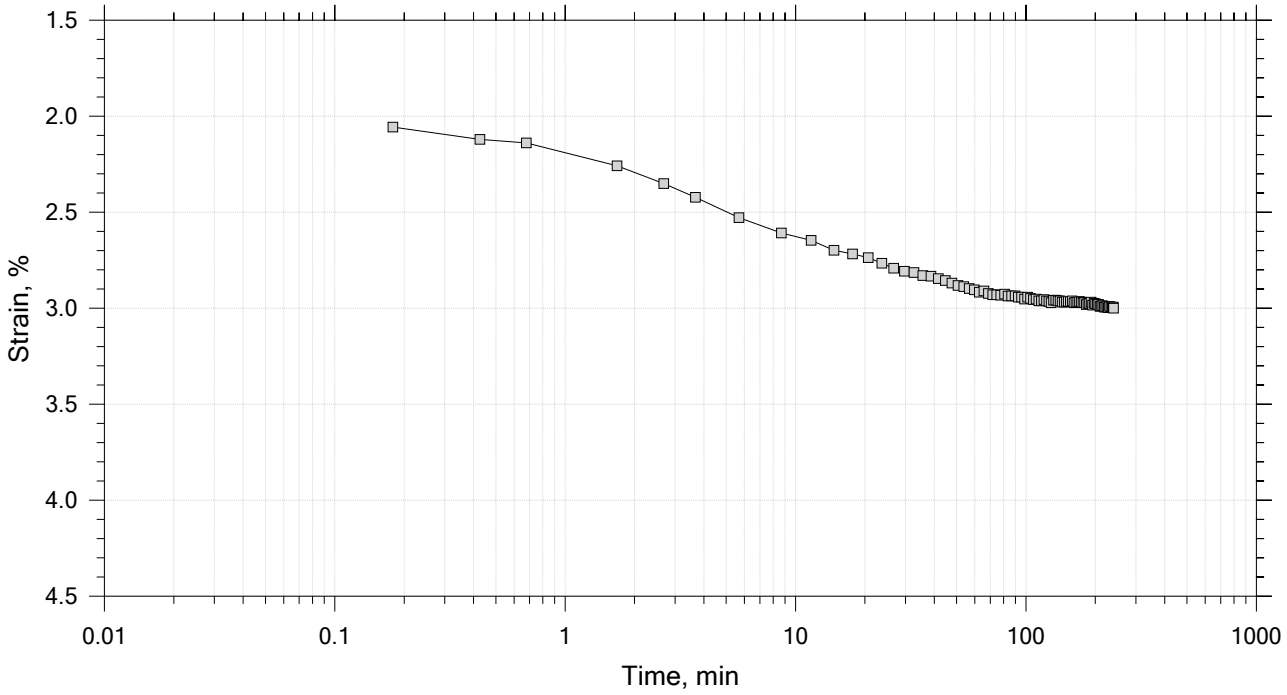
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

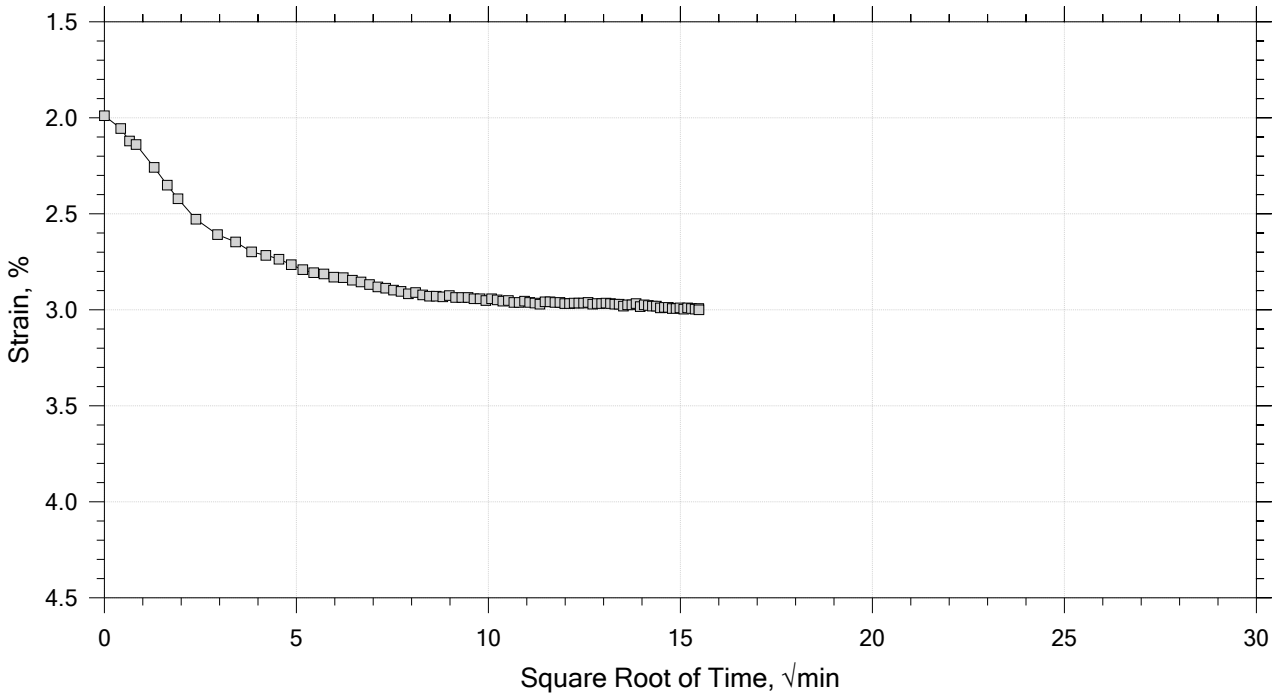
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 15

Constant Load Step

Stress: 0.5 tsf





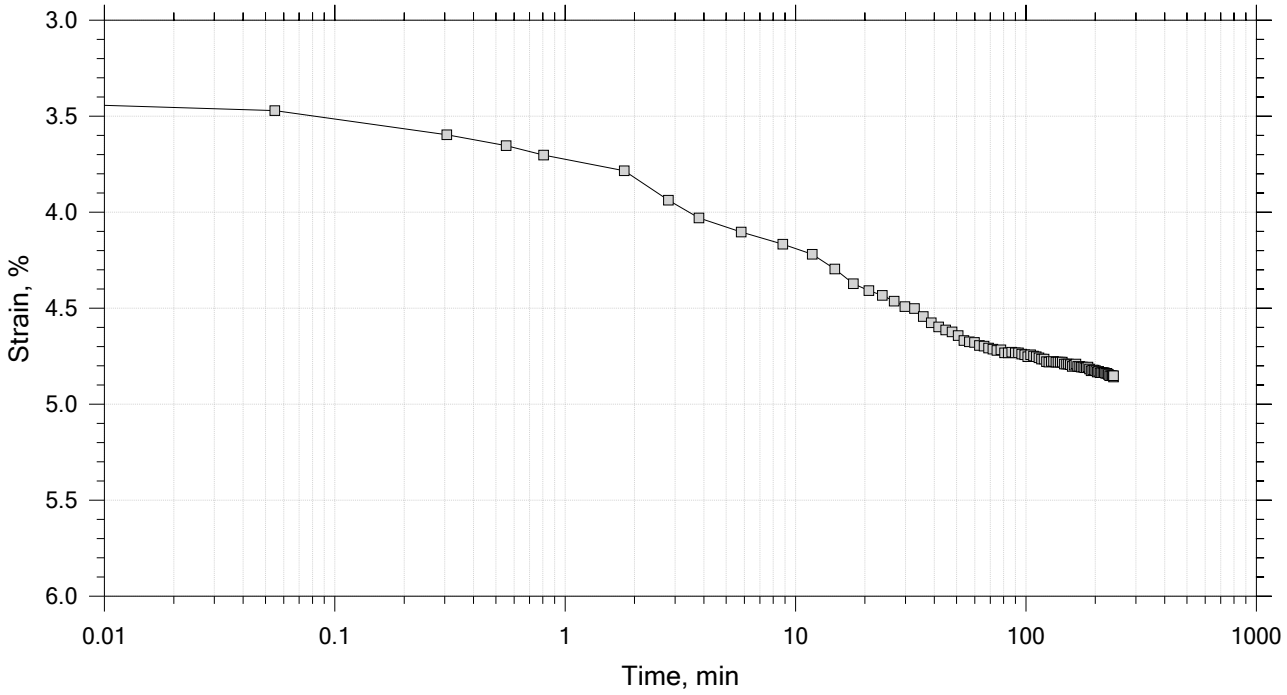
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

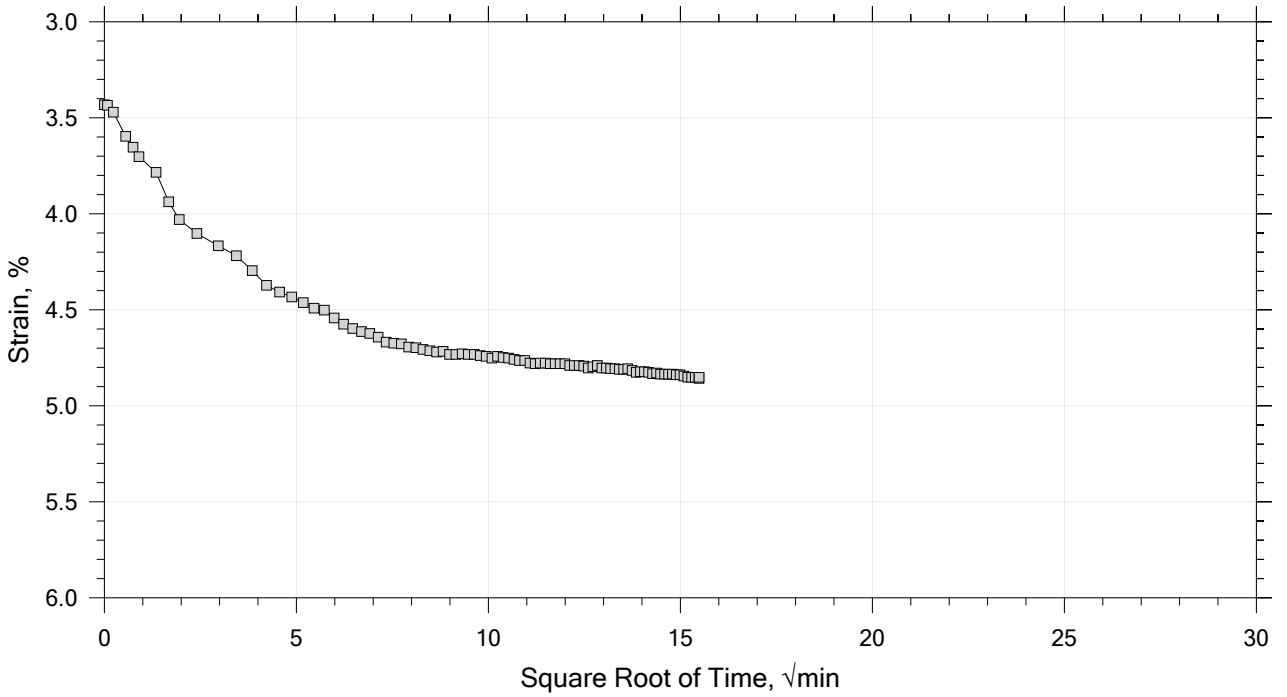
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 15

Constant Load Step

Stress: 1 tsf





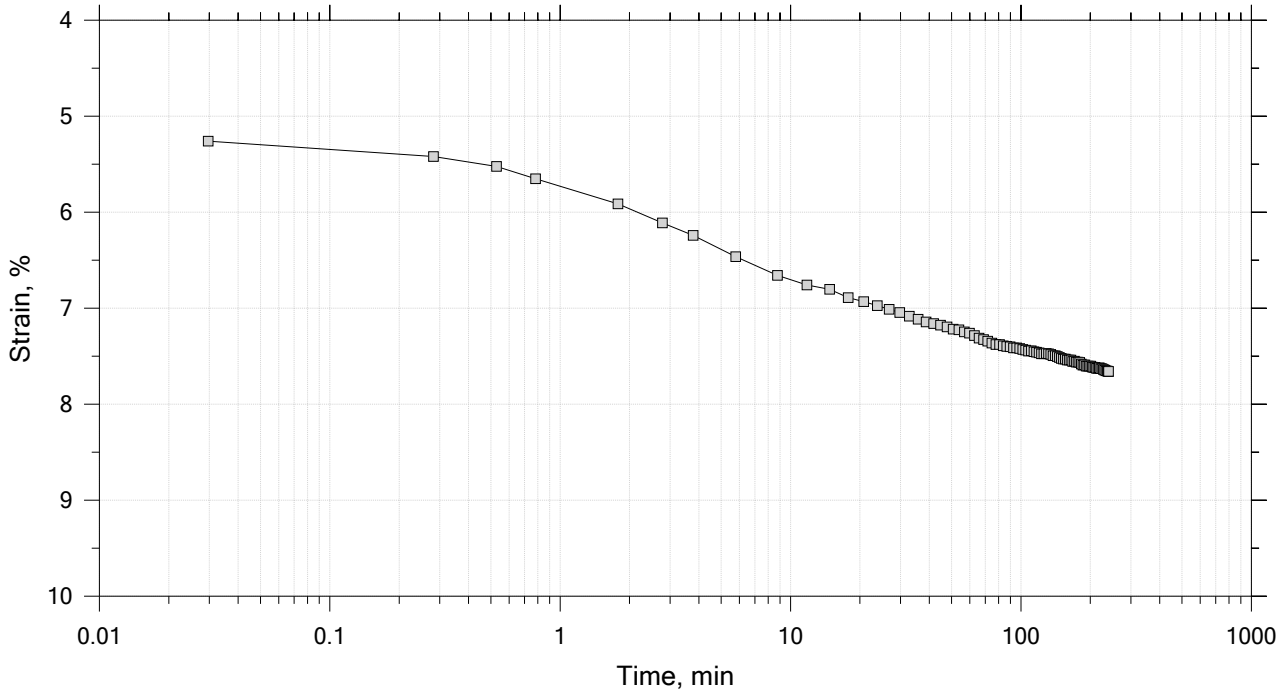
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

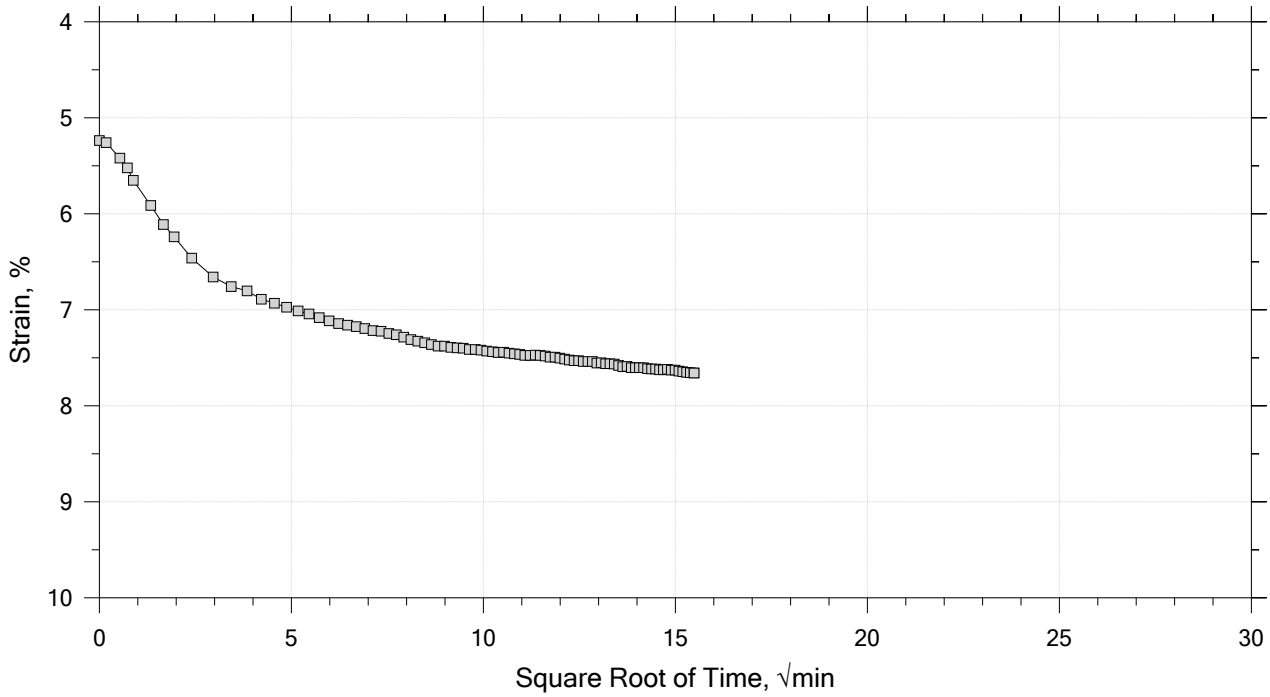
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 15

