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GEOTECHNICAL DESIGN REPORT
JOHNSON ROAD BRIDGE NO. 5792
OVER INTERSTATE 295
FALMOUTH, MAINE

Prepared for:
Maine Department of Transportation
Augusta, Maine

March 2022
09.0026024.00

Prepared by:
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VIA EMAIL

March 22, 2022
File No. 09.0026024.00

Ms. Laura Krusinski
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Re: Geotechnical Design Report
Replacement of Johnson Road Bridge No. 5792 over Interstate 295
MaineDOT WIN 21721.00
Falmouth, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report, which includes geotechnical design recommendations for the replacement of Johnson Road Bridge No. 5792 in Falmouth, Maine. Our work was completed under GZA GeoEnvironmental, Inc.'s (GZA's) June 8, 2015 General Consulting Agreement (GCA CTM20150608000000000793) with the Maine Department of Transportation (MaineDOT) Bridge Program, and incorporates GZA's Proposal No. 09.P000017.20, dated May 7, 2019, and the *Limitations* Included in **Appendix A** of this report.

It has been a pleasure serving MaineDOT on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the replacement of Johnson Road Bridge No. 5792 in Falmouth, Maine. Our services were completed in accordance with GZA's June 8, 2015 General Consulting Agreement (GCA CTM20150608000000000793) with the Maine Department of Transportation (MaineDOT) Bridge Program, and incorporates GZA's Proposal No. 09.P000019.20, dated May 7, 2019, and the *Limitations* Included in **Appendix A** of this report. Hoyle, Tanner Associates, Inc. (Hoyle, Tanner) is serving as the bridge designer for MaineDOT.

1.1 BACKGROUND

The project includes replacement of the Johnson Road Bridge No. 5792 over Interstate 295 in Falmouth, Maine. The project location is shown on **Figure 1**. The existing bridge consists of an approximately 40-foot simply-supported span on the west approach, a common pier (Pier 1), and three additional spans with a 180-foot-long continuous superstructure creating a total length of approximately 220 feet. The bridge was built in 1957 and consists of steel stringers with a concrete bridge deck and bituminous overlay. The abutments are composed of reinforced concrete founded on H-piles. The total bridge width is 33 feet.

The existing approach embankments are at approximately El. 48 to 63 at the west and east approaches, respectively, which is approximately 23 to 25 feet above existing site grades. The existing plans show that a stabilization berm was constructed on the north and south sides of the west approach embankment, which was observed by GZA during our site visit.

An existing 9-foot-wide culvert is present beneath the roadway between approximately Sta. 2+15 and 2+60 carrying Chenery Brook from northwest to southeast beneath Johnson Road. We understand that the top and bottom of the culvert structure range between approximately 11.5 to 20.5 feet below the existing roadway grade. During a GZA site visit, we observed that the brook has eroded the eastern bank of the stream adjacent to the southern embankment slope at approximately Sta. 3+25.

It is GZA's understanding that a full bridge replacement is planned for the project. The replacement bridge is planned to consist of a two-span bridge with integral abutments and a center pier in the I-295 median, all supported by driven H-piles. The replacement bridge will be approximately 244 feet long and 41 feet wide. The roadway will be raised approximately 3.5 and 2.5 feet above existing pavement grades within about 100 feet of the bridge at the west and east approaches, up to approximately El. 53 to 65, respectively. The embankments will be widened to facilitate increased lane widths and a sidewalk along the north side of Johnson Road. The top of the embankment will extend approximately 8 feet north of the current roadway near the bridge approaches, from approximately Sta 2+00 to 4+32 and 6+77 to 10+50 with a maximum fill height of 10 feet above grade and a slope inclination of 2 horizontal to 1 vertical (2H:1V), except between Sta. 2+75 and 3+75 where a 1.5H:1V slope inclination with a Turf Reinforced Mat (TRM) is proposed and between the driveway at Sta. 2+70 and 2+75 and 3+75 to 4+00 where the inclination will transition from 2H:1V to 1.5H:1V. Additional details of the proposed embankments are presented later in this report. The centerline will be shifted slightly to the north as well within the project limits.

Based on the current plans, there is no planned grade raise or widening west of Sta. 2+75, over or close enough to the existing culvert to affect the structure. We understand the town of Falmouth is evaluating extending the proposed sidewalk over this culvert as a separate project, but that was not evaluated as part of the bridge project.

A temporary detour is planned for the duration of construction. Therefore, staged construction is not required. It is GZA's understanding that existing abutments, piers, and piles will not be reused to support the bridge.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide geotechnical design recommendations for the proposed bridge and approach modifications. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions during preliminary design and reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data;
- Coordinated and observed subsurface explorations, consisting of eight test borings and five piezocone penetration tests (CPTs), to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability and settlement of raised and widened embankments; frost susceptibility; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; downdrag considerations; lateral pile design considerations; pile drivability; lateral earth pressures on abutments and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles; lateral earth pressures; seismic design parameters; embankment settlement mitigation; embankment global stability mitigation; geotechnical construction considerations; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed an exploration program in 2019 consisting of eight (8) test borings and five (5) cone penetration tests (CPTs). The following sections describe the subsurface program.

2.1 TEST BORINGS

Borings were drilled using 3- and 4-inch casing, and drive- or spin-and-wash drilling techniques, as noted on the boring logs. Standard penetration testing (SPT) and split spoon sampling were performed continuously in the upper portion of some borings and generally at 5-foot typical intervals using a 24-inch-long, 1-3/8-inch inside-diameter sampler. The borings were generally backfilled with ¾-inch crushed stone and/or soil cuttings and topped with asphalt cold patch in roadway areas. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Additional details of each program are described below.

The as-drilled boring locations and elevations were surveyed by MaineDOT and provided to GZA and are shown on the logs and in **Figure 2**. Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 88).

Borings BB-FJR-102 and -107 were conducted by Summit Geoengineering Services (SGS) between May 28 and 30, 2019 and were drilled adjacent to CPT locations to measure undrained shear strengths for evaluation of site-specific coefficients necessary for CPT data interpretation. SGS conducted field vanes shear tests in pairs in 5-foot intervals within the clay layer.

Borings BB-FJR-101 and BB-FJR-103 through BB-FJR-106 were drilled between May 28 and August 21, 2019 by New England Boring Contractors (NEBC). The borings were drilled using a track-mounted Mobile B-53 drill rig and were drilled to depths of approximately 49 to 131 feet and terminated approximately 0 to 10 feet into bedrock. The SPTs were conducted using automatic hammer NEBC No. D19, which had a rated hammer efficiency factor at the time of drilling of 0.895. Field vane shear tests were taken in pairs at approximately 5- to 10-foot typical intervals within the silt and clay layers. Field vane shear tests were conducted with Geonor 55x110 mm or 65x130 mm rectangular vanes using procedures and rods in accordance with MaineDOT guidelines. Vane types used for each test are documented on the logs. Peak and residual torque values were measured and correlated to undrained shear strength values using the MaineDOT correlation charts. A total of 7 thin-walled tube samples were taken from the borings drilled by NEBC and logged by GZA for laboratory consolidation and shear strength testing.