Constant Load Step

Stress: 2 tsf





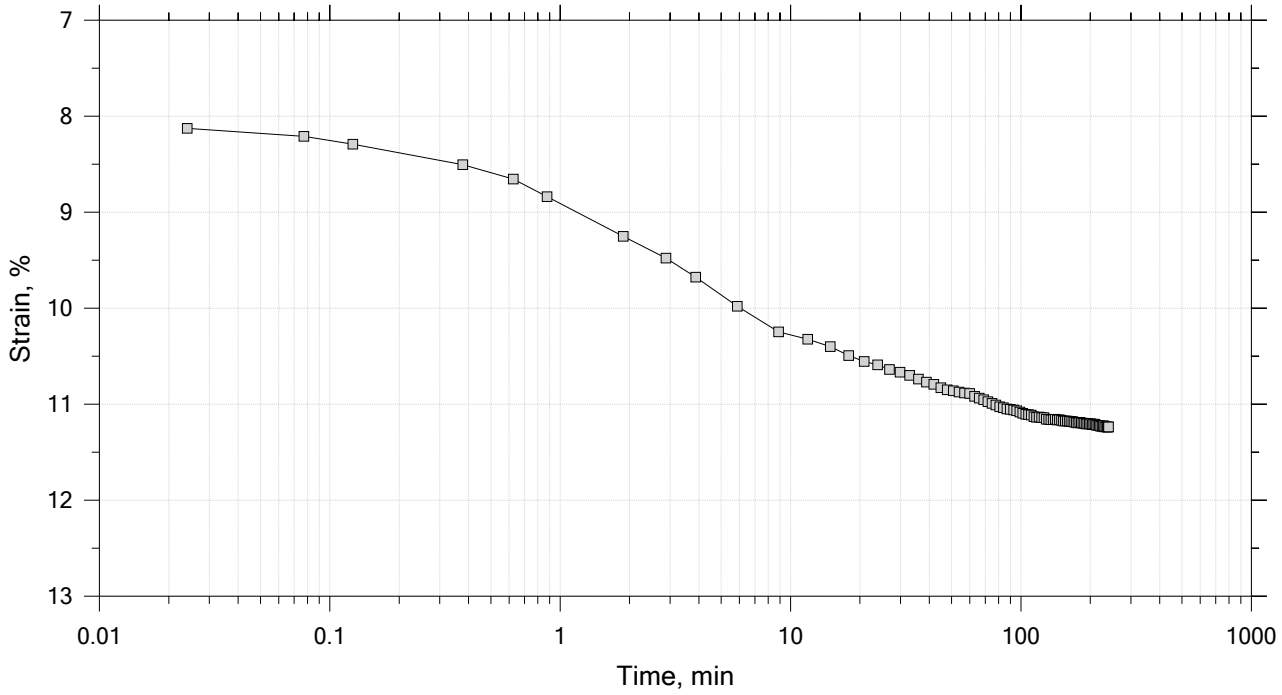
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

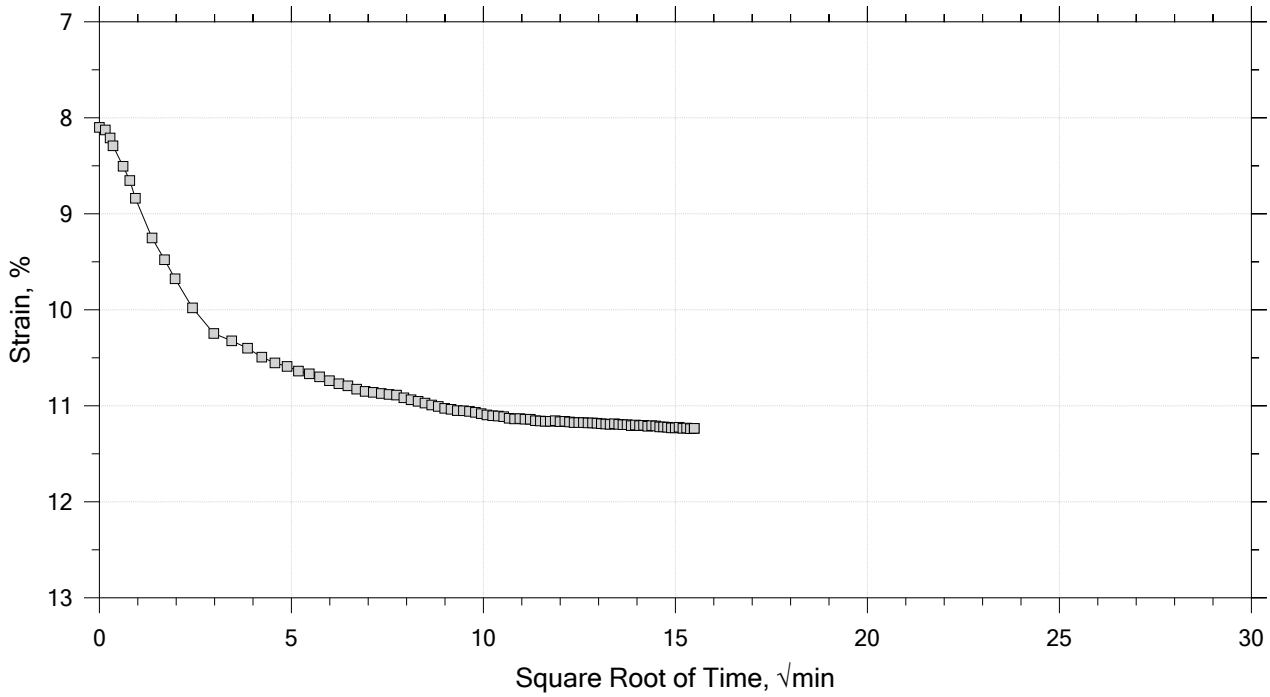
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 15

Constant Load Step

Stress: 4 tsf





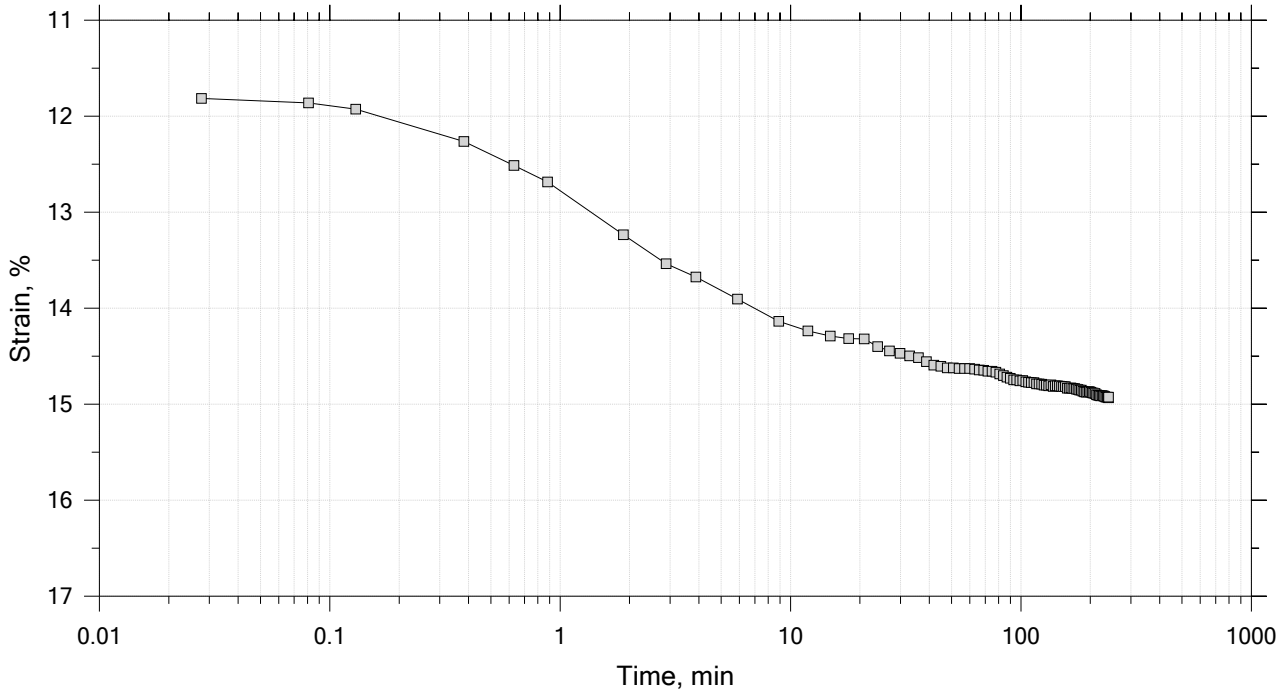
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

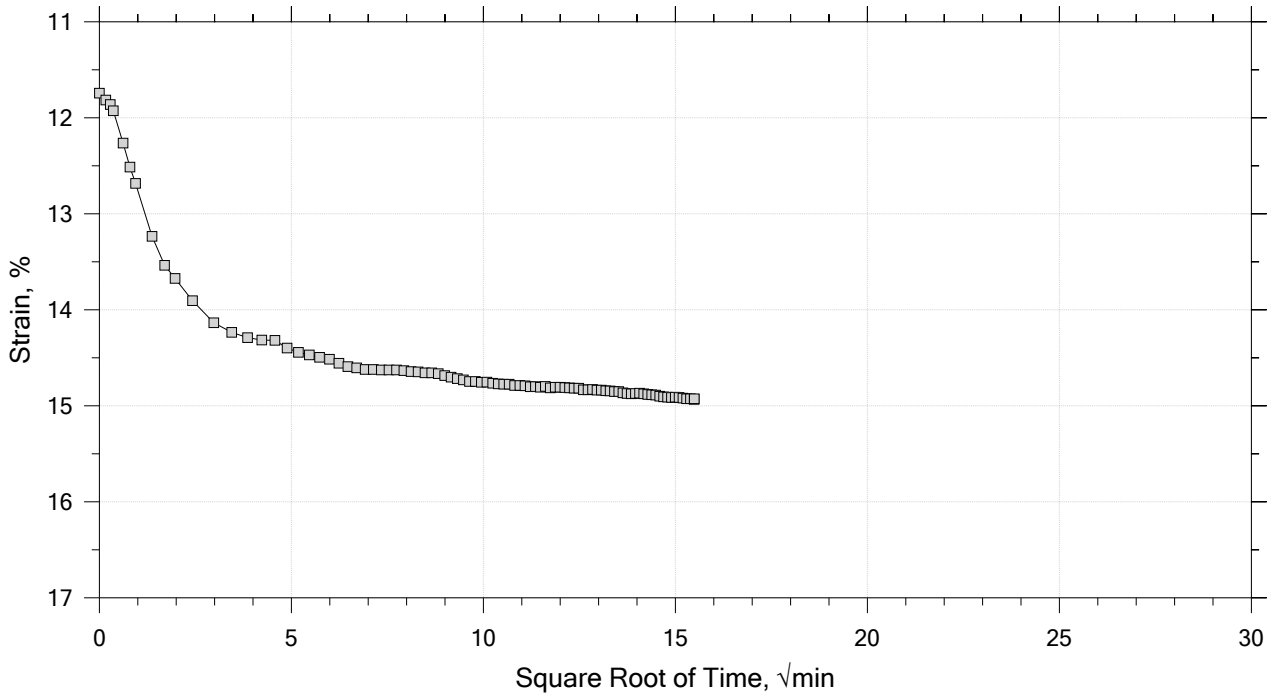
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 15

Constant Load Step

Stress: 8 tsf

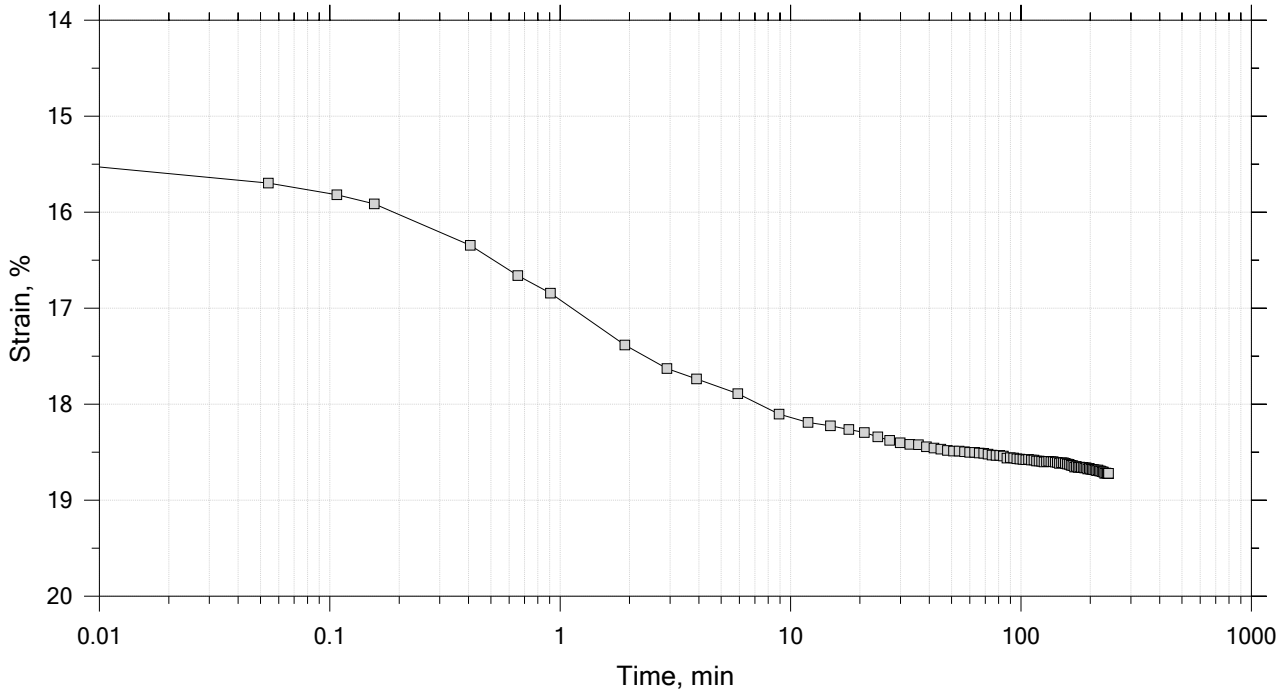


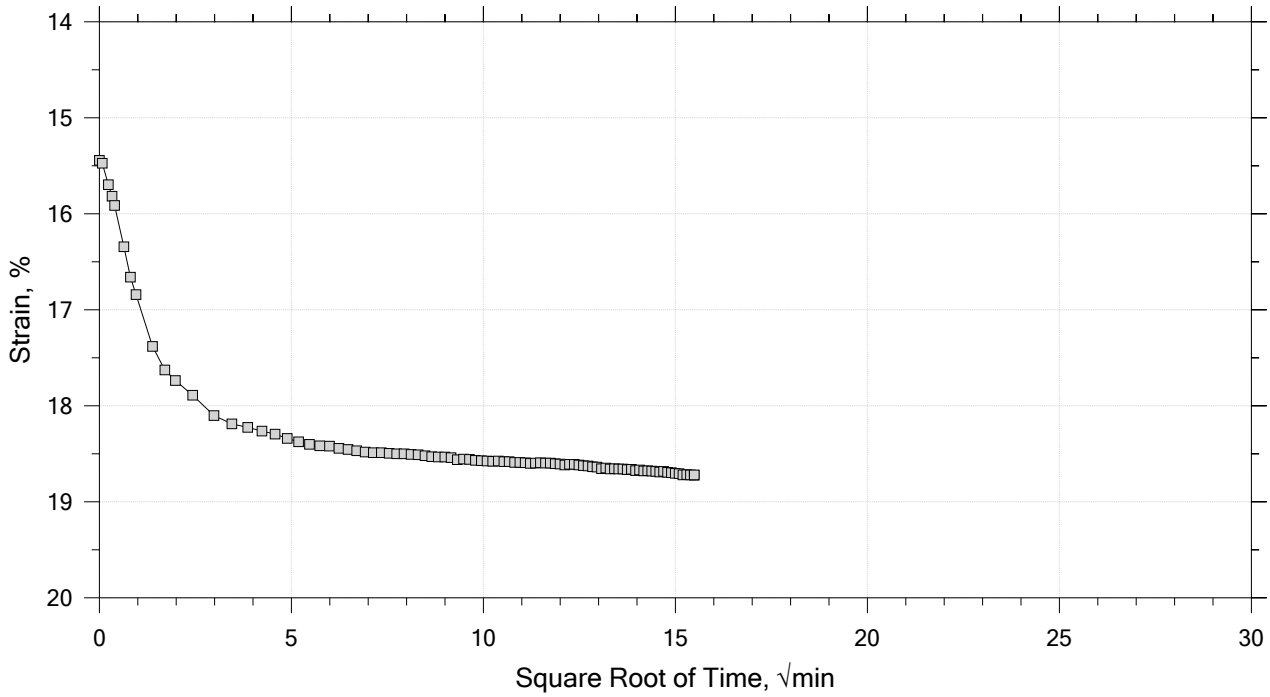


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

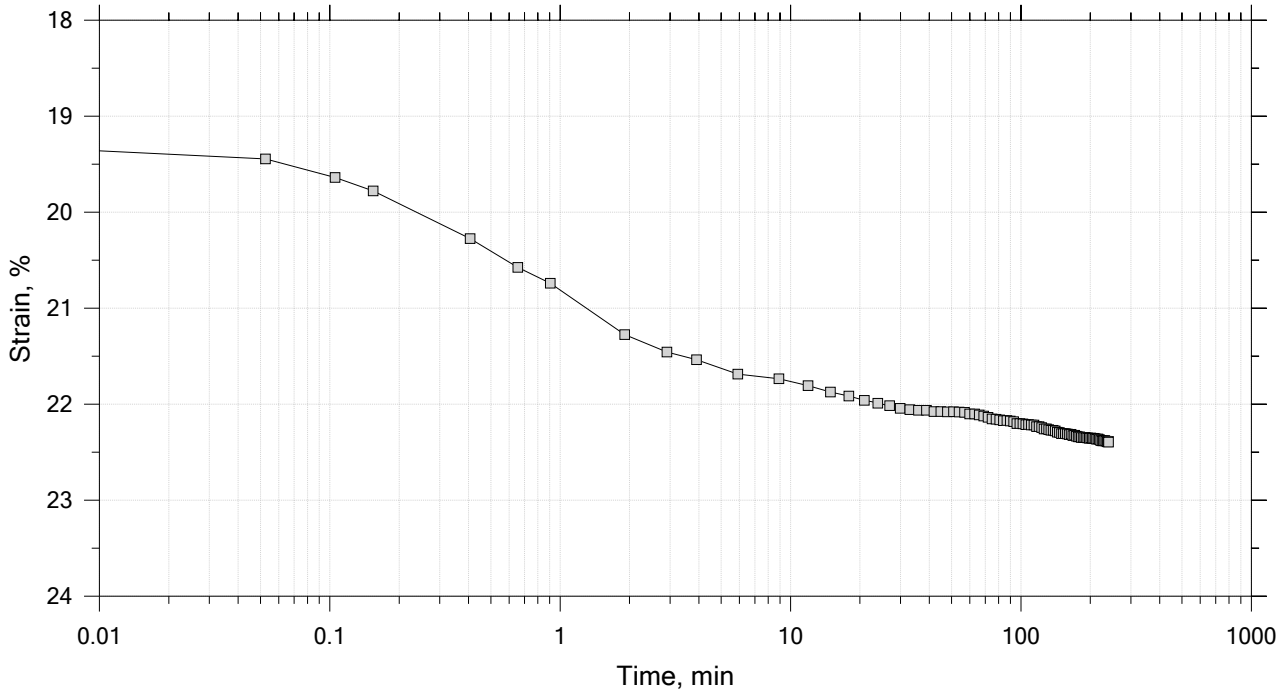


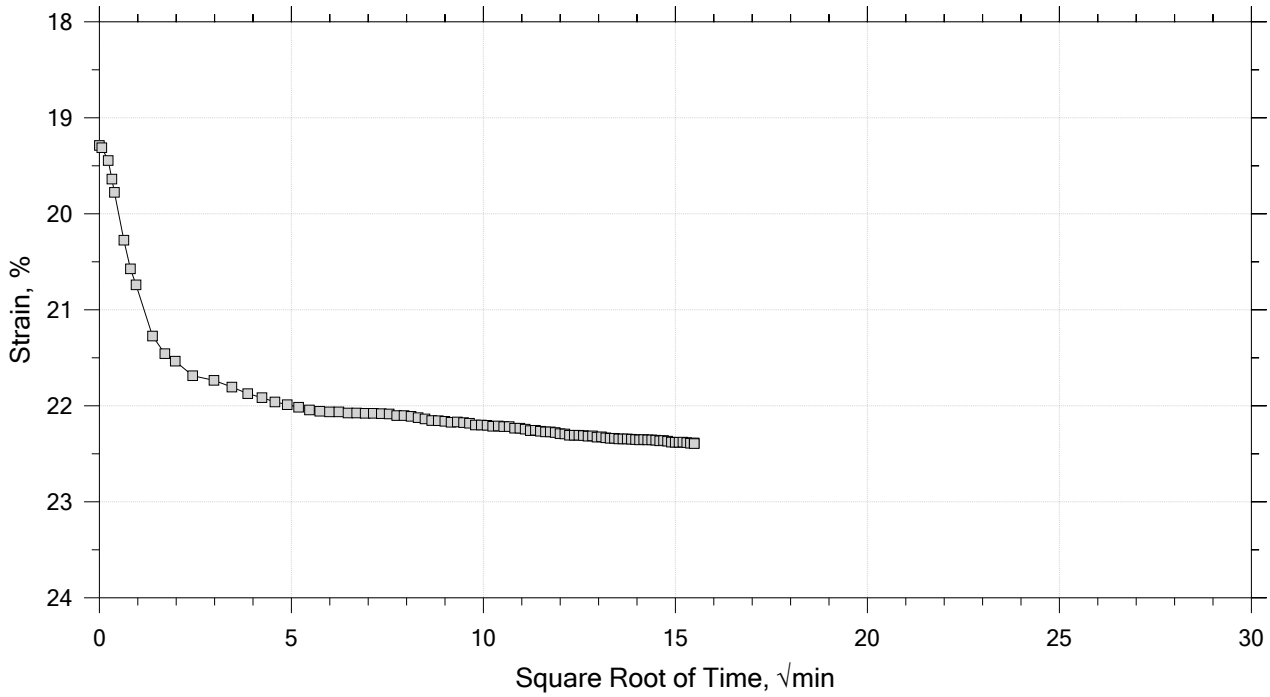


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

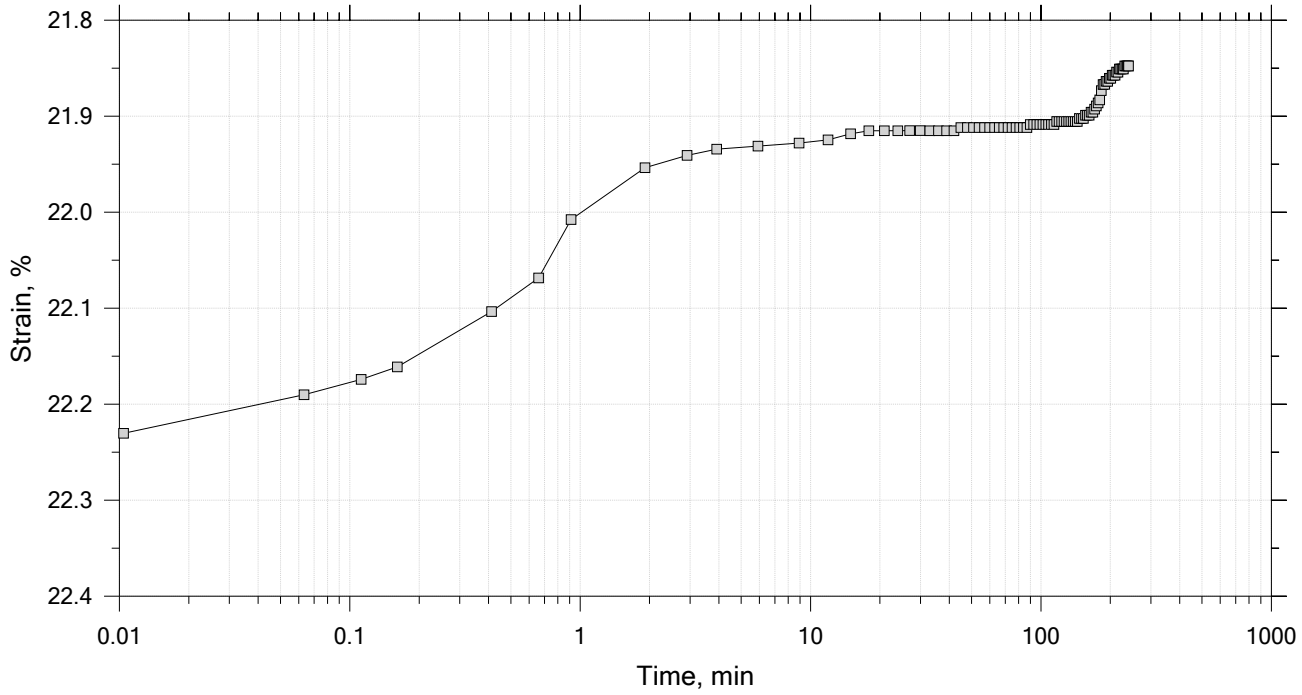


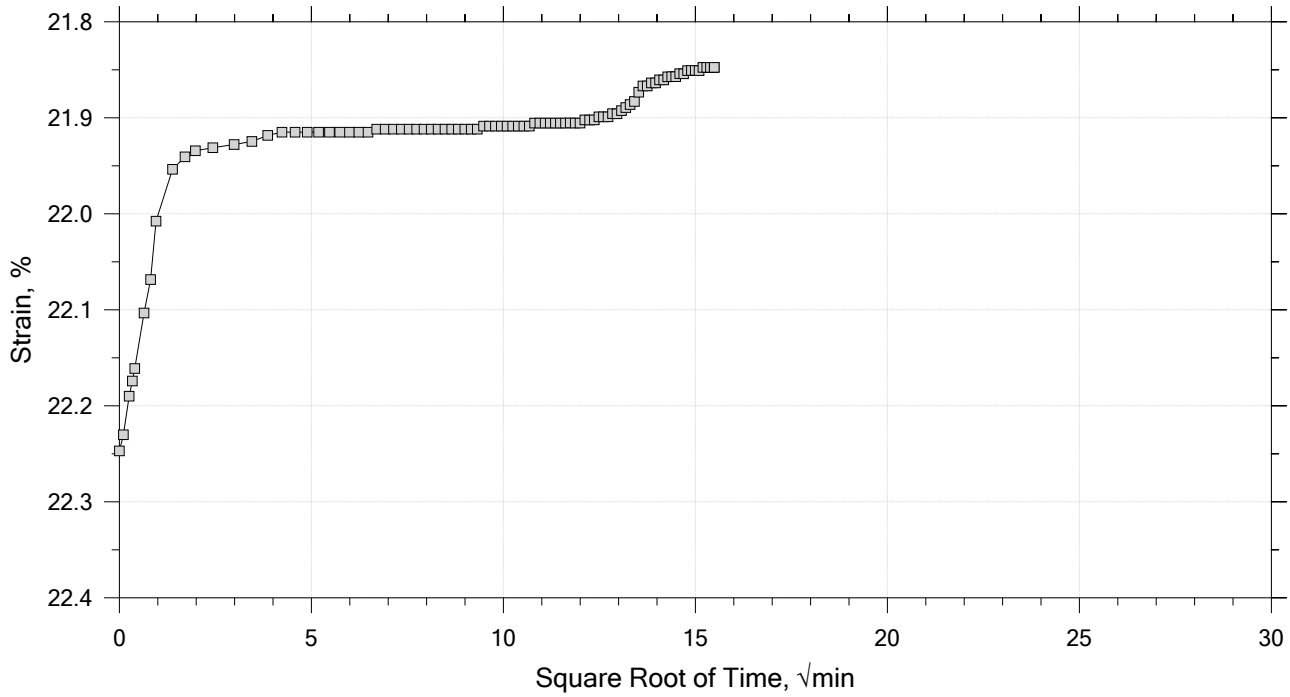


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

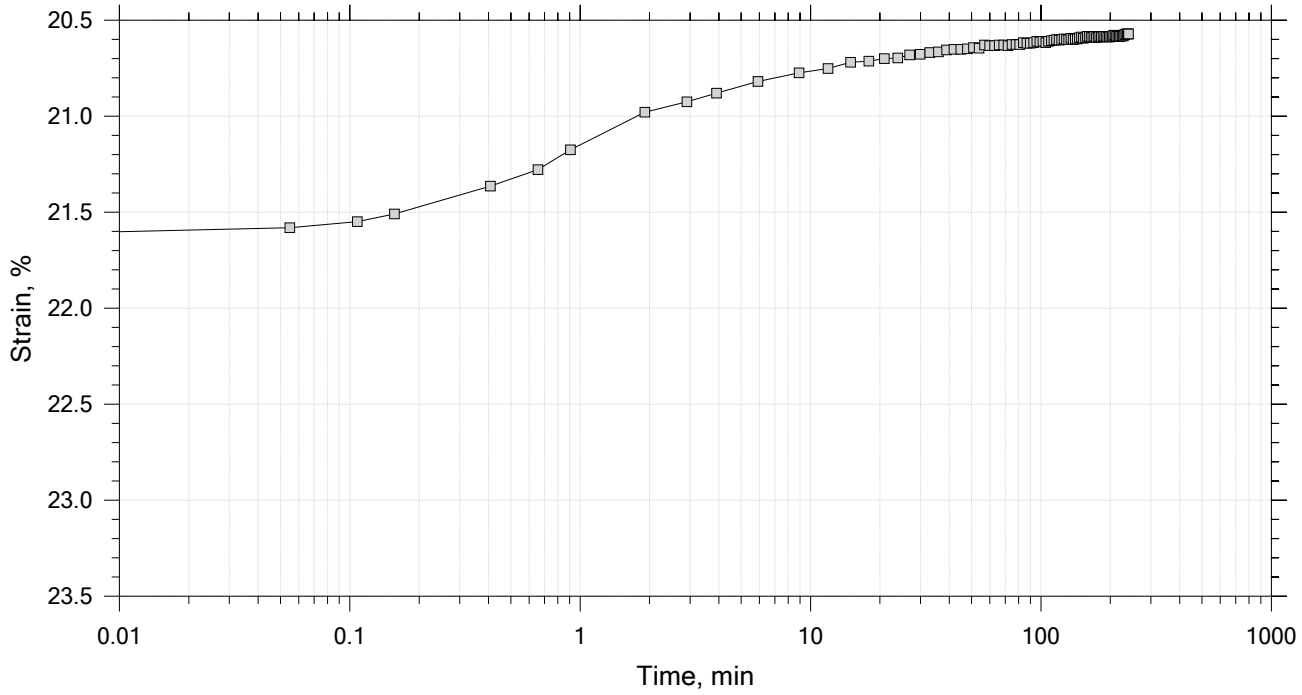


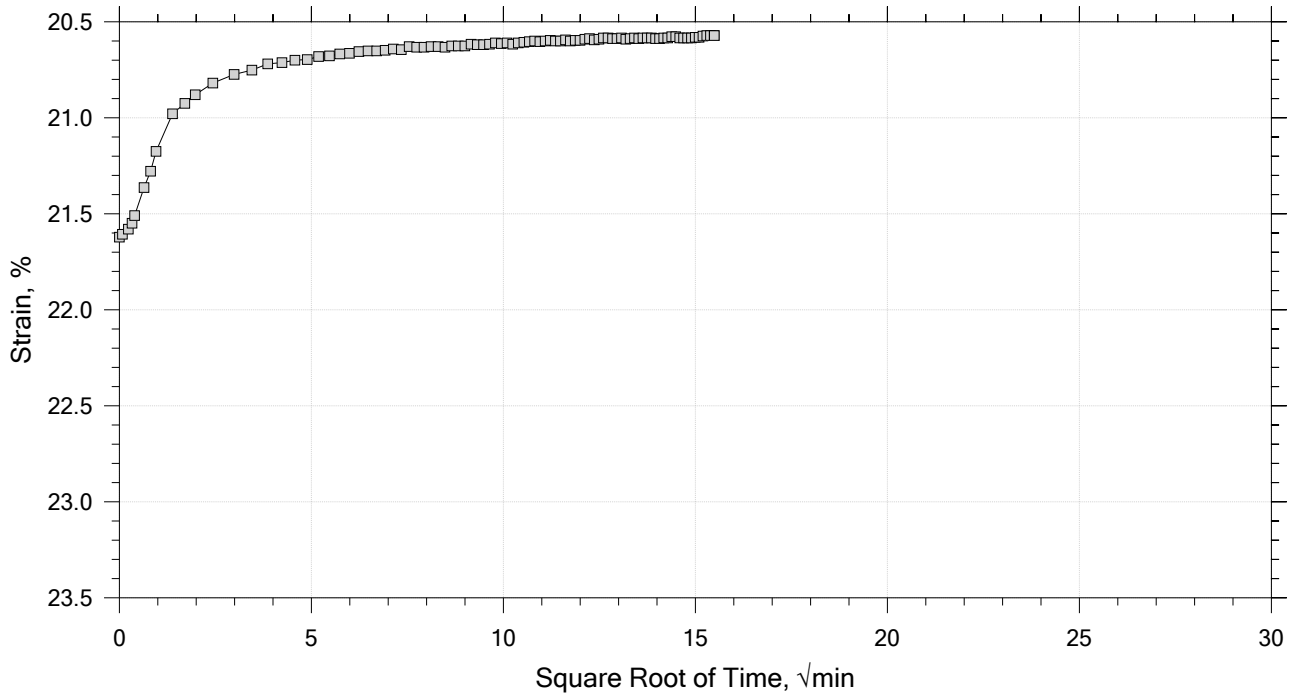


Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
Test No.: IP-1	Sample Type: tube	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B,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