2.2 CONE PENETRATION TESTING

GZA retained SGS to complete five (5) CPTs, designated CPT-FJR-101 through CPT-FJR-105, between May 28 and June 3, 2019.

The as-drilled CPT locations and elevations were surveyed by MaineDOT and provided to GZA as shown on **Figure 2**.

The CPTs were performed in accordance with ASTM D5778. CPT-FJR-101 through -103 and CPT-FJR -105 were advanced using a track-mounted PowerProbe 9500 VTR and CPT-FJR-104 was advanced with a truck-mounted PowerProbe 9630 Pro with a Vertek digital cone. CPTs were advanced to depths ranging from approximately 21.3 to 105.4 feet, where refusal was encountered. Parameters obtained include cone resistance (qc), sleeve friction (fs), and piezocone pore pressure (u2). Dissipation tests were conducted at CPT-FJR-101, -102, and -103 in the marine clay. Downhole shear wave velocity testing was conducted in each CPT (i.e., sCPTs) at 1-meter intervals throughout the CPT depths.

The data report submitted to GZA by SGS dated June 10, 2019 containing the raw CPT results is included in **Appendix C**. SGS also provided GZA with Excel files containing the raw data collected from the CPTs for use in our engineering evaluations.

GZA utilized the analytical software CPetIT by Geologismiki to develop reports of correlated soil types and engineering properties based on the raw data provided by SGS. These reports and a summary of empirical correlations associated with different properties are included in **Appendix C**.

3.0 LABORATORY TESTING

GZA retained three laboratories to complete a laboratory testing program, including Thielsch Engineering of Cranston, Rhode Island, to assess the gradation and index properties of the soil, and Soil Metrics of Cape Elizabeth, Maine and Geotesting of Acton, Massachusetts to assess shear strength and compressibility of cohesive soils. The testing program included:

- Ten (10) gradation analyses;
- Two (2) hydrometer tests;
- Twelve (12) MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Thirty-two (32) moisture content tests;
- Fourteen (14) Atterberg limits analyses;
- Six (6) incremental consolidation tests;
- Two (2) Constant Rate-of-Strain (CRS) consolidation tests;
- Four (4) K_o-consolidated undrained triaxial compression (CKoUTC) tests; and
- Five (5) K_o consolidated direct simple shear tests.

Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial unit in the vicinity of the bridges consists of the Presumpscot Formation, described as gray-blue, silty clay. Marine sand and gravel deposits and glacial till deposits were also mapped in the area of the bridges. Directly to the east of the bridge, thin drift is mapped, indicating less than 10 feet of glacial till over bedrock .

Bedrock in the vicinity of the site is mapped[2] as the Hutchins Corner Formation, consisting primarily of fine to medium grained, medium gray quartz-plagioclase-biotite granofels or gneiss, heavily intruded by pegmatite.

[1] Bernotavicz, Alexa, 1999, Surficial geology of the Portland East quadrangle, Maine: Maine Geological Survey, Open-File Map 99-95, map, scale 1:24,000. Maine Geological Survey Maps. 996. http://digitalmaine.com/mgs_maps/996

[2] Hussey, Arthur M., II, 2003, Bedrock geology of the Portland East quadrangle, Maine: Maine Geological Survey, Open-File Map 03-90, 12 p. report, 21 figures, 1 plate, photographs, color map, cross section, scale 1:24,000. Maine Geological Survey Maps. 33. http://digitalmaine.com/mgs_maps/33

4.2 SUBSURFACE PROFILE

Up to six soil units were encountered in the test borings and CPTs overlying bedrock: Fill, Upper Marine Sand, Marine Clay Crust, Marine Clay, Lower Marine Sand, and Glacial Till. Approximately 1 to 2 feet of asphalt pavement was encountered in the roadway borings. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Soil descriptions are based on GZA borings and CPTs. Detailed descriptions and interpretations of the materials encountered at specific locations are provided in the boring logs and CPTs in **Appendices B and C**. An interpretive subsurface profile was developed representing generalized stratification along the proposed bridge alignment and is presented on **Figure 3**. The strata thicknesses and elevations of the borings are summarized in **Table 1**.

GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Fill	2 to 25	Brown, loose to very dense, fine to coarse SAND, trace to some Gravel, trace to some Silt. (USCS: SP-SM, SM, SP). MaineDOT Frost Classification: 0-III *Encountered in all borings.*
Upper Maine Sand	4 to 15	Varies from: Grey, loose to medium dense, silty Sand to: fine to coarse SAND, little to some Gravel, trace Silt. (USCS: SM, SP-SM, SP) MaineDOT Frost Classification: I-III *Encountered in all borings except BB-FJR-102, -102A, and -106.*
Marine Clay Crust	3 to 11	Grey to olive, medium stiff to stiff, Silty CLAY, trace to some Sand. (USCS: CL) MaineDOT Frost Classification: III-IV *Encountered in all borings.*
Marine Clay	5 to 37	Grey, soft to stiff, Silty CLAY, trace to little Sand. (USCS: CL) MaineDOT Frost Classification: III-IV *Encountered in all borings except BB-FJR-106 and -107.*
Lower Marine Sand	4 to 33	Grey/brown, very loose to dense, fine to coarse SAND, trace to some Silt, trace Gravel. (USCS: SM, SP-SM) MaineDOT Frost Classification: II-III *Encountered in all borings except BB-FJR-106 and -107.*
Glacial Till	0 to 22	Grey, very dense, fine to coarse SAND, little to some Gravel, little to some Silt. (USCS: SM, SP-SM) MaineDOT Frost Classification: II *Encountered in boring BB-FJR-103*
Top of Bedrock Elevation	Encountered Top of Rock:	Abutment 1: Approximate El. -72.6 Center Pier: Approximate El. -19.9 Abutment 2: Approximate El. 23.3

Additional index and engineering properties of the Marine Clay determined from the laboratory test data are summarized below and plotted on **Figure 4**:

- Twenty-five (25) water content tests: 46 percent to 25 percent (generally decreases with depth)
- Fourteen (14) Atterberg limits tests: LL=47-31, PI=25-14 (generally decreases with depth)

- 59 In-situ Field Vanes and 9 Laboratory Shear Strength Tests; Peak Shear Strength=250 to 1,950 pounds per square foot (psf), Residual Shear Strength=14 to 577 psf
  - Peak Undrained shear strength S_u=250 – 800 psf (outside of existing embankment)
  - Undrained shear strength S_u=450 – 1,950 psf (under existing embankment)
- Eight consolidation tests:
  - Preconsolidation Pressure=2,100-3,100 psf
  - Virgin Compression Ratio (CR)=0.16-0.25
  - Recompression Ratio (RR)=0.012-0.025
  - Coefficient of Consolidation (Cv)=0.025-0.21 ft^2/day

4.2.1 Bedrock

Bedrock was cored in borings BB-FJR-103, -105 and -106. Bedrock was described as very hard to hard, fresh, medium grained, gray and white, Schist/Gneiss with some pegmatite intrusions. In general, the joints are described as very close to widely spaced, low to high angle, smooth to rough, planar to undulating, fresh to discolored, and tight to wide. A hard, fresh, aphanitic, black, Slate was encountered in the bottom of BB-FJR-103. The Rock Quality Designation (RQD) in the core runs ranged from 80 to 93 percent.