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 15
Constant Load Step
Stress: 2 tsf





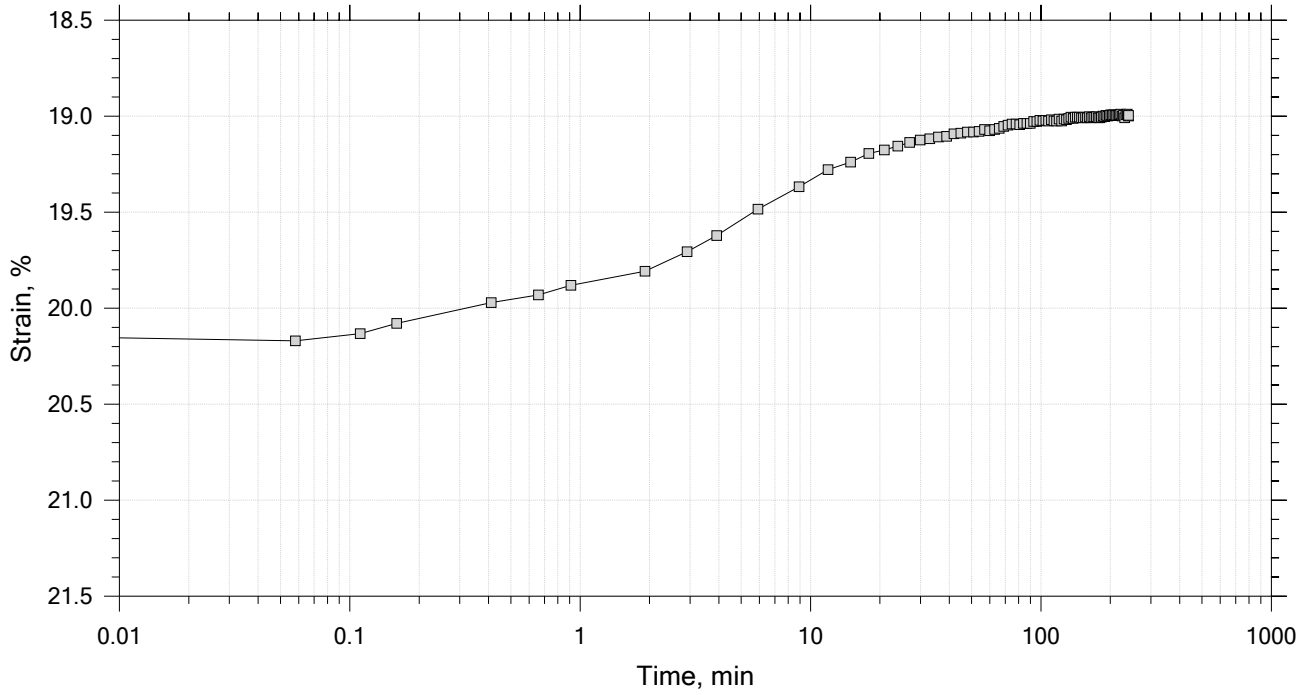
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

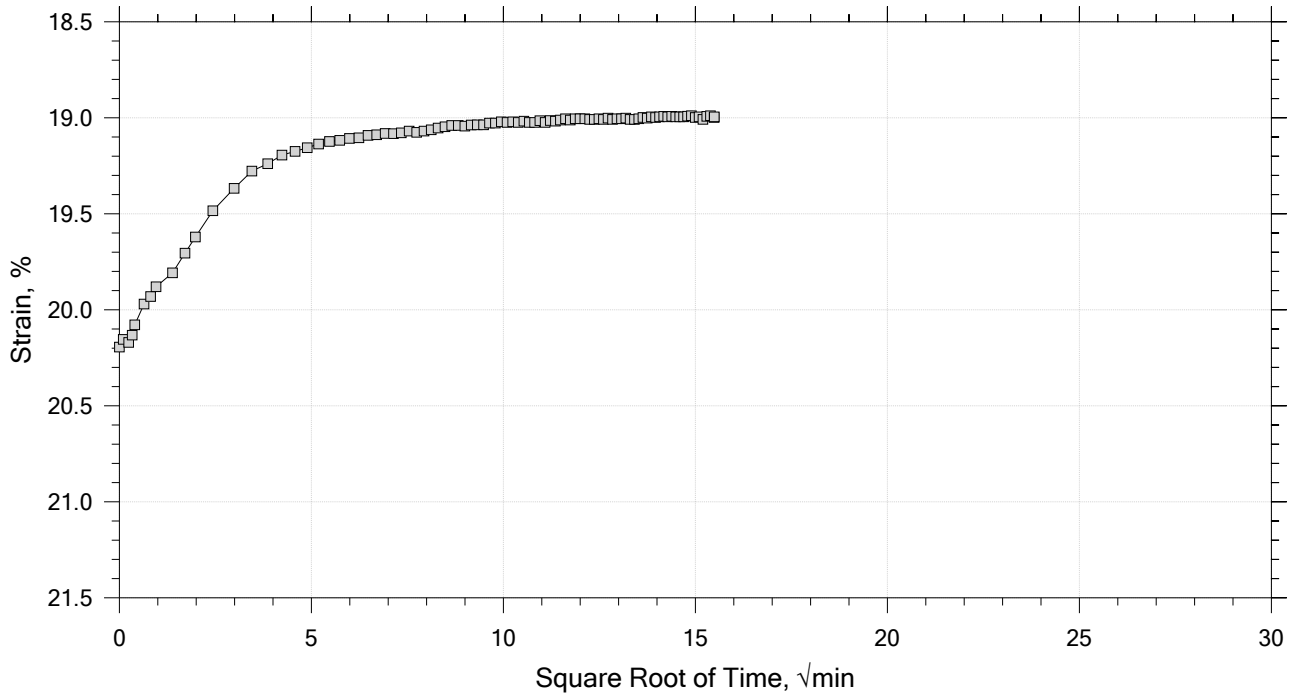
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 15

Constant Load Step

Stress: 0.5 tsf

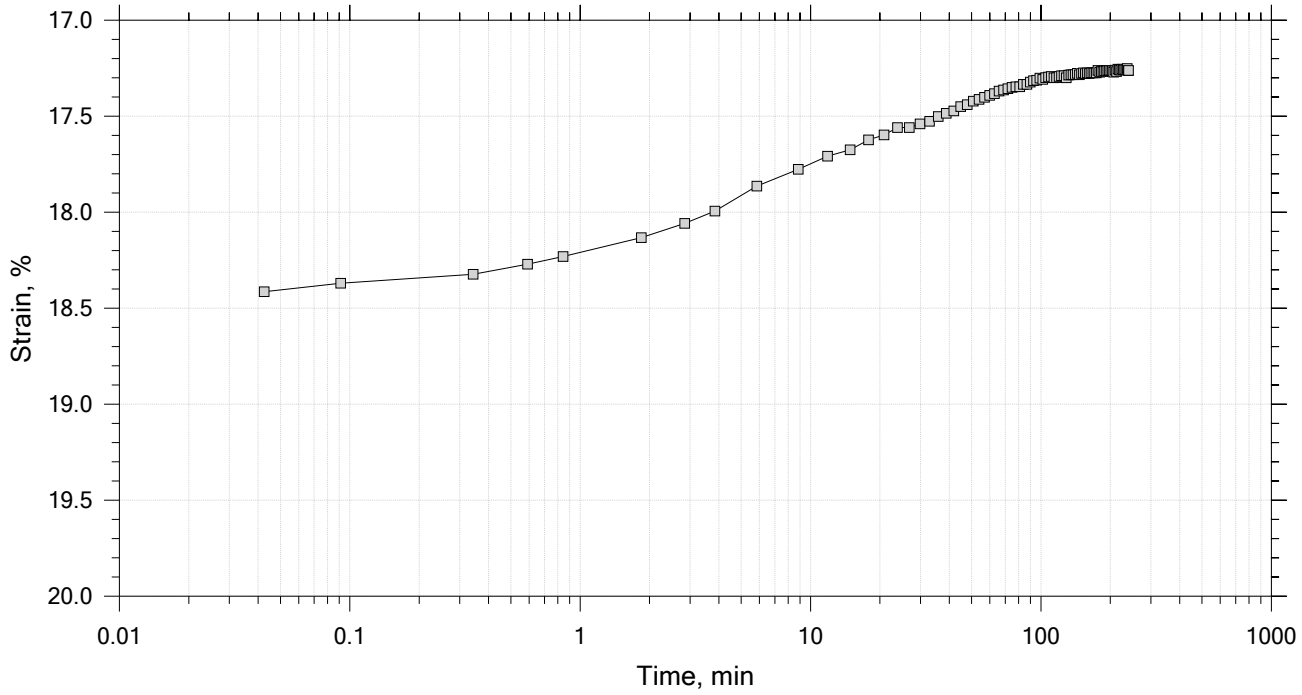


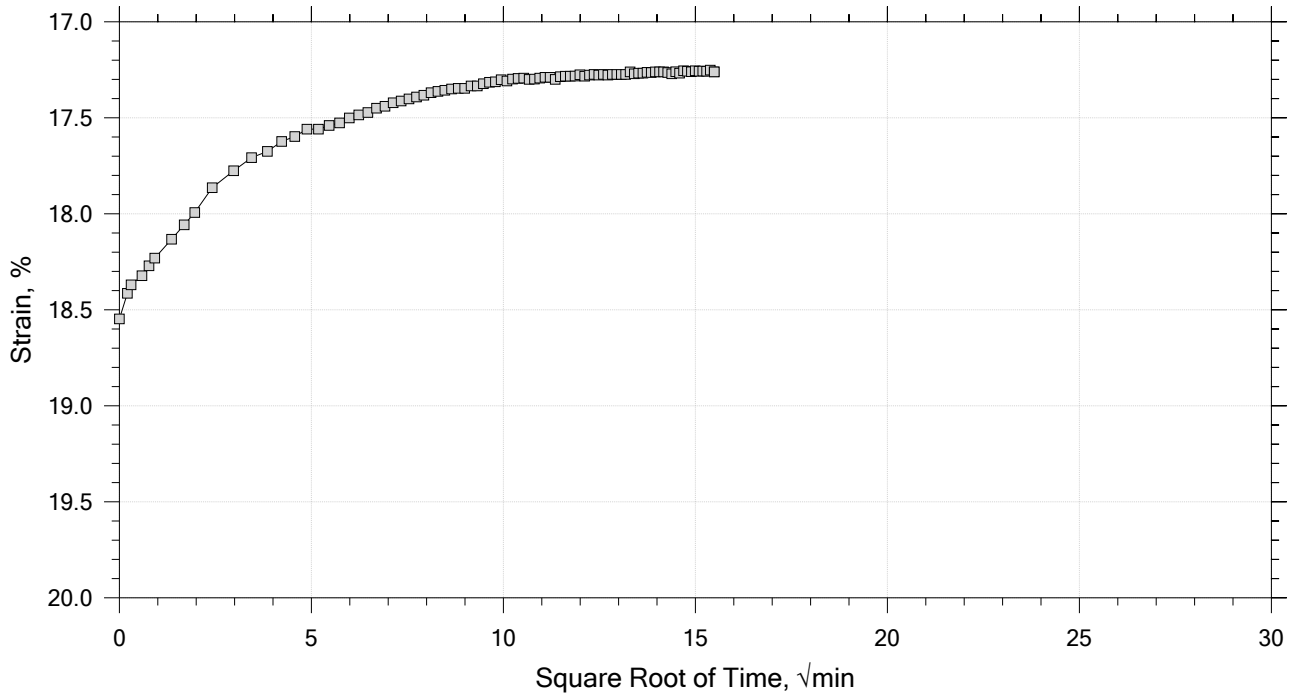


Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
Test No.: IP-1	Sample Type: tube	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B,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

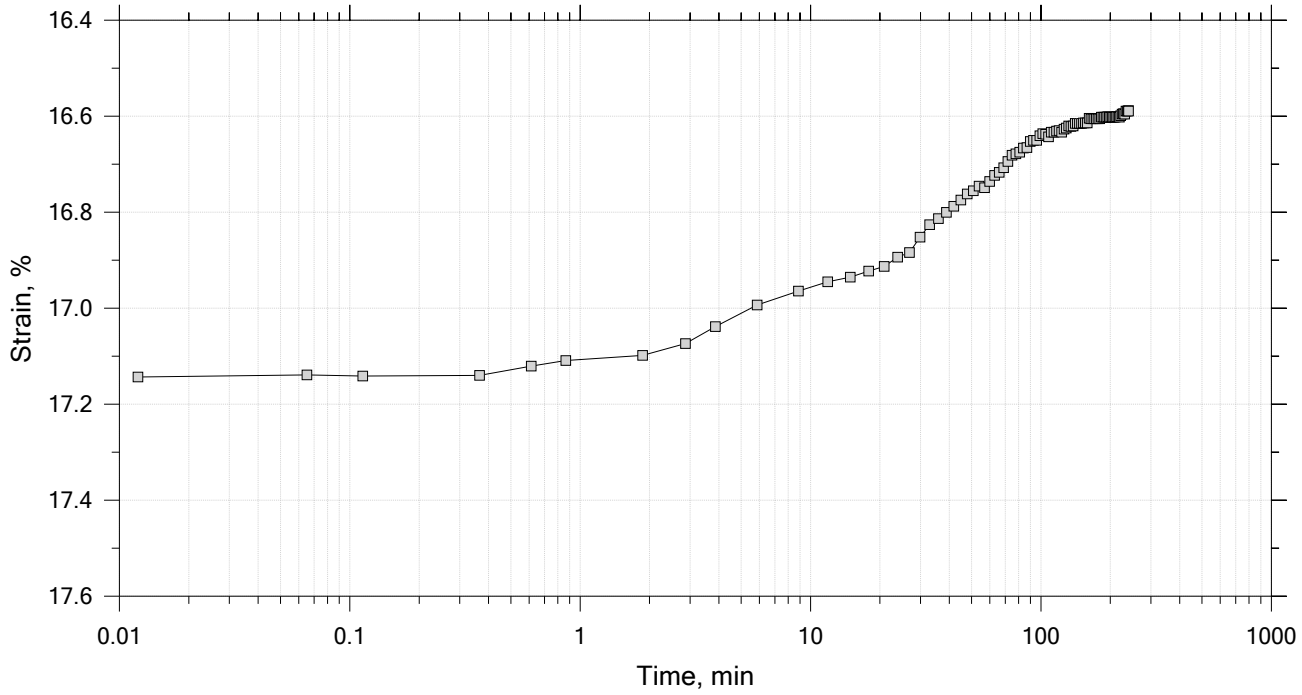


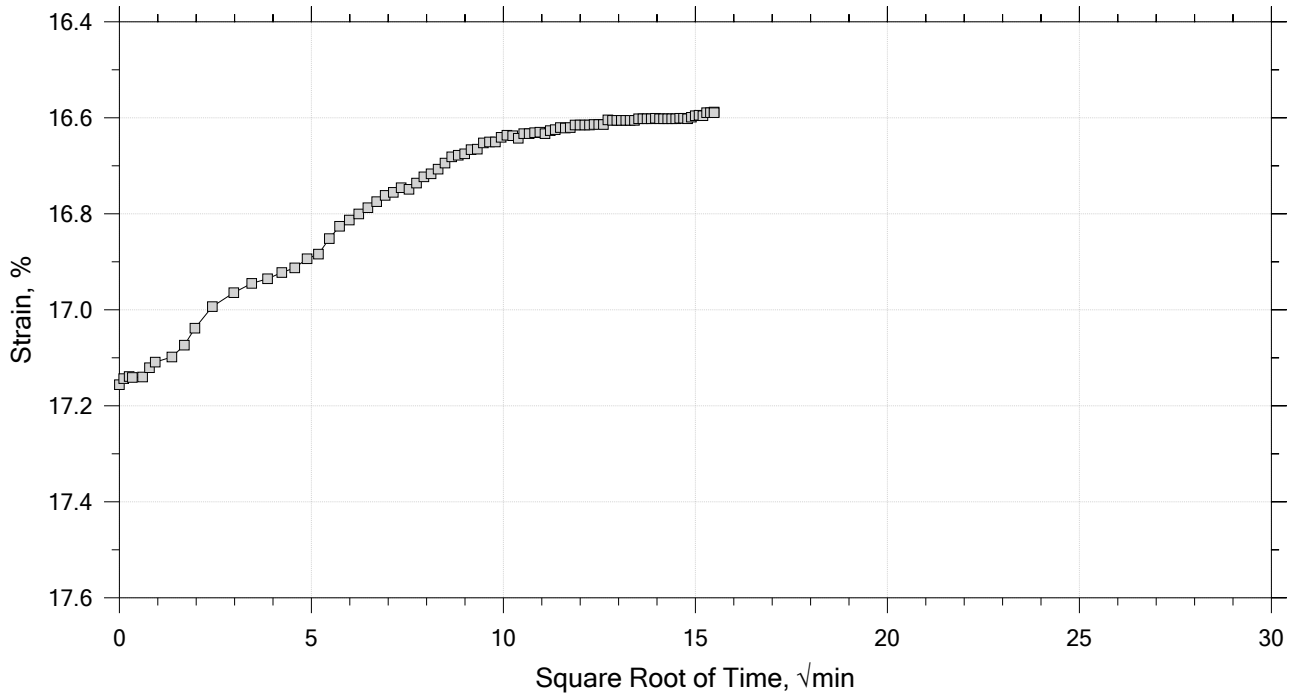


GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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





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

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

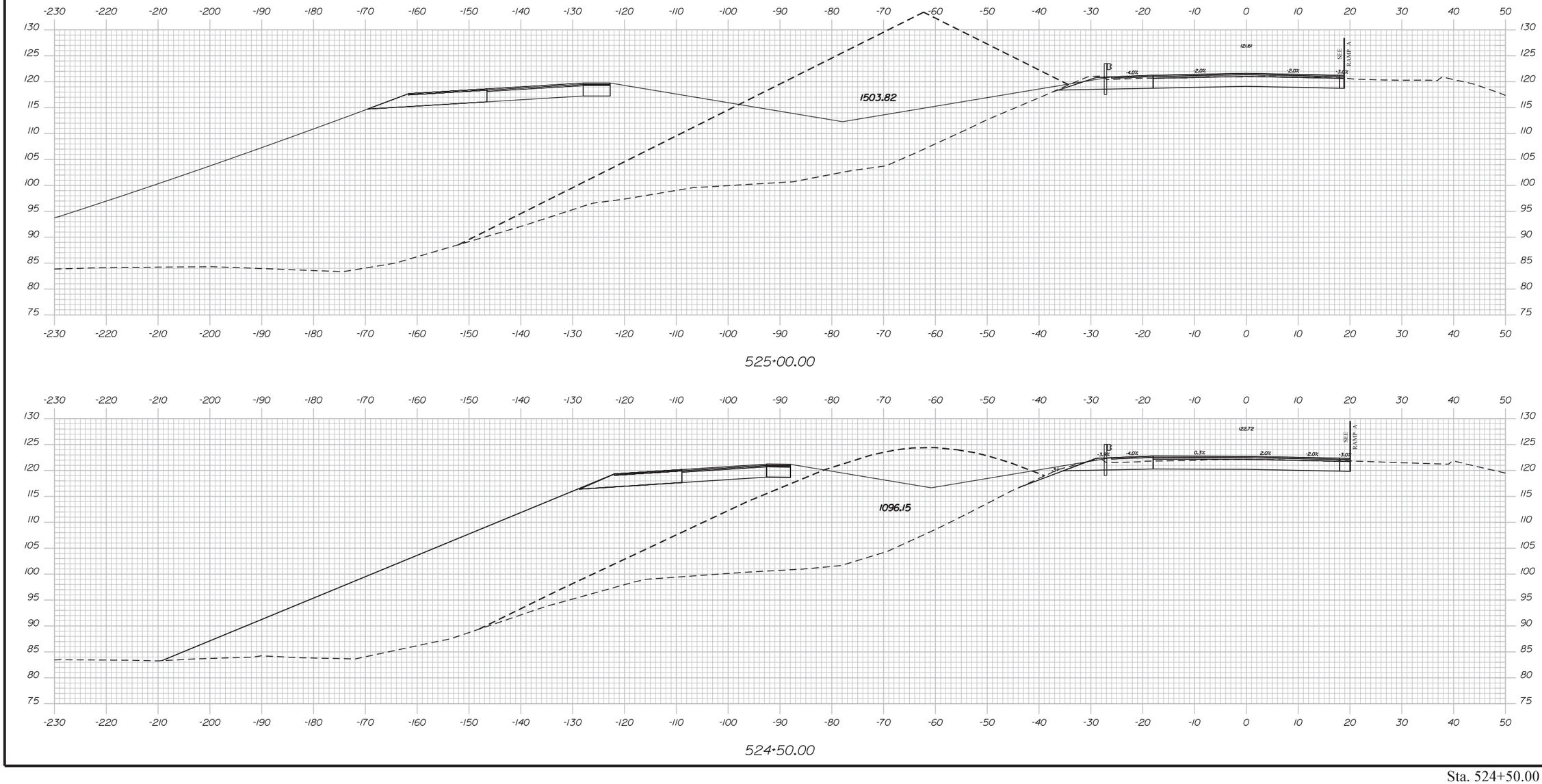
GeoTesting EXPRESS	Project: RT9/I-395 Connector-Wilson St.	Location: Brewer & Eddington, ME	Project No.: GTX-311345
	Boring No.: BB-BWS-301	Tested By: md	Checked By: anm
	Sample No.: 3U	Test Date: 02/18/20	Depth: 29-31 ft
	Test No.: IP-1	Sample Type: tube	Elevation: ---
	Description: Moist, gray clay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

Appendix D

Calculations

Preliminary Stockpile Cross Sections
(as provided by MaineDOT)

PRELIMINARY



525+00.00

524+50.00

Sta. 524+50.00 to Sta. 525+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018915.20

WIN
WIN 18915.20

BRIDGE NO. 1564

HISTORIC PLANS

PROJ. MANAGER	$projmanager1$		
DESIGN-DETAILED	$designer1$	$detailer1$	$projdate1$
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	$designer2$	$detailer2$	$projdate2$
DESIGN3-DETAILED3	$designer3$	$detailer3$	$projdate3$
REVISIONS 1	-----		$revdate1$
REVISIONS 2	-----		$revdate2$
REVISIONS 3	$revision3$		$revdate3$
REVISIONS 4	$revision4$		$revdate4$
FIELD CHANGES	$fieldchange$		$fieldDate$

SIGNATURE

P.E. NUMBER

DATE

WILSON STREET BRIDGE
I-395 AND ROUTE 9
BREWER PENOBSCOT COUNTY

CROSS SECTIONS

SHEET NUMBER

Xse

OF 9

Username: David.Show Date:1/9/2020

Division: BRIDGE

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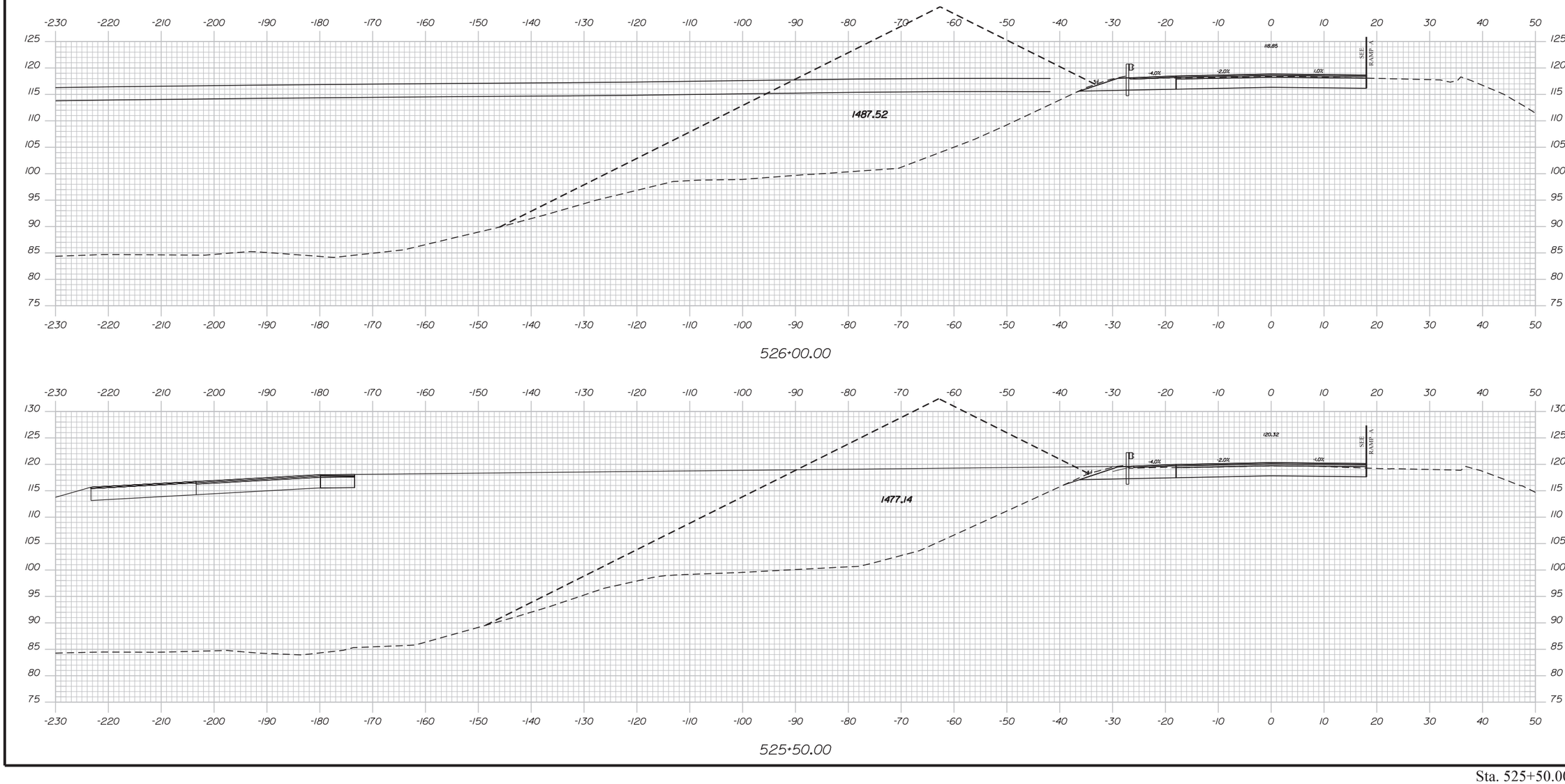
PRELIMINARY



Sta. 525+50.00 to Sta. 526+00.00

Username: David.Shaw Date:1/9/2020

Division: BRIDGE

Filename: ...\MSTA\Xsect_excavation.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018915.20

WIN
WIN 18915.20

BRIDGE NO. 1564

HISTORIC PLANS

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CHECKED-REVIEWED	-----	-----	-----
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REVISIONS 4	$revision4$		$revdate4$
FIELD CHANGES	$fieldchange$		$fielddate$

SIGNATURE

P.E. NUMBER

DATE

WILSON STREET BRIDGE
I-395 AND ROUTE 9
PENOBSCOT COUNTY
BREWER

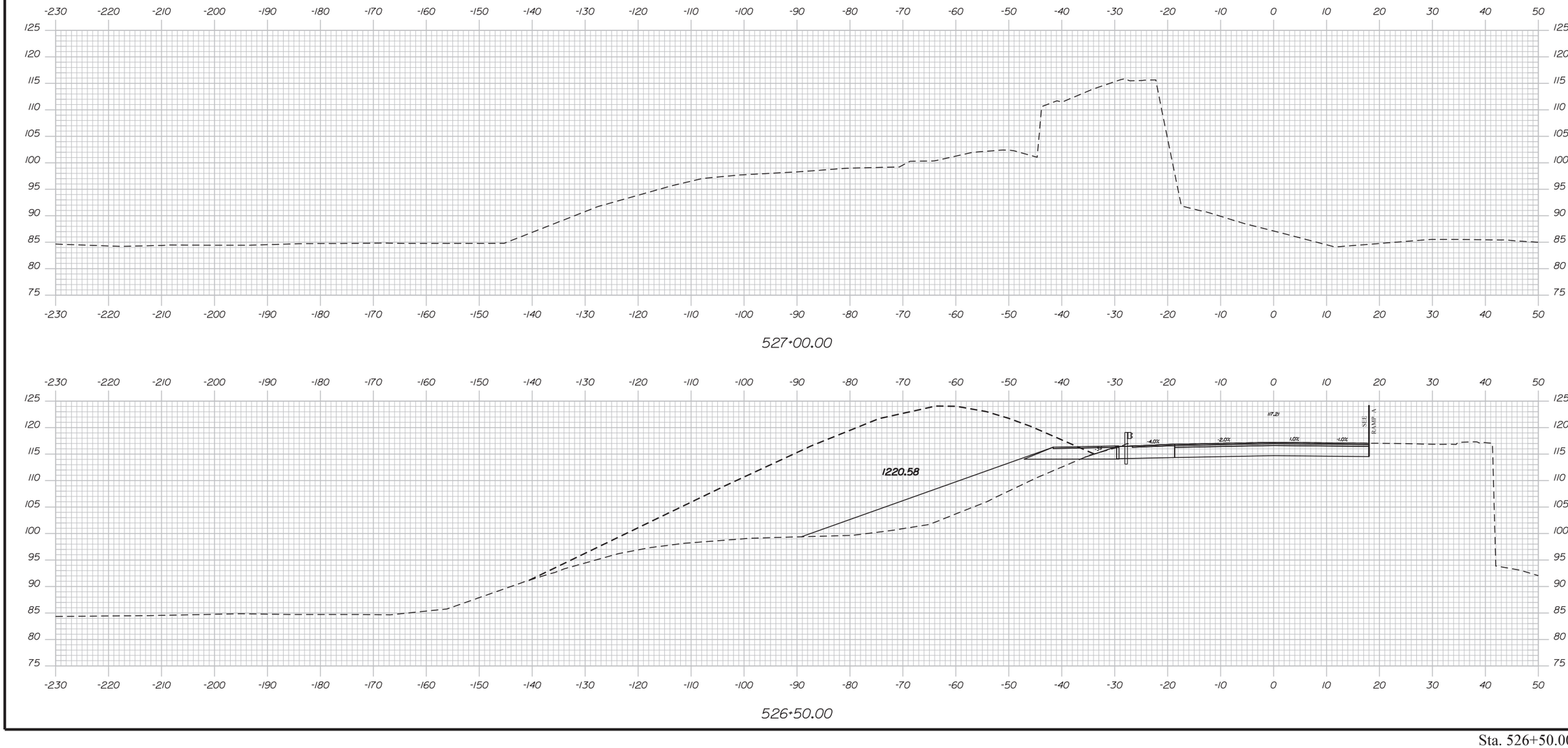
CROSS SECTIONS

SHEET NUMBER

Xse

OF 9

PRELIMINARY



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018915.20

WIN
WIN 18915.20

BRIDGE NO. 1564

HISTORIC PLANS

	BY	DATE
PROJ. MANAGER	$PROJMANAGER$	
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CHECKED-REVIEWED	-----	-----
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SIGNATURE