4.2.2 Groundwater

The groundwater level was measured in the completed borings after drilling at depths of 3.6 to 16.4 feet below existing grade, corresponding approximately to El. 21.4 to 50.4. Water levels measured in the borings were likely influenced by the addition of drill water during rotary wash drilling. The groundwater level was also interpreted from the CPTs to be at depths of 5 to 15 feet, corresponding approximately to El. 24.0 to El. 39.1. The CPT measurements are considered more representative of in-situ groundwater conditions. In general, the data indicate that groundwater was encountered approximately 5 to 13 feet below grade outside of the existing embankment footprint.

A groundwater observation well was installed in boring BB-FJR-104 to a depth of 70 feet bgs, in the sand beneath the marine clay, and the initial groundwater level was measured to a depth of 3.6 feet bgs. An additional five ground water level readings were taken between August 19, 2019, and August 21, 2019, and all measured to be approximately 0.6 feet bgs corresponding to EL. 28.9, which appears to represent artesian conditions.

Fluctuations in groundwater levels will occur due to variations in season, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary from those encountered in the borings at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO LRFD) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix E**.

5.2 APPROACH EMBANKMENTS

The new bridge will be shifted approximately 2 feet to the north of the existing alignment and the profile will be widened approximately 8 feet north to accommodate a new sidewalk. The roadway profile will be raised approximately 3.5 feet above existing roadway grade at the west (Abutment 1) approach (approximately Sta. 4+33) and 2.5 feet above grade at the east (Abutment 2) approach (approximately Sta. 6+74), resulting in total embankment heights of approximately 27 feet above adjacent grades. The embankment height tapers quickly east of the Abutment 2 approach. The raise in grade tapers to about 1 foot above existing roadway grade within about 125 feet of each abutment. The widened embankments will extend approximately 10 to 15 feet north of the current embankment limits at the bridge approaches and are planned to have maximum new fill heights up to 10 feet above existing site grades.

Approach embankment side slopes will be constructed with an inclination of 2H:1V or less, except for a portion of slope between Sta. 2+75 and 3+75 where the inclination will be 1.5H:1V to limit impacts outside of the right-of-way, and between the driveway at Sta. 2+70 and 2+75 and 3+75 to 4+00 where the inclination will transition between 2H:1V and 1.5H:1V. The over-steepened slopes are proposed to have a surface treatment of a Turf Reinforced Mat (TRM).

5.2.1 Embankment Performance Criteria

We understand that the settlement criteria that have been adopted by MaineDOT for this project include a maximum of 2 inches of pavement settlement within 100 feet of the abutment in the first 5 years post-construction, and additional 2 inches in the following 5 years (total 4 inches in the first 10 years).

The basis for acceptable global stability for embankments is specified in AASHTO LRFD Article 11.6.2.3 and is summarized as follows:

- Resistance factor of 0.75 (corresponding to a safety factor of 1.3) for slopes that do not support structures, considered for the cross-section analyses; and
- Resistance factor of 0.65 (corresponding to a safety factor of 1.5) for slopes that support structures, considered for the abutment/profile analyses.

Pseudostatic analyses were also conducted, which are interpreted to indicate a low likelihood of significant slope movement if the factor of safety is greater than 1.

5.2.2 Soil Profile and Properties

The profiles considered for bridge approach embankment design considerations included new engineered fill, existing fill, upper marine sand, marine clay, and lower marine sand overlying bedrock. The strength and compressibility properties of the marine clay deposit are the primary contributors to the performance of the embankments. Development of the design parameters for this deposit is described below.

5.2.2.1 Strength

GZA developed marine clay shear strength profiles using the results of field vane shear testing, CKoUDSS testing, and correlation to CPT side friction measurements. The individual undrained shear strength, S_u, data points/profiles and interpreted design values are plotted versus elevation on **Figure 4** for Abutment 1. The primary profile (bold black) was developed to be consistent with S_u outside the footprint of the embankments (including data from borings BB-FJR-102, -104, and CPT-FJR-101, -102, and -103). Since consolidation of the clay under existing embankment loads is known to have resulted in increased undrained shear strengths there, the interpreted S_u beneath the embankment is based on borings conducted through the existing roadway (including borings BB-FJR-103 and CPT-FJR-104).

Abutment 2 encountered a thin layer of marine clay generally consistent with the crust at Abutment 1. The Abutment 2 shear strength profile is based on BB-FJR-106 through the existing embankment, and BB-FJR-107 and CPT-FJR-105 outside of the existing embankment.

Friction angles for granular soil strata were developed using correlations between corrected SPT N-values and friction angle.

Based on the subsurface data and laboratory and in-situ testing results and our experience with similar deposits in the area, GZA interpreted the strength properties as follows:

DESIGN SOIL PROFILES – ABUTMENTS					
Soil Unit	**Total Unit Weight (pcf)**	**West Approach/ Abutment 1**		**East Approach/ Abutment 2**	
		Effective Friction Angle (deg) / Undrained Shear Strength (psf)	**Estimated Thickness[1,2] (feet)**	**Effective Friction Angle (deg) / Undrained Shear Strength (psf)**	**Estimated Thickness[1,2] (feet)**
Proposed Fill (Common Borrow, Sand, Gravel)	125	34	0 - 3.5	34	0 - 2.5
Proposed Lightweight Fill	20	40	3	--	--
Upper Existing Fill	130	40	16	40	20
Lower Existing Fill	125	36	10	36	4
Upper Marine Sand	120	30	4 - 15	30	4 - 5
Marine Clay (crust)	118	S_u = 1,000 - 450 psf	5 - 9	S_u = 2,000 psf	8 - 11
Marine Clay (under Embankment)	114	S_u = 600 - 1000 psf	32	--	--
Marine Clay (outside Embankment)	114	S_u = 450 - 1000 psf	30 - 32	--	--
Lower Marine Sand	125	32	17 - 26	--	--
Glacial Till	130	38	22	--	--
Depth to Bedrock below Existing Pavement Grade	--	120		39	

Note:

1. Estimated thicknesses of fill strata are measured from existing approach embankment pavement grades.
2. “--" indicates layer was not encountered in the borings at that location.