P.E. NUMBER

DATE

WILSON STREET BRIDGE
I-395 AND ROUTE 9
BREWER PENOBSCOT COUNTY

CROSS SECTIONS

SHEET NUMBER

Xse

OF 9

Sta. 526+50.00 to Sta. 527+00.00

Filename: ...\MSTA\Xsect_excavation.dgn Division: BRIDGE Username: David.Shaw Date:1/9/2020

Seismic Site Classification

PROBLEM STATEMENT & OBJECTIVE

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

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the test boring near the proposed stockpile (BB-BWS-301), recommend a **Seismic Site Class D**.

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
3. International Building Code 2009.
4. ASCE/SEI 7-05 Minimum Design Loads For Buildings and Other Structures
5. International Building Code 2012.
6. ASCE/SEI 7-10 Minimum Design Loads For Buildings and Other Structures

AVAILABLE INFORMATION

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

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. WOH/WOR = SPT N-value of 1.
3. For test boring BB-BWS-301, used Method C and assumed S_u values per field vane and historic boring data.

PROCEDURE

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

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

$$\overline{V_s} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\overline{V_{si}}}}$$

where

V_{si} = shear wave velocity of *i* th soil (ft/s).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

PROCEDURE

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

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th soil layer not to exceed 100 ft (blows/ft).
d_i = thickness of *i* th soil layer (ft).
n = total number of distinctive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and n.

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

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of *i* th cohesionless soil layer (blows/ft).
d_i = thickness of *i* th cohesionless soil layer (ft).
m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and m.

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

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

where

s_{ui} = undrained shear strength of *i* th cohesive soil layer (psf), not to exceed 5000 psf
d_i = thickness of *i* th cohesive soil layer (ft).
k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.
i = any one of the layers between 1 and k.

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

SITE CLASS DEFINITIONS

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

Table 3.4.2.1-1—Site Class Definitions

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

Exceptions:

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

where:

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

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

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

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

CALCULATIONS - METHOD C

Exploration ID: BB-BWS-301
Ground Surface El.: 100.5

					Cohesionless		Cohesive	
Sample Number	**Depth (ft)**	**Elevation (ft)**	**Description**	**d (ft)**	**SPT N (blows/ft)**	**d/N**	**Su (psf)**	**d/Su**
1D	0.0	100.5	SILT (Fill)	5.0	9	0.556		
2D	5.0	95.5	SILT (Fill)	5.0	12	0.417		
3D	10.0	90.5	SILT (Fill)	2.0	8	0.250		
4D	12.0	88.5	SILT (Fill)	7.0	6	1.167		
5D	19.0	81.5	CLAY (Marine Deposit)	5.0			1500	0.003
6D	24.0	76.5	CLAY (Marine Deposit)	10.0			1500	0.007
7D	34.0	66.5	CLAY (Marine Deposit)	5.0			800	0.006
8D	39.0	61.5	SAND (Glacial Till)	2.1	32	0.066		
R	41.1	59.4	BEDROCK	58.9	100	0.589		
					Σd/N =	3.044	Σd/Su =	0.016

Total Thickness of Cohesionless (ft) =	80.0	$N_{ch\text{-}bar}$ (blows/ft) =	26.3	$S_{u\text{-}bar}$ (psf) =	1231
Total Thickness of Cohesive (ft) =	20.0	Site $Class_N$ =	D	Site $Class_{Su}$ =	D
Total Thickness (ft) =	100.0	**Site Class =**	**D**		

RESULTS SUMMARY

Boring Number	Parameter	Average Value	Site Class
BB-BWS-301	$N_{ch\text{-}bar}$	26.3	D
BB-BWS-301	$S_{u\text{-}bar}$	1230.8	D

CONCLUSIONS & RECOMMENDATIONS

Based on the above results, recommend a Seismic Site Class D.

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Global Stability

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 1
Client:	Maine Department of Transportation	Date:	25-Mar-2020
Project:	I-395 - Route 9 Connector	Computed by:	EMS
Subject:	Global Stability	Checked by:	EAF

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for the proposed stockpile.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. Slide version 7.0 by RocScience.
3. Maine DOT Bridge Design Guide, 2003, with 2014 updates.

AVAILABLE INFORMATION

1. Plan set titled, "State of Maine Department of Transportation, Wilson Street Bridge, Cross Sections," dated January 2020.
2. Historic plan set and boring information titled "Wilson St. Over MCRR in the Town of Brewer, Penobscot County," dated 1982.

ASSUMPTIONS

1. Water level will be modeled at El. 88.5 based on water level observed in BB-BWS-103.
2. Seismic cases will have a seismic force of As/2 (0.107g/2) = 0.05 g based on the seismic site class calculations .
3. A 250 psf traffic surcharge will be modeled in Wilson Street.

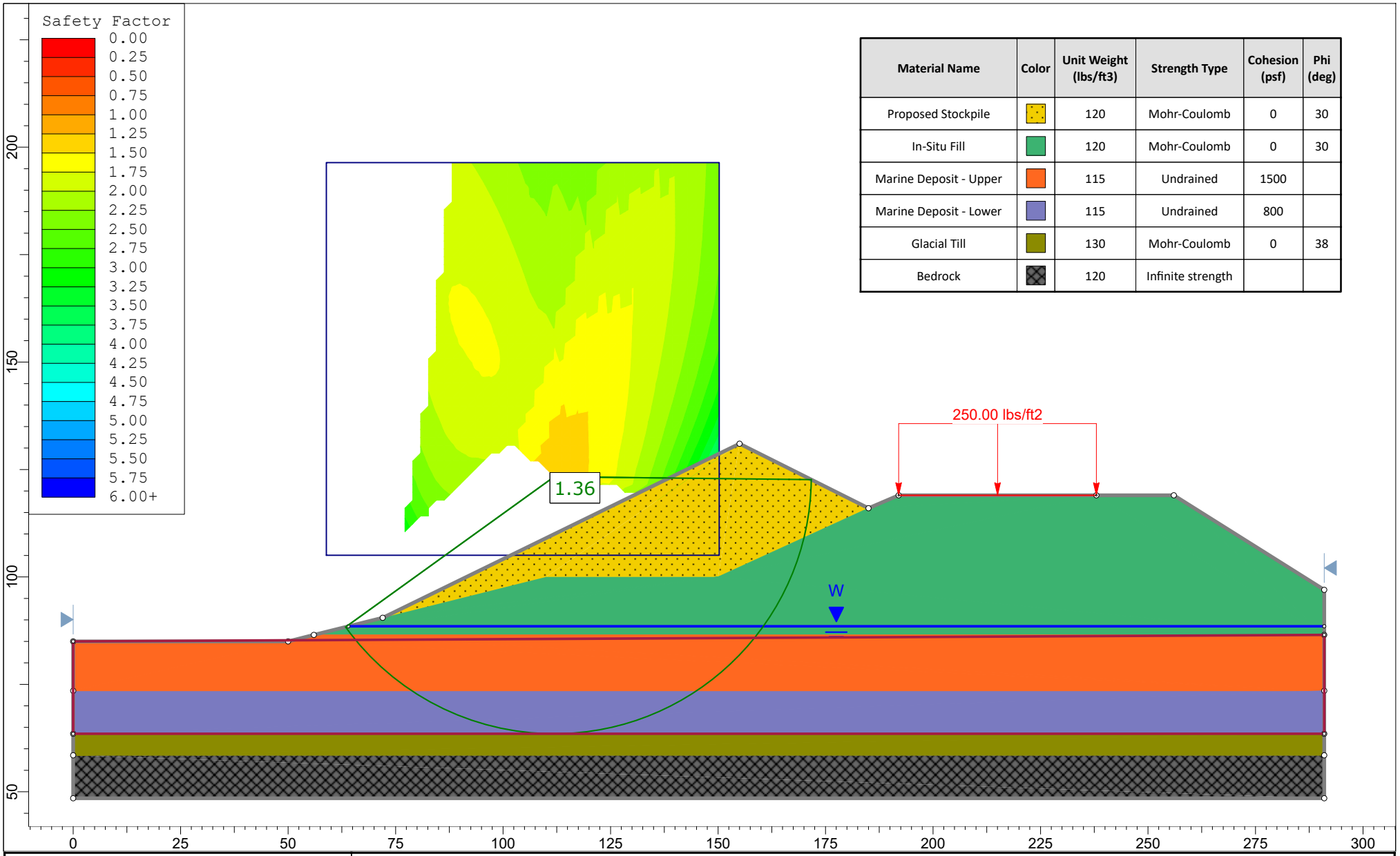
SOIL PROPERTIES

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Proposed Stockpile	120	30	0
In-Situ Fill	120	30	0
Marine Deposit - Upper	115	0	1500
Marine Deposit - Lower	115	0	800
Glacial Till	130	38	0
Bedrock	infinite strength		

RESULTS AND CONCLUSIONS

	F.S.	
	Static	Seismic
Stockpile	1.36	1.21

Based on AASHTO LRFD Section 11.6.2.3, an acceptable resistance factor for where the slope does not contain or support a structure is 0.75 (F.S. = 1/0.75 = 1.3).
Based on Maine DOT Bridge Design Guide Section 5.9.4, a minimum seismic factor of safety of 1.0 is acceptable for slope stability.

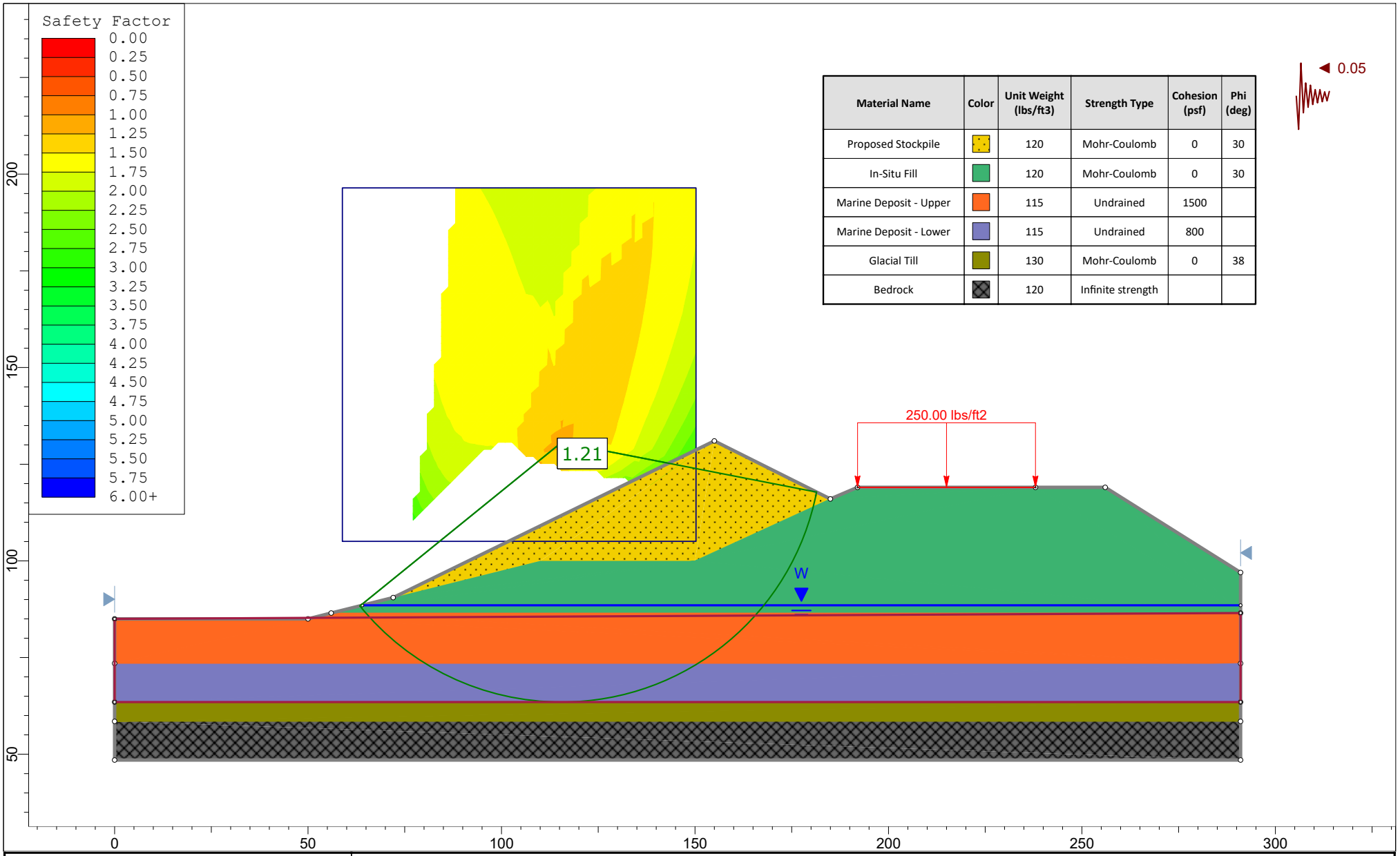


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Stockpile		120	Mohr-Coulomb	0	30
In-Situ Fill		120	Mohr-Coulomb	0	30
Marine Deposit - Upper		115	Undrained	1500	
Marine Deposit - Lower		115	Undrained	800	
Glacial Till		130	Mohr-Coulomb	0	38
Bedrock		120	Infinite strength		

rocscience

SLIDEINTERPRET 8.018

Project SLIDE - An Interactive Slope Stability Program		
Analysis Description		
Drawn By	*Scale* 1:370	*Company*
Date 3/25/2020, 4:13:43 PM		*File Name* 2020-0410-HAI-Stockpile Global Stability-static-D3.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Proposed Stockpile		120	Mohr-Coulomb	0	30
In-Situ Fill		120	Mohr-Coulomb	0	30
Marine Deposit - Upper		115	Undrained	1500	
Marine Deposit - Lower		115	Undrained	800	
Glacial Till		130	Mohr-Coulomb	0	38
Bedrock		120	Infinite strength		

rocscience

SLIDEINTERPRET 8.018

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description			
Drawn By		*Scale* 1:411	*Company*
Date	3/25/2020, 4:13:43 PM		*File Name* 2020-0410-HAI-Stockpile Global Stability-seismic-D3.slmd

Settlement

HALEY ALDRICH	CALCULATIONS	File No.:	132076-005
		Sheet:	1 of 2
Client:	Maine Department of Transportation	Date:	9-Apr-2020
Project:	I-395 - Route 9 Connector	Computed by:	EMS
Subject:	Settlement of Stockpile	Checked by:	EAF

PROBLEM STATEMENT & OBJECTIVE

Calculate the settlement for the proposed soil stockpile.