5.2.2.2 Compressibility

Based on the subsurface conditions encountered at Abutment 2, the stiff clay crust is considered to have minimal compressibility beneath the planned 2.5-foot fill thickness that will occur under the roadway and the north shoulder fill. Therefore, compressibility evaluations were focused on the Abutment 1 approach.

Eight consolidation tests (six incremental and two CRS) were completed on samples taken at the site beneath the current roadway embankment (two tests) and outside of the current embankment (six tests) where less or minimal fill was placed over original grades.

The maximum past pressure results from the consolidation tests were compared to values derived using the Stress History and Normalized Soil Engineering Properties (SHANSEP) concept based on the design undrained shear strength profiles in **Figure 4**, utilizing a ratio of undrained shear strength to effective stress (S_u/P) of 0.23 for normally consolidated clay, which was estimated from comparison of laboratory strength and consolidation test results that indicated normally consolidated to lightly overconsolidated clay. The preconsolidation profiles are generally consistent with the laboratory test data and the SHANSEP-based values outside of the existing embankment footprint.

We interpreted the combined results to show that the marine clay crust layer is overconsolidated and will remain so under additional embankment loads. The underlying marine clay at the Abutment 1 approach is shown to be lightly to moderately overconsolidated (by approximately 500 to 1,000 psf) outside the current embankment footprint and approximately normally consolidated under the present roadway embankment. We anticipate that the conditions interpreted outside of the current embankment footprint would apply to the entire site prior to the circa-1957 construction.

Future settlement of overconsolidated deposits is anticipated to occur rapidly and be of moderate magnitude. Normally consolidated deposits tend to be more compressible and the settlement is expected to occur more slowly.

Based on the laboratory and in-situ testing results and our experience with similar Presumpscot clay deposits in the area, GZA interpreted the index and compressibility properties as follows:

SOIL PROPERTIES FOR SETTLEMENT ANALYSES			
Material Property	**Soil Layers**		
	Marine Clay Crust	**Upper Marine Clay (AB 1)**	**Lower Marine Clay (AB 1)**
Modified Recompression Ratio (RR)	0.02	0.02	0.02
Modified Compression Ratio (CR)	0.24	0.24	0.24
Consolidation Coefficient (Cv)	0.15 ft^2/day	0.15 ft^2/day	0.15 ft^2/day
Secondary Compression Coefficient (Cαe)	0.002	0.007	0.007
Unit Weight	116 pcf	114 pcf	114 pcf

It is noted that the Cv value selected for design is in the upper range of values interpreted from the laboratory test results. Considering the time-based settlement criteria presented below, a higher Cv results in greater settlement within the 5- and 10-year evaluation periods and therefore is conservative. Considering the granular marine sands above and below the marine clay, GZA considered the marine clay to have double drainage.

5.2.3 Settlement Modelling and Historic Embankment Construction

GZA modeled the settlement at the Abutment 1 approach using Settle3™, starting with a relatively flat pre-construction ground surface at approximately El. 24. Initial embankment construction occurred in the mid to late 1950s. GZA utilized historic drawings to estimate the original embankment top elevation and dimensions.

Subsurface stratification and soil properties were developed and utilized as previously described. The stress history profile developed outside of the embankment footprint was utilized as the initial condition for the model, using the values at the approach beneath the embankment as shown on **Figure 4**. This modeling approach in Settle3™ develops the effective stress profile considering differential loading at the time of the proposed construction. Where necessary, the subsurface conditions beyond the limits of the test borings were extrapolated to provide a complete subsurface model in the areas of interest for Settle 3™.

Our results estimate approximately 16 to 24 inches of settlement would have occurred under the existing embankments, which is generally consistent with the pavement shim thickness estimated from the borings and an original design thickness of 3 inches.

5.2.4 Analysis of Future Settlement

As previously noted, the design intent is to limit post-construction settlement to 2 inches or less over each successive five-year period for the first 10 years. The currently proposed raised and widened embankment was modeled as being constructed approximately 63 years after initial embankment construction, with final paving being completed 6 months after fill placement. Therefore, the initial five-year settlement values correspond to 6 to 66 months after completion of fill placement. GZA developed the three-dimensional model for analysis of future settlement at the Abutment 1 approach using proposed grading supplied by HTA.

GZA evaluated two alternatives for the Abutment 1 approach including no-mitigation (base analysis), and lightweight fill mitigation. These two mitigation schemes are described in more detail in the following section. If final pavement is placed within 6 months after fill placement, additional primary consolidation settlement should be anticipated.

5.2.4.1 Settlement Mitigation Alternatives

No Mitigation

GZA evaluated a base model where filling is completed to design grade without lightweight materials, vertical drains or surcharge. The results indicate estimated total settlement (combined primary and secondary settlement) ranging from approximately 1.5 to 3.5 inches within 5 years at the approaches and additional 1 to 1.5 inches in the following 5 years. During the 75-year life of the bridge we anticipate up to approximately 5 to 8 inches of total settlement.

The No Mitigation results indicate that predicted settlements are beyond the criteria by approximately 0.5 to 1.5 inches in the first five years. Therefore, the lightweight fill scheme was evaluated.

Lightweight Fill

GZA evaluated ultra-lightweight foamed glass aggregate (ULFGA) material placed beneath the roadway section within 100 feet of Abutment 1 to limit stress increase and associated settlement. The thickness of the approximately 20-pounds-per-cubic-foot material was selected such that there would be a slight stress increase, generally equivalent to about 1 foot or less of standard-weight fill, beneath the paved portion of the roadway, and a reduced but still significant stress increase beneath the widened portion of the embankment, which is generally beneath the sidewalk. Based on our Settle3™ analyses, we predict post-construction settlements of approximately 0.5 to 1.5 inches in the first 5 years and an additional 1 to 1.5 inches in the following 5 years within the limits of the roadway, which are well within the settlement criteria provided by MaineDOT. Therefore, the use of lightweight fill is considered the preferred alternative for settlement mitigation. During the 75-year life of the bridge we anticipate up to approximately 4 to 5 inches of total settlement.

The calculations included in **Appendix E** provide additional details and plotted results of the settlement analyses.

5.2.5 Embankment Slope Stability

GZA evaluated the stability of both approaches at critical cross-sections near the proposed abutments and in the longitudinal direction beneath each abutment using the cross-sections and properties shown in **Appendix E**.

Evaluations were conducted using the computer analytical software Slope/W 2020 (Slope/W), developed by Geo-Slope International, based on the Modified Bishop method. A grid and radius search technique was used to identify the slip surface with the lowest factor of safety. A 250-psf surcharge load was also included within the limits of the proposed travelway. Slope/W output figures showing the minimum factor of safety for each analysis are presented in **Appendix E**. The plotted contours above the slope indicate relative factors of safety associated with center points of the analyzed circular surfaces. Additional details of the analyses and results are presented below. The strength profiles presented in **Section 5.2.2** were used for these analyses.

5.2.5.1 Longitudinal Profiles

Static Analysis

The analyzed profiles considered the interpreted typical subsurface conditions along the project baseline, including the proposed presence of ULFGA. The beneficial reinforcing effect of the proposed HP 14x117 piles supporting the abutments was excluded from the Slope/W model.

The results indicate a minimum factor of safety of 1.8 for the Abutment 1 profile and 1.6 for the Abutment 2 profile. Therefore, the global stability is acceptable in the longitudinal direction.

5.2.5.2 Cross-Sections

Static Analysis

Considering the relatively thin soil profiles and high strength soils at Abutment 2, global stability is considered suitable by observation and was not analyzed in Slope/W.

The analyzed profiles considered the typical subsurface conditions at the Abutment 1 approach locations as described herein. The two analyzed sections are at Sta. 3+50 and 4+00, representing conditions with varying embankment heights. Existing-condition models were evaluated at both sections to consider current embankment factors of safety in the Left-to-Right (L-R) direction, primarily for comparison to and for benchmarking the finite element modeling results. A Slope/W model was developed for the proposed 22-foot-wide travelway with a 250-psf traffic surcharge pressure placed over the travel lanes and side slopes generally of 2H:1V. The R-L section at 3+50 considers the proposed steepened 1.5H:1V slope that is protected by TRM surface treatment. Type D subbase gravel was modeled beneath the TRM. The evaluations are summarized as follows:

- *Sta. 3+50:* 25-foot-high embankment, widened to the left (north) and a maximum 2.5-foot grade raise, 1.5H:1V with TRM surface treatment on northern slope. L-R and R-L analyses. Proposed ULFGA section included in model. Presence of existing brook considered in L-R model.
- *Sta. 4+00, Abutment 1 Approach:* 25-foot-high embankment, widened to the left (north) and a maximum 3.5-foot grade raise. L-R and R-L analyses. Proposed ULFGA section included in model.

The results are summarized in the table below.

SUMMARY OF GLOBAL STABILITY EVALUATION	
Analysis Case	**Minimum Factor of Safety (Static)**
Sta. 3+50, Right-to-Left	1.5
Sta. 3+50, Left-to-Right	1.0
Sta. 4+00, Right-to-Left	1.2
Sta. 4+00, Left-to-Right	1.1

Aside from Sta. 3+50 R-L, the results show that the calculated factor of safety against rotational failure is less than 1.3 and therefore not acceptable for the planned embankment sections. Therefore, GZA evaluated stability mitigation alternatives.

Potential mitigation alternatives to improve stability include flattening the permanent slope inclination, utilizing additional lightweight fill to reduce the driving force, constructing a stabilizing toe berm to increase the resisting force, and driving piles to provide additional shear resistance through the potential slip surface. In general, a toe berm is the lowest-cost stability mitigation alternative and is preferred when space and logistical considerations allow. On the northern slope nearest to I-295, sufficient space is available, and a toe berm was selected as the preferred alternative at Sta. 4+00 R-L. On the southern slope, the presence of the Portland Water District right-of-way and the existing brook make a toe berm and slope flattening unfeasible for either L-R section. Flattening the slope is also not feasible. The very steep brook side slopes appear to have experienced progressive sloughing which is likely to continue and could threaten stability of the roadway embankment in the future. Therefore, pile reinforcement which can mitigate deep rotational circles is the preferred alternative on the southern slope, for both Sta. 3+50 and 4+00 L-R.

Station 3+50 and 4+00 Left-to-Right

Stability mitigation using piles was evaluated using Plaxis2D™ finite element software. The Brom's method has been used to evaluate the soil-pile interaction on past projects, but that method requires evaluation of a "short-pile" or "long-pile" model, which is empirically based on the depth of embedment into supporting soils below the slip surface. Finite element modeling was considered to directly evaluate the safety factor provided by varying pile lengths with the intent of limiting the length of reinforcing piles and associated cost.

A c-phi reduction analysis was completed in Plaxis2D™, in which all soil strengths are reduced at a uniform ratio until failure occurs. The inverse of the soil strength reduction ratio corresponds to a factor of safety against failure. GZA evaluated continuous sheet pile walls at both stations. Discrete HP14x89 piles driven at various spacing were also evaluated at Sta. 4+00. H-Piles are not appropriate from Sta. 3+00 to 3+75 due to the potential for scour adjacent to the brook.

The Plaxis2D™ results indicate that a sheet pile with a section modulus of approximately 62 in^3/ft can achieve the required factor of safety of 1.3 with 15 feet of embedment into the marine sand underlying the clay, which corresponds to a pile length of 70 feet below existing grade. In order to achieve the required factor of safety with the discrete HP14x89 piles, the spacing would have to be reduced to 2 feet on center. GZA compared the average section area per foot of slope and found that the sheet pile walls utilize approximately 4.7 in^2/foot compared to 13.1 in^2/ft of the 2-foot-spaced H-piles. Therefore, the preferred alternative for global stability mitigation is a permanent sheet pile wall.

We did not evaluate the ability of the sheet pile wall to support the embankment slope if the brook bank is scoured to the extent that the brook side of the sheets becomes exposed and the sheet piles act as a retaining wall. In our opinion, the potential for this degree of erosion to occur is low and would take a number of extreme storm events to manifest.

Station 4+00 Right-to-Left

GZA evaluated the proposed slope by adding a toe berm to the north of the embankment, ultimately extending the limits of the existing toe berm constructed with the original embankment. We evaluated a toe berm geometry that matches the existing ground surface contour at approximately 130 feet left of the baseline, just south of an existing marshy area, then rises at a 2H:1V slope until reaching El. 35, then remains flat until reaching the base of the widened highway embankment slope (approximately 50 feet left). The proposed toe berm meets the criteria for static condition for global stability with a factor of safety of 1.3.

Seismic Analysis

A pseudostatic analysis was conducted at Sta. 4+00 R-L, with the recommended toe berm, including earthquake loading based on the Newmark method by incorporating a horizontal seismic coefficient, k_h, defined as half of the maximum peak ground acceleration at the ground surface (0.105 g). This value is justified by the likely development of some degree of deformation of the overall soil mass above the slip surface, in accordance with AASHTO LRFD Article 11.6.5.2.2. For this analysis, the stability is evaluated for the slip surface that is shown to have the lowest safety factor in the static analysis. The calculated pseudostatic factor of safety against rotational failure is predicted to be approximately 1.0 for the critical slip surface beneath Abutment 1 approach which corresponds to the lowest static factor of safety for the project, indicating that significant slope deformations are not likely.