REFERENCES

1. Settle3D version 4.0 by RocScience.

AVAILABLE INFORMATION

1. Current boring log BB-BWS-301.
2. Plan set titled, "Wilson Street Bridge, I-395 and Route 9, Cross Sections," by MaineDOT dated 9 January 2020.

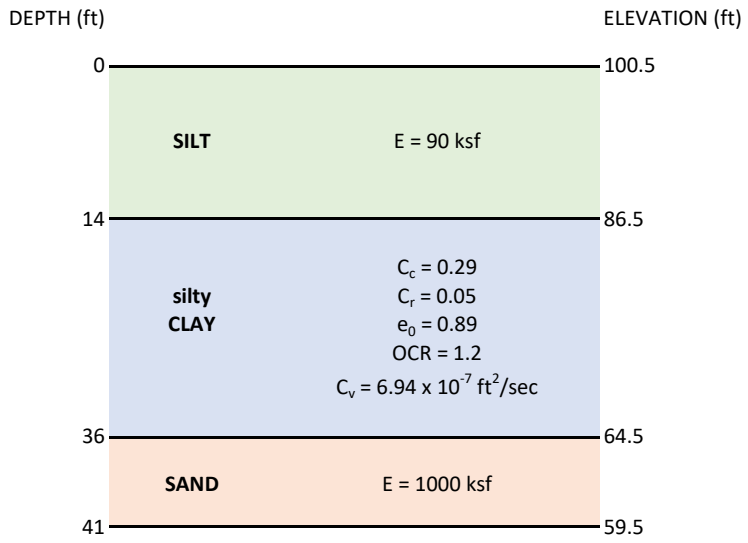
ASSUMPTIONS

1. Stockpile is placed adjacent to existing roadway between Sta. 524+00 and 527+00.
2. Stockpile is triangular with a base width of 120 ft.
3. Stockpile height varies, up to a maximum height of 26 ft.
4. Stockpile unit weight is 125 pcf.
5. Groundwater depth is 12 ft.
6. Elastic soil moduli estimated after AASHTO 2004 with 2006 interims.

STOCKPILE GEOMETRY

Station	Max. Stockpile Height (ft)	Load Magnitude (ksf)
524+00	0	0.000
524+50	17	2.125
525+00	23	2.875
525+50	23	2.875
526+00	26	3.250
526+50	21	2.625
527+00	0	0.000

SOIL PROFILE AND PROPERTIES (not to scale)



SUMMARY OF RESULTS

Time After Stockpile Placement (months)	At Center of Stockpile			At Northern Edge of Wilson Street Shoulder			At Center of Wilson Street		
	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)	Elastic Settlement (in)	Consolidation Settlement (in)	Total Settlement (in)
6	4.5	2.5	7.0	0.5	0.5	1.0	<1/4	<1/4	1/4
9	4.5	3.0	7.5	0.5	0.5	1.0	<1/4	<1/4	1/4
12	4.5	3.5	8.0	0.5	0.5	1.0	<1/4	1/4	1/4

Settle3D Analysis Information

Project Settings

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Stress Computation Method	Westergaard
Time-dependent Consolidation Analysis	
Time Units	months
Permeability Units	feet/second
Minimum settlement ratio for subgrade modulus	0.9

Use average properties to calculate layered stresses

Improve consolidation accuracy

Ignore negative effective stresses in settlement calculations

Stage Settings

Stage #	Name	Time [months]
1	Initial	0
2	6 Months	6
3	9 Months	9
4	1 Year	12

Results

Time taken to compute: 4.76856 seconds

Stage: Initial = 0 mon

Data Type	Minimum	Maximum
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Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	4.42418
Secondary Settlement [in]	0	0
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Loading Stress YY [ksf]	0	0
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Effective Stress XX [ksf]	0	3.0504
Effective Stress YY [ksf]	0	3.0504
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Total Stress XX [ksf]	0	5.7851
Total Stress YY [ksf]	0	5.7851
Modulus of Subgrade Reaction (Total) [ksf/ft]	-7.97069e-005	9.22378
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-7.97069e-005	9.22378
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.0360548
Pore Water Pressure [ksf]	0	2.7347
Excess Pore Water Pressure [ksf]	0	1.99702
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.00144863	3.4331
Over-consolidation Ratio	1	1.2
Void Ratio	0	0.89
Permeability [ft/s]	0	3.19751e-010
Coefficient of Consolidation [ft^2/s]	0	6.94e-007
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	0
Undrained Shear Strength	0	0.0547325

Stage: 6 Months = 6 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	6.87763
Total Consolidation Settlement [in]	0	2.46492
Virgin Consolidation Settlement [in]	0	1.97297
Recompression Consolidation Settlement [in]	0	0.492587
Immediate Settlement [in]	0	4.42418
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-2.11154e-006	3.25
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	3.9755
Effective Stress XX [ksf]	0	3.9755
Effective Stress YY [ksf]	0	3.9755
Total Stress ZZ [ksf]	0	5.7851
Total Stress XX [ksf]	0	5.7851
Total Stress YY [ksf]	0	5.7851
Modulus of Subgrade Reaction (Total) [ksf/ft]	-5.6282e-005	6.54823
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-7.97069e-005	9.22378
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000191509	28.1057
Total Strain	9.58843e-008	0.0424321
Pore Water Pressure [ksf]	0	2.05112
Excess Pore Water Pressure [ksf]	0	1.23992
Degree of Consolidation [%]	0	99.9544
Pre-consolidation Stress [ksf]	0.00144863	3.97321
Over-consolidation Ratio	1	1.19897
Void Ratio	0	0.889981
Permeability [ft/s]	0	1.85456e-009
Coefficient of Consolidation [ft^2/s]	0	6.94e-007
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	51.034
Undrained Shear Strength	0	0.0615378

Stage: 9 Months = 9 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	7.35963
Total Consolidation Settlement [in]	0	2.95273
Virgin Consolidation Settlement [in]	0	2.40264
Recompression Consolidation Settlement [in]	0	0.550731
Immediate Settlement [in]	0	4.42418
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-2.11154e-006	3.25
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	3.9755
Effective Stress XX [ksf]	0	3.9755
Effective Stress YY [ksf]	0	3.9755
Total Stress ZZ [ksf]	0	5.7851
Total Stress XX [ksf]	0	5.7851
Total Stress YY [ksf]	0	5.7851
Modulus of Subgrade Reaction (Total) [ksf/ft]	-5.28309e-005	6.17885
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-7.97069e-005	9.22378
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.000156682	23.0571
Total Strain	9.58843e-008	0.0424846
Pore Water Pressure [ksf]	0	1.89696
Excess Pore Water Pressure [ksf]	-0.000209771	1.08576
Degree of Consolidation [%]	0	99.9833
Pre-consolidation Stress [ksf]	0.00144863	3.97321
Over-consolidation Ratio	1	1.19813
Void Ratio	0	0.889966
Permeability [ft/s]	0	1.85456e-009
Coefficient of Consolidation [ft^2/s]	0	6.94e-007
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	61.2546
Undrained Shear Strength	0	0.0615968

Stage: 1 Year = 12 mon

Data Type	Minimum	Maximum
Total Settlement [in]	0	7.76687
Total Consolidation Settlement [in]	0	3.36892
Virgin Consolidation Settlement [in]	0	2.8159
Recompression Consolidation Settlement [in]	0	0.553012
Immediate Settlement [in]	0	4.42418
Secondary Settlement [in]	0	0
Loading Stress ZZ [ksf]	-2.11154e-006	3.25
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	3.9755
Effective Stress XX [ksf]	0	3.9755
Effective Stress YY [ksf]	0	3.9755
Total Stress ZZ [ksf]	0	5.7851
Total Stress XX [ksf]	0	5.7851
Total Stress YY [ksf]	0	5.7851
Modulus of Subgrade Reaction (Total) [ksf/ft]	-5.02936e-005	5.91624
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	-7.97069e-005	9.22378
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	-0.00013629	20.4518
Total Strain	9.58843e-008	0.0425124
Pore Water Pressure [ksf]	0	1.8096
Excess Pore Water Pressure [ksf]	0	0.938423
Degree of Consolidation [%]	0	99.9504
Pre-consolidation Stress [ksf]	0.00144863	3.97321
Over-consolidation Ratio	1	1.19732
Void Ratio	0	0.889951
Permeability [ft/s]	0	1.85456e-009
Coefficient of Consolidation [ft^2/s]	0	6.94e-007
Hydroconsolidation Settlement [in]	0	0
Average Degree of Consolidation [%]	0	69.1815
Undrained Shear Strength	0	0.0616281

Loads

1. Rectangular Load: "Rectangular Load 1"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
-1.06581e-014	4.61853e-014	0
90	4.61853e-014	0
90	50	2.125
-1.06581e-014	50	0

2. Rectangular Load: "Rectangular Load 2"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
0	50	0
90	50	2.125
90	100	2.875
0	100	0

3. Rectangular Load: "Rectangular Load 3"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
0	100	0
90	100	2.875
90	150	2.875
0	150	0

4. Rectangular Load: "Rectangular Load 4"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
0	150	0
90	150	2.875
90	200	3.25
0	200	0

5. Rectangular Load: "Rectangular Load 5"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
0	200	0
90	200	3.25
90	250	2.625
0	250	0

6. Rectangular Load: "Rectangular Load 6"

Length	90 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	4500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
0	250	0
90	250	2.625
90	300	0
0	300	0

7. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	3.55271e-015	0
120	3.55271e-015	0
120	50	0
90	50	2.125

8. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	50	2.125
120	50	0
120	100	0
90	100	2.875

9. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	100	2.875
120	100	0
120	150	0
90	150	2.875

10. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	150	2.875
120	150	0
120	200	0
90	200	3.25

11. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	200	3.25
120	200	0
120	250	0
90	250	2.625

12. Rectangular Load: "Rectangular Load 7"

Length	30 ft
Width	50 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	1500 ft^2
Depth	0 ft
Installation Stage	Initial = 0 mon

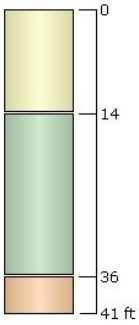
Coordinates and Load

X [ft]	Y [ft]	Load Magnitude [ksf]
90	250	2.625
120	250	0
120	300	0
90	300	0

Soil Layers

Ground Surface Drained: Yes

Layer #	Type	Thickness [ft]	Depth [ft]	Drained at Bottom
1	Existing Fill SILT	14	0	No
2	Marine Deposit CLAY	22	14	Yes
3	Glacial Till SAND	5	36	Yes



Soil Properties

Property	Existing Fill SILT	Marine Deposit CLAY	Glacial Till SAND
Color			
Unit Weight [kips/ft^3]	0.12	0.115	0.13
Saturated Unit Weight [kips/ft^3]	0.12	0.115	0.13
K0	1	1	1
Immediate Settlement	Enabled	Disabled	Enabled
Es [ksf]	90	-	1000
Esur [ksf]	90	-	1000
Primary Consolidation	Disabled	Enabled	Disabled
Material Type		Non-Linear	
Cc	-	0.29	-
Cr	-	0.05	-
e0	-	0.89	-
OCR	-	1.2	-
Cv [ft^2/s]	-	6.94e-007	-
Cvr [ft^2/s]	-	6.94e-007	-
B-bar	-	1	-
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Piezo Line ID	1	1	1

Groundwater

Groundwater method	Piezometric Lines
Water Unit Weight	0.0624 kips/ft^3

Piezometric Line Entities

ID	Depth (ft)
1	12 ft

Query Points

Point #	Query Point Name	(X,Y) Location	Number of Divisions
1	Query Point 1	126, 200	Auto: 53
2	Query Point 2	153, 200	Auto: 53

Query Lines

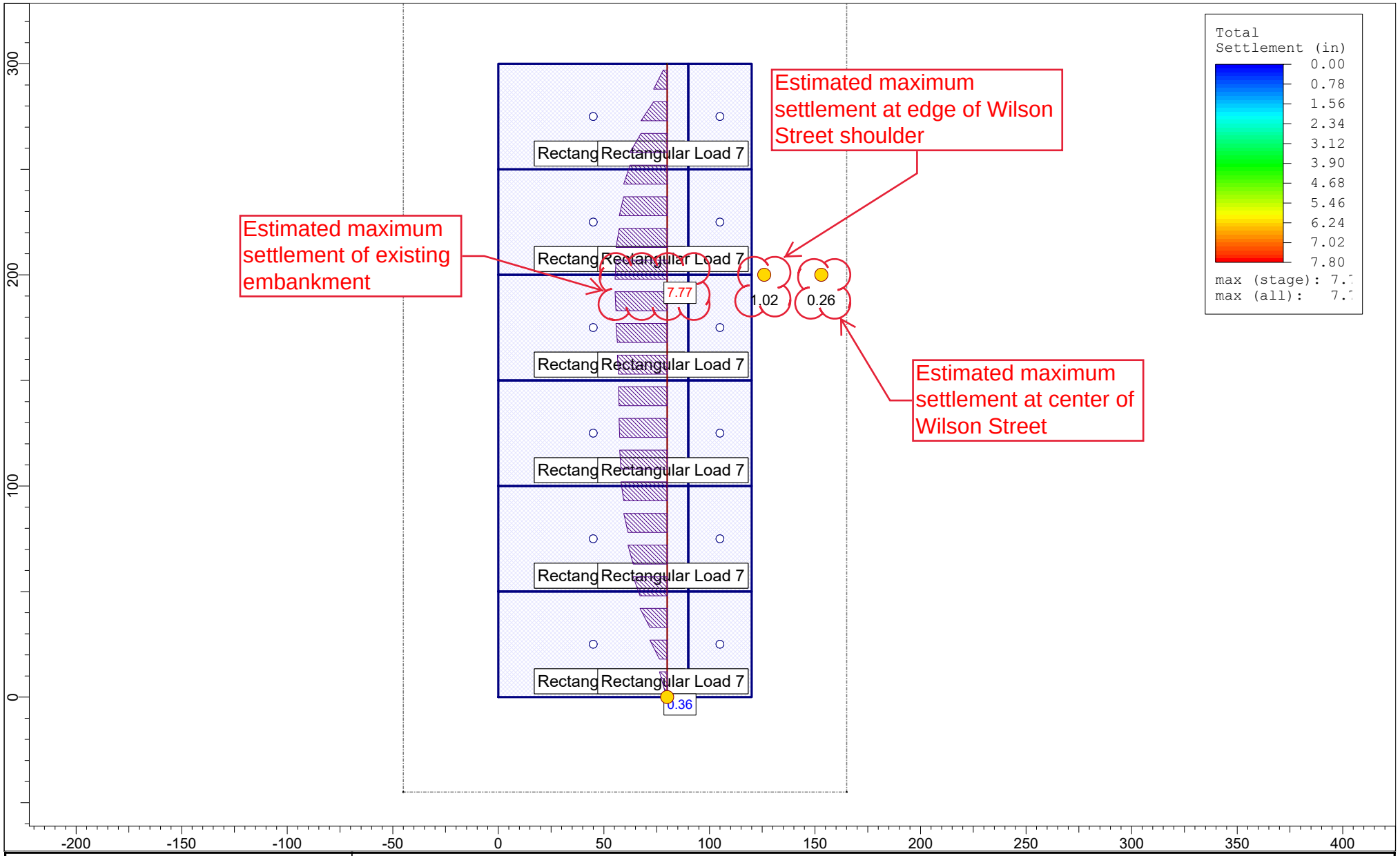
Line #	Query Line Name	Start Location	End Location	Horizontal Divisions	Vertical Divisions
1	Query Line 1	80, -4.26326e-014	80, 300	20	Auto: 53