5.3 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments), marine sand, marine clay and underlying sand and glacial till overlying bedrock. Seismic site class was determined in general accordance with AASHTO LRFD Table C3.10.3.1, considering the measured shear wave velocities in the sCPTs, which ranged from 553 to 684 feet per second. These averages consider the soil profile only, which ranged from 50 to 105 feet deep at the sCPTs. Site Class E is defined as an average shear wave velocity below 600 feet per second. Based on the shear wave velocity criterion, the bridge should be assigned to Site Class E.

The available subsurface data indicates that the natural materials encountered at the site are sufficiently cohesive or dense that the potential for liquefaction is low.

5.4 EVALUATION OF FOUNDATIONS

5.4.1 Foundation Type Assessment

H-piles driven to refusal on or near bedrock are the preferred foundation type for the proposed integral abutments and central pier. Probable bedrock was encountered at depths of approximately 115, 30, and 50 feet below the bottom of abutment/pile cap level at Abutments 1 and 2 and the central pier, respectively.

5.4.2 Pile Design Considerations

Based on our experience within similar soils, we anticipate that the proposed H-piles will be driven to refusal on or near the top of rock to achieve the required axial geotechnical resistance. The soil profile includes a significant thickness of medium stiff marine clay that is sensitive and will lose most of its strength temporarily during pile driving, underlain by medium dense fine sand which will also have limited friction resistance during driving. Therefore, we estimate friction resistance on the order of 5 to 20 percent of the required nominal resistance during driving.

Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile loading is not anticipated on the piles.

By utilizing steel H-piles for support of the abutments and pier, initial total and differential settlement will be limited to elastic compression of the piles and should be approximately ½ inch. Additional settlement on the order of ½ inch or less may occur over time due to downdrag loading.

5.4.3 Load and Resistance Factors

Piles should be designed for the controlling nominal resistance of the piles, which could be structural or geotechnical. In GZA's experience with piles gaining a significant portion of their geotechnical resistance in very dense soil or on bedrock which are verified using dynamic pile analysis methods, the drivability resistance generally controls the geotechnical and nominal pile resistance. For this condition, the piles will be driven to a nominal resistance calculated by dividing the maximum factored pile load (Strength load case) by a resistance factor of 0.65, per AASHTO LRFD Table 10.5.5.2.3-1. Resistance factors for service and extreme limit state design should be taken as 1.0. A load factor of 1.0 is recommended for downdrag loads for strength limit state design based on local practice.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor ϕ_c=0.50, per AASHTO LRFD Article 10.7.3.2.3 for hard driving condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per AASHTO LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.70 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), live load surcharge (LS) loads, and components and attachments (DC) loads using the load factors for permanent loads (γ_p) provided in AASHTO LRFD Table 3.4.1-2 for strength limit state foundation design.

5.4.4 Pile Type

The abutments are planned to be supported on ASTM A572, Grade 50 (fy=50 kips per square inch [ksi]) steel HP14x117 piles, oriented for weak-axis bending parallel to the abutments. Each abutment will include five piles. The central pier will be supported by 18 HP14x89 piles, comprised of three rows of six plumb piles, oriented with the strong-axis parallel to the length of the pier, which is skewed approximately 12 degrees from the longitudinal axis of the bridge. The piles will be spaced 3-feet on-center longitudinally and 6-feet on-center laterally.

5.4.5 Downdrag

As discussed previously, the proposed settlement mitigation scheme will not mitigate all of the settlement over the 75-year design life of the bridge relative to the abutment piles. Our evaluations indicate approximately 5.4 inches of settlement could occur at the proposed pile locations over the bridge design life. Based on the subsurface stratification and estimated settlement, we initially estimated an unfactored downdrag load using the β-method in accordance with NAVFAC DM 7.2-211, and as recommended by Sandford et al, “Bitumen Coatings Reduce Downdrag on Piles for Route 1 Interchange Bridges.” The calculated downdrag load from this conventional static method was approximately 169 kips which resulted in a required nominal resistance greater than the structural resistance of the proposed HP14x117 section. In order to complete a more detailed evaluation, GZA opted to use the finite element method (FEM).

GZA used FEM (Plaxis2D™) to evaluate downdrag using the methodology developed by Fellenius[3] to calculate downdrag load applied to the pile by settling soil around its perimeter using FEM considering the neutral plane concept. The neutral plane occurs along a deep foundation element at the depth where soil and pile settlements are equal, resulting in zero mobilized side resistance at that location. Above the neutral plane, settlement of the surrounding soil develops negative side resistance and transfer of downdrag axial compression load to the pile.

The primary difference between the FEM approach and the conventional β-method approach is that the FEM method is able to consider load displacement behavior, and only generates downdrag consistent with the level of strain between the pile and the soil. In this case it shows that the settlement in the lower portion of the clay layer does not develop enough strain to develop the full shear strength of the clay, resulting in a smaller estimated downdrag load. The details of the modelling process are described below.

GZA developed an axisymmetric model in Plaxis2D™ with the design soil profile described previously for Abutment 1. In the axisymmetric model, the model space corresponds to one quadrant of the actual geometry. The axisymmetric model in Plaxis2D™ cannot effectively model the complex shape of an H-pile. Therefore, a pile equivalent to an HP14x117 was developed using a perimeter length equal to an HP14x117 perimeter and a cylindrical beam element with a reduced elastic modulus (E) so that the product of pile area and modulus (EA) would be equal to an HP14x117 pile, which is required for representative elastic pile deformation behavior in the model. The soil-pile interface utilized Mohr-Coulomb soil properties that would allow development of

[3] Tan, S. A., &; Fellenius, B. H. (2016). Negative skin friction pile concepts with soil–structure interaction. Geotechnical Research, 3(4), 137–147. https://doi.org/10.1680/jgere.16.00006

typical ultimate shear strength values with sufficient shear displacement, which is consistent with assumptions when calculating nominal geotechnical resistance.

The Plaxis2D™ soil constitutive model did not include secondary compression, which is the primary cause of downdrag for the project. Therefore, the values of CR and RR were increased until the model estimated 5.4 inches of long-term settlement due to fill loads. This produced appropriate soil-pile differential movement and neutral plane location to develop downdrag under an applied load increase of 350 psf, which was the stress increase calculated in the marine clay stratum in Settle3.

The Plaxis2D™ modeling steps were as follows:

1. No-pile model:
    a. Apply existing fill material (from top of pile to existing grade) as 888 psf of surcharge load on the pile top elevation.
    b. Add additional 350-psf surcharge pressure, equivalent to the stress increase in the marine clay from Settle3 analyses.
    c. Modify CR/RR values in Plaxis2D™ to closely match total clay settlement over the bridge design life.
2. Pile model:
    a. Apply existing fill material (from pile top elevation to existing ground surface) as 888 psf of surcharge load on the ground surface at the pile top elevation.
    b. Add pile element to the model and apply the maximum factored load (410 kips) to the pile beam element. Reset displacement in the model.
    c. Add 350 psf of additional surcharge load to represent the new fill.
    d. Use load distribution in the pile from the modeling results to calculate downdrag.