Field Point Grid

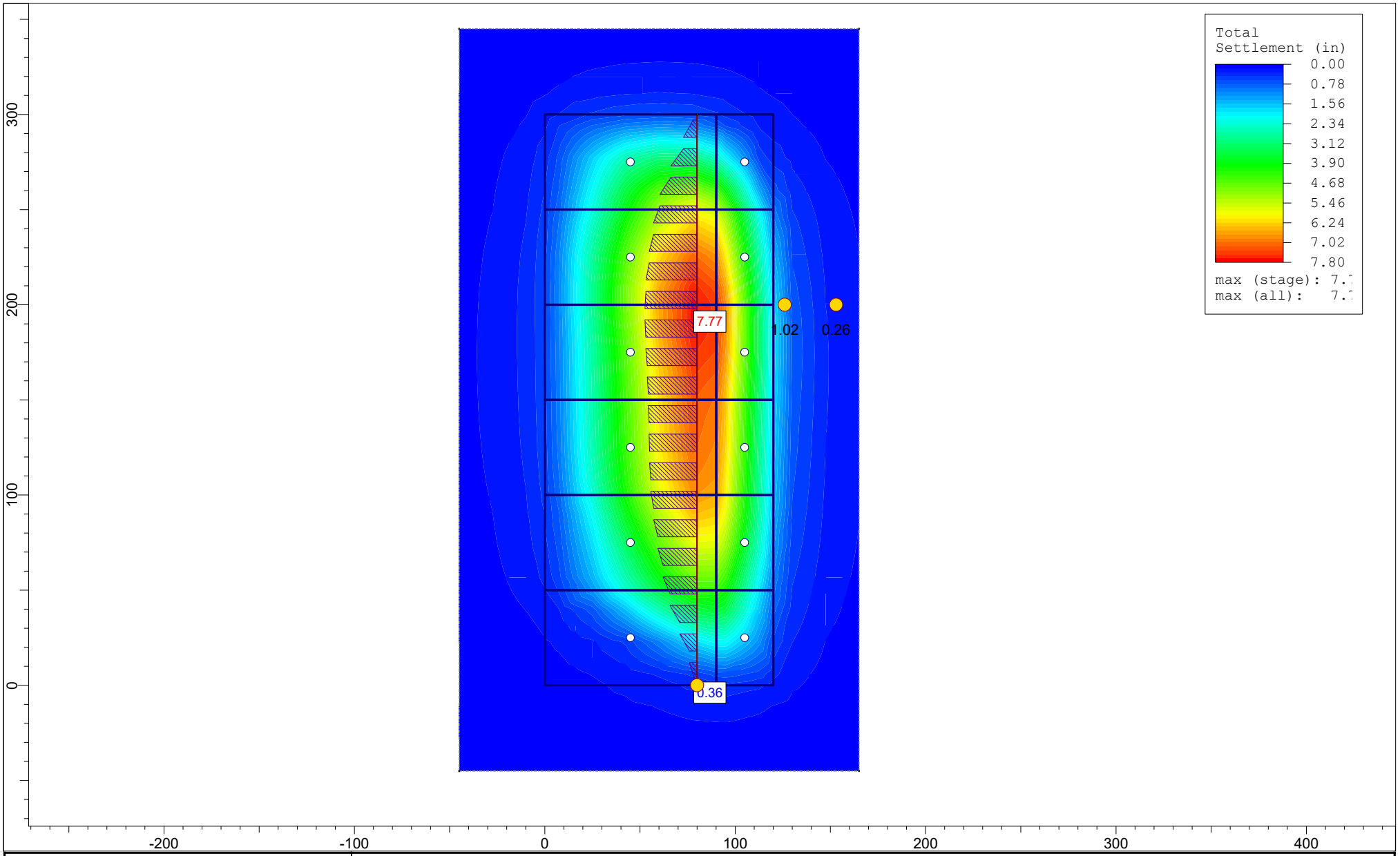
Number of points	436
Expansion Factor	2

Grid Coordinates

X [ft]	Y [ft]
165	345
165	-45
-45	-45
-45	345



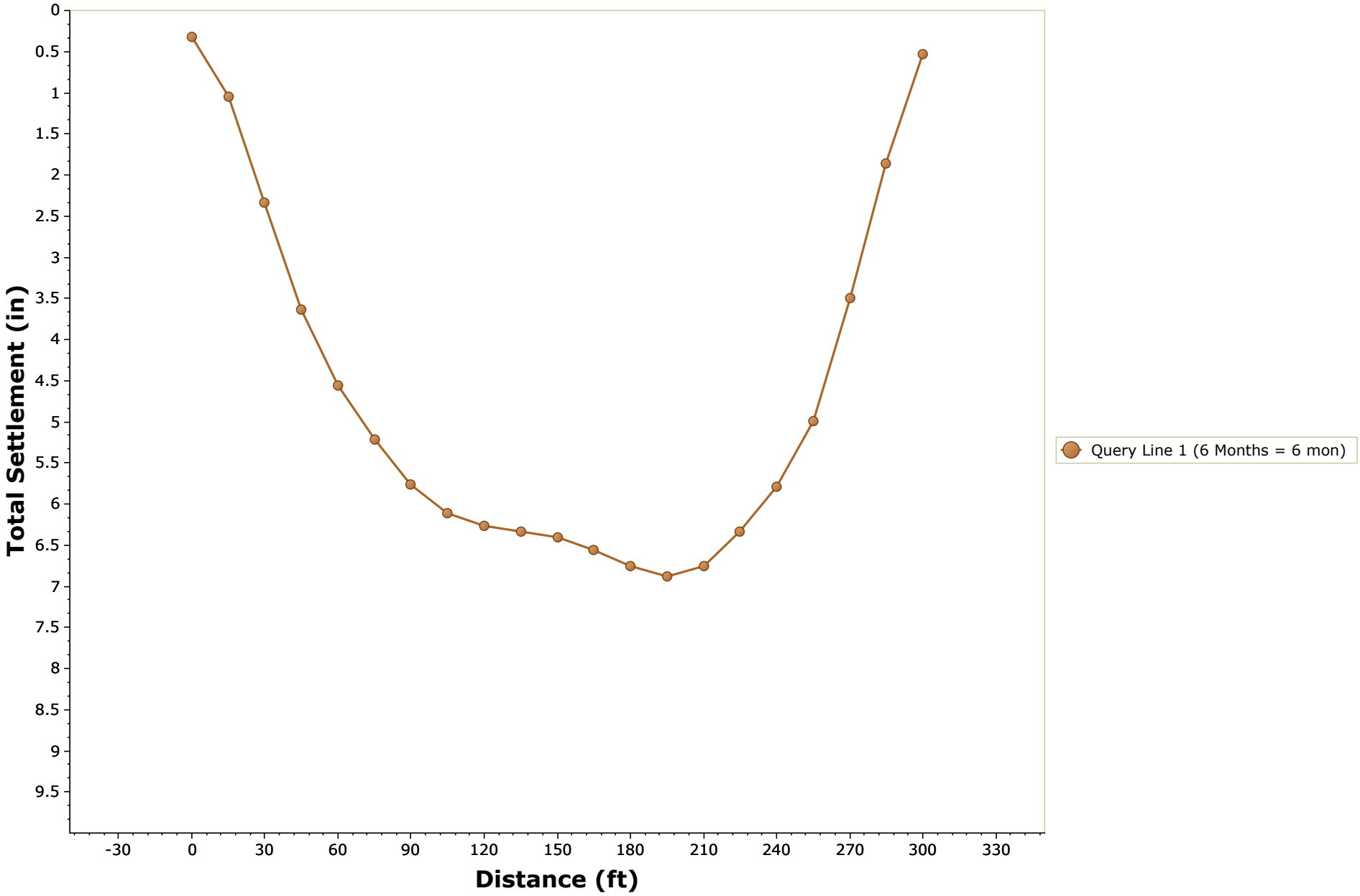
rocscience	Project		
	Analysis Description		
	Drawn By	Company	
SETTLE3D 4.010	Date: 3/25/2020, 8:56:34 AM	File Name: 2020-0409-Stockpile Settlement-D3.s3z	



rocscience	*Project*	
	Analysis Description	
	Drawn By	*Company*
	Date 3/25/2020, 8:56:34 AM	*File Name* 2020-0409-Stockpile Settlement-D3.s3z

SETTLE3D 4.010

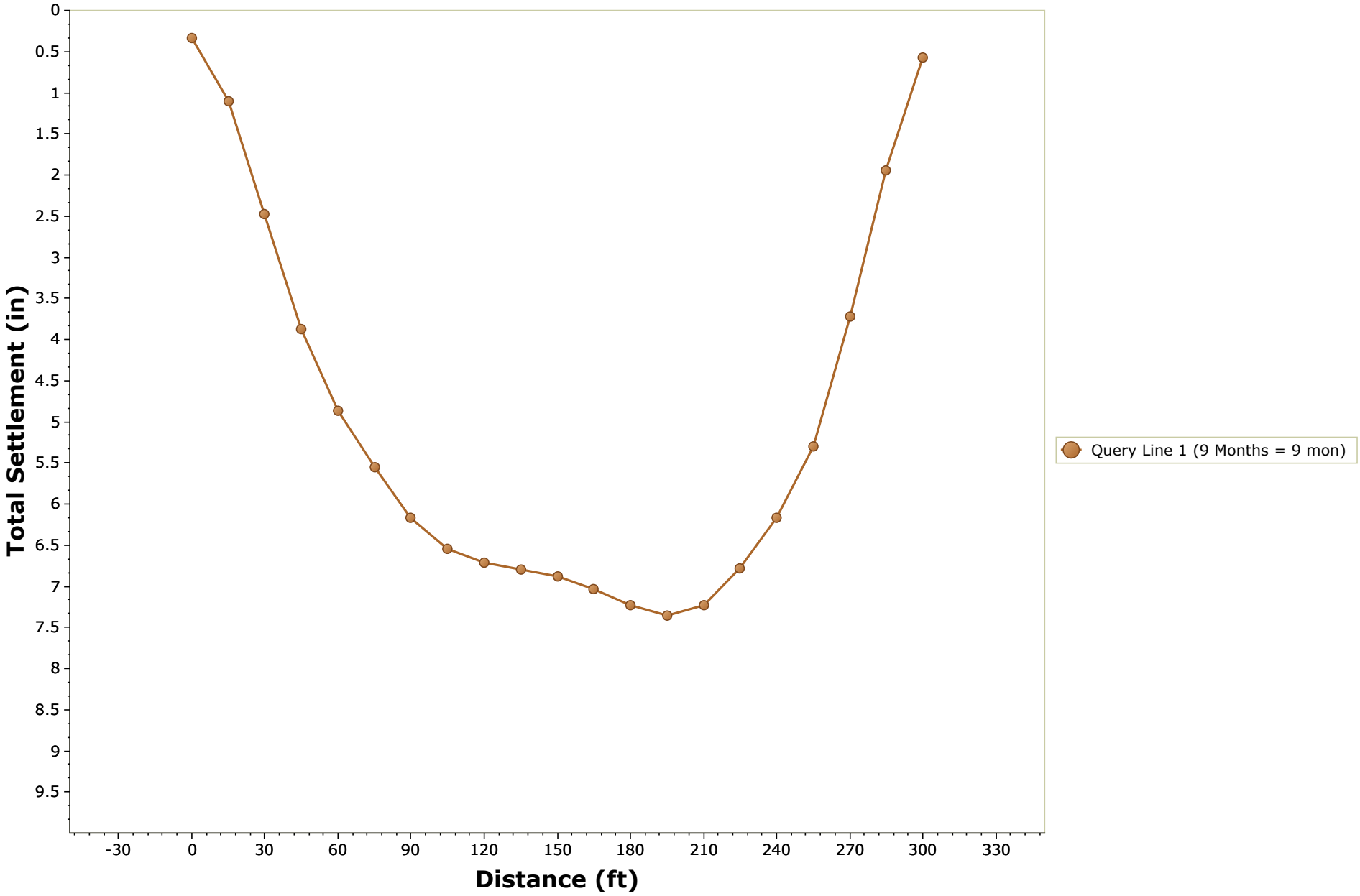
Distance vs. Total Settlement



Reference Stage: None

Total Settlement at Depth = 0 ft

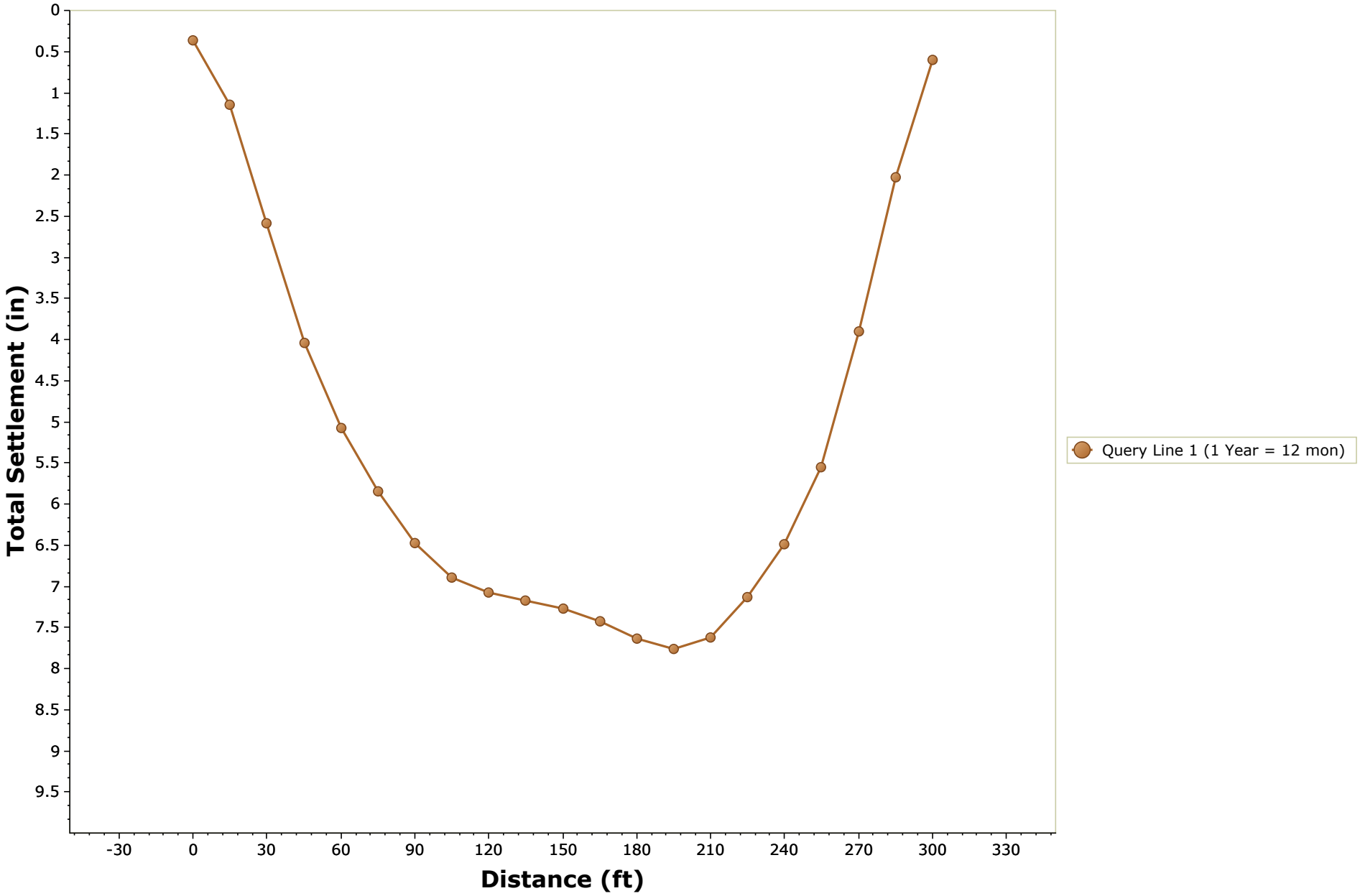
Distance vs. Total Settlement



Reference Stage: None

Total Settlement at Depth = 0 ft

Distance vs. Total Settlement



Reference Stage: None

Total Settlement at Depth = 0 ft