The results indicate that the maximum load in the pile, occurring at the neutral plane, is approximately 539 kips. The recommended downdrag load considered for pile design is equal to the difference between the maximum load in the pile and the maximum factored pile load of 410 kips, equal to 539 kips. Based on past practice, a load factor of 1.0 was applied to the calculated downdrag resistance, which results in a maximum factored load equal to the maximum load calculated in Plaxis2D™.

Considering the more heavily overconsolidated and less compressible properties of the marine clay crust at Abutment 2, settlement is not anticipated to be large enough to induce downdrag loading on the Abutment 2 piles.

5.4.6 Pile Loads

Hoyle, Tanner provided a maximum factored axial load for abutment piles of 410 kips per pile for the strength condition, and the factored downdrag load at Abutment 1 is 129 kips; therefore, piles should be installed to a nominal axial resistance of at least 829 kips, calculated by dividing the maximum factored axial load of 539 kips by a geotechnical resistance factor of 0.65. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted on one pile during construction in accordance with AASHTO LRFD requirements to assess nominal geotechnical pile resistance. It is noted that there is no downdrag anticipated at Abutment 2, but for simplicity of construction, the piles at both abutments will be installed to achieve the same nominal resistance.

Hoyle, Tanner provided a maximum factored loads and moments applied at the center of the pile cap at the pier and the proposed pile and cap configuration for GZA's use in the pile group analyses. The maximum factored axial load per pile was calculated from these analyses, as presented in **Section 5.4.7**.

5.4.7 Lateral Pile Analysis

Abutments

GZA conducted lateral pile analyses using LPILE 2019® (LPILE®). Hoyle, Tanner provide a maximum thermal deflection of 1.375 inches and a factored axial load of 410 kips. Downdrag was not included in the axial load as it will act below the pile head and will not contribute to the maximum pile stress. We assumed a fixed-head condition (zero rotation) and imposed the estimated thermal deflection at the pile head. The ground surface was assumed at the bottom of abutment/top of pile for thermal contraction. The analysis used ASTM A572 Grade 50, HP 14x117 piles aligned for weak-axis bending parallel to the bridge centerline. GZA's lateral pile analysis for the integral abutments was conducted in accordance with Section 5.4.2.4 of the MaineDOT BDG and the recommendations included in the "Integral Abutment Bridge Design Guidelines" by the Vermont Department of Transportation (VTrans).

GZA developed the soil profile tabulated below based on the soil conditions encountered in the test borings and laboratory testing results. Abutment 2 piles are shorter in length, therefore GZA considers the Abutment 2 the controlling profile and is considered representative for both abutments.

L-PILE® INPUT PARAMETERS Abutment 2, Typical Shortest Pile Length Approx. 30 ft						
Stratum	**Soil Model**	**Top of Layer Elevation (ft-NAVD 88)**	**Layer Thickness (ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_t, γ_e (pcf)**
Upper Existing Fill	Reese Sand	53	13.9	250	40	130
Upper Marine Sand**	Reese Sand	39.1	4.5	35	30	58
Marine Clay Crust	Stiff Clay w/o free water	34.6	11.6	E_{50} = 0.007	1,000	53
Top of Rock	--	23	--	--	--	--

Notes:
1. Soil strata were modelled after boring BB-FJR-106.
2. ** indicates the top of layer is the approximate ground water elevation based on the boring logs and/or CPTs.
3. pci = pounds per cubic inch, deg = degrees, psf = pounds per square foot, γ_t = total unit weight (used above anticipated groundwater level), γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per square foot.

The initial lateral pile analysis resulted in a combined axial and bending stress of 84.7 ksi, which is beyond the elastic range of the pile. Therefore, GZA conducted a plastic hinge analysis.

The initial analysis using VTrans methodology considered pile head boundary conditions including a thermal deflection of 1.375 inches and a fixed-head condition (zero slope condition). The plastic moment resistance of the pile was calculated using the comparison of factored axial pile load and resistance, unbraced lengths, and effective length factors resulting from the initial LPILE® analysis. The structural capacity of the pile was checked to determine if a plastic hinge formed for the specified displacement and specified axial thrust load under combined flexure based on the provisions of Article 6 of AASHTO LRFD and as per the recommendations of Section 5.4.2.4 of the MaineDOT BDG. This involves comparing the ratios of nominal and maximum factored axial load and moment in the pile using the interaction equation presented in AASHTO LRFD 6.9.2.2. This calculation indicates that the combined stress resulting from an axial load of 410 kips and calculated pile head moment of 361 ft-kip results in formation of a plastic hinge.

The hinge allows the pile head to rotate with a constant moment (i.e., the plastic moment). The pile head transforms from a fixed connection to a pinned connection, thereby changing the effective length and design end condition parameter (K-value) of the top segment for stability checks in accordance with AASHTO LRFD Table C4.6.2.5-1. The structural resistance of the top segment of the pile is enhanced since the plastic hinge condition is anticipated to occur.

Since the resulting moment from the first LPILE® iteration exceeded the calculated plastic hinge moment, a second LPILE® analysis was performed using the original specified pile head displacement and the plastic moment. This change results in a reduction of the axial buckling resistance of the upper segment of the pile (Segment 1) where the plastic hinge occurs.

The results of the second analysis indicates that the axial structural resistance including bending is adequate, and the demand ratio for combined bending is less than 1.0 in Segment 2. Therefore, with the exception of the plastic hinge location within the upper 1 foot of the pile below the abutment, the pile remains within the elastic range over the remainder of its length and is stable against buckling. The results also indicate that the nominal structural resistance in Segment 1 is adequate to support the design loads. Therefore, the proposed HP14x117 pile type and weak-axis configuration is suitable for support of the integral abutment deformation and axial loads. The integral abutment pile calculations are included in **Appendix E**.

Central Pier

GZA conducted lateral pile group analyses using GROUP 2015® (Group). We developed design subsurface profiles for use in Group that included new fill, marine clay, and marine sand. The soil profiles and recommended design properties are summarized in the following tables, and pier cap geometry, pile sizes, and load combinations provided by Hoyle, Tanner are presented in **Appendix E**. Load combinations were provided for multiple strength and service cases, and Hoyle, Tanner recommended using the largest factored loads for axial and moments applied to the bottom of column from different Strength load combinations. The Group model considered pile lateral resistance reduction factors (i.e., p-multipliers) in the direction of primary loading. The p-multipliers were developed based on the loading direction, pile diameter and pile spacing. These p-multipliers ranged from .88 to 0.6 and are summarized per pile in the results presented in **Appendix E**.

GROUP INPUT PARAMETERS Central Pier						
Stratum	**Soil Model**	**Top of Layer Elevation (ft- NAVD 88)**	**Layer Thickness (ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_e (pcf)**
New Fill**	Reese Sand	29.8	4	45	32	63
Upper Marine Sand	Reese Sand	25.8	2.9	35	30	58
Marine Clay (Crust)	Stiff Clay w/o Free Water	22.9	8.5	E_{50} = 0.007	600 psf	53
Marine Clay	Soft Clay	14.4	30.3	E_{50} = 0.008	450 psf	53
Lower Marine Sand	Reese Sand	-15.9	4	55	32	73
Top of Rock	--	-19.9	--	--	--	--

Notes:
1. Recommended modulus and unit weight values assume groundwater level at El. 30.0.
2. pci = pounds per cubic inch, deg = degrees, psf = pounds per square foot,
3. γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per square foot.
4. Top of pile/Bottom of pile cap at EL. 25.8, Fill soils modeled from bottom of cap to EL. 29.8 (top of Cap).
5. ** indicates the top of layer is the approximate ground water elevation based on the boring logs.

Ultimate unit side friction, unit tip resistance, axial-load-displacement curves and torque-rotation curves were automatically generated by Group using the soil models above for the analysis.

GZA conducted analyses for the pier using pile group configurations developed by Hoyle, Tanner, which were iteratively modified based in interim Group analyses. The pier piles will consist of ASTM A572, Grade 50 (fy=50 ksi) HP14x89 steel H-piles. GZA's final analysis geometry includes three rows of six plumb piles (18 total). GZA provided the analysis results to Hoyle, Tanner who evaluated the modelled foundation performance as part of the overall structural design and found it to be adequate.

The pile stresses and loads from the Group results indicate a maximum combined stress of 17.9 ksi and a maximum factored axial pile demand of 358 kips. The maximum stresses in the table are combined stresses including axial and bending components. Pile cap movement is predicted to be approximately 0.1 inches vertical (down), 0.2 inches laterally in the longitudinal direction, and 0.02 inches laterally in the transverse direction. The results show that piles will not be in tension under the applied loads. The required nominal pile resistance is 552 kips, calculated by dividing the maximum factored axial load by the driven pile resistance factor of 0.65 for the strength condition. The results indicate that the pier piles are capable of achieving fixity and remaining elastic under the proposed loads.

5.4.8 Design-Phase Pile Drivability Analysis

GZA completed wave equation analyses to assess the drivability of the HP 14x117 (abutments) and HP 14x89 (pier) pile with nominal geotechnical resistances of 829 and 552 kips, respectively. The goal of the analysis was to evaluate the range of rated energy necessary to install the piles to a nominal resistance without exceeding the allowable driving stress.

The analyzed pile lengths were selected assuming that the piles would be driven to the top of rock elevations encountered at the abutments and pier. Estimated side resistance during pile installation was used as an input in wave equation analyses, including a range of 5 to 20 percent contribution of skin friction resistance to the maximum factored pile resistance. Analyses for the Abutment 1 piles were completed using a Delmag D46-32 diesel hammer with a ram weight of 10,140 pounds and a rated energy of approximately 126,750 foot-pounds (ft-lbs) operating at the maximum fuel setting (Fuel Setting 1). Analyses for the Abutment 2 and pier piles were completed using a Delmag D25-32 diesel hammer with a ram weight of 5,500 pounds and a rated energy of approximately 58,245 foot-pounds (ft-lbs) operating at Fuel Setting 4 (minimum). The results are summarized below.

DESIGN-PHASE WAVE EQUATION ANALYSIS RESULTS					
Pile Location and Type	**Embedded Pile Length**	**Driving System Rated Energy (ft-lbs)**	**Required Nominal Geotechnical Resistance (kips)**	**Max Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
Abutment 1 HP 14x117	115 feet	126,750	829	40	11
Abutment 2 HP 14x117	30 feet	58,245	829	40	14
Pier HP 14x89	48 feet	58,245	552	38	8

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer system type and energy are judged acceptable to install the piles to the required nominal resistance noted. A number of commonly available diesel pile hammers are available in the noted rated energy ranges. It should be noted that GZA evaluations were conducted to limit the number of pile hammers for the project and that a smaller hammer could be capable of installing the Abutment 2 and pier piles. Construction-phase drivability analyses should be conducted for the hammer(s) proposed by the contractor.

5.4.9 Lateral Earth Pressures

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil.

Hoyle, Tanner provided a thermal expansion for use in abutment design of 0.78 inches. The abutment height is approximately 11.7 feet resulting in a calculated abutment rotation of 0.0056 feet/foot. Considering the calculated rotation exceeds the threshold of 0.005 for use of a Rankine passive coefficients, GZA recommends using the MassDOT methodology, which results in a passive earth pressure coefficient of 4.15 for design of backwalls and wingwalls. A range of 4.15 to 5.0 is appropriate for a passive earth pressure coefficient for lightweight fill considering the MassDOT Methodology, and the possible higher friction angle, therefore the coefficient that generates the largest load scenario should be used for design. Design lateral earth pressure recommendations were developed based on this equation, as presented in **Appendix E**, and are provided in **Section 6.3** of this report.

AASHTO LRFD Commentary C3.10.9.1 specifies that specifies that bridges in Seismic Zone 1 are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

5.4.10 Frost Protection

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,275, and with low-moisture content (10-20 percent) soils, the estimated depth of frost penetration is approximately 5.75 feet.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN AND SETTLEMENT MITIGATION

We recommend the use of ULFGA to reduce the stress increase in the marine clay resulting from new embankment fills in the vicinity of the new abutments. The basic limits of the ULFGA should extend vertically from the base of the new pavement section, and from the back of the new Abutment 1 to a distance of 100 feet from the back of the abutment. The limits of the ULFGA consistent with GZA's settlement evaluations and predictions are as follows:

Station (Baseline)	**ULFGA Thickness**	
	32' Lt. to 18'	**-18 Lt' to 15' Rt.**
3+32 to 4+32	5'	3'

The thicknesses recommended above should be considered the minimum within the offsets listed, including irregularities in the cross section around guardrails and underdrains.

The ULFGA should be completely encapsulated with non-woven separation geotextile (MaineDOT Standard Specification 722.04). Installation of the ULFGA should be completed in accordance with *Special Provision Section 203, Excavation and Embankment (Ultra-Lightweight Foamed Glass Aggregate)*.

Embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter, except between Sta. 2+75 and 3+75 Left, where the slope angle will be 1.5H:1V. Slopes should be provided with loam and seed for permanent erosion protection. Where slopes steeper than 1.75H:1V are planned, TRM should be employed to promote vegetation growth and limit surface erosion. Beneath the TRM and extending through areas with inclinations steeper than 2H:1V, we recommend utilizing MaineDOT Type D Aggregate for Subbase compacted to 95 percent relative compaction according to AASHTO T-180 (Modified Proctor). The TRM should be underlain by a 2-inch-thick compost blanket (Special Provision 613) and hydraulically seeded with a special seed mix (Special Provision 717.03). The TRM should be installed in accordance with Special Provision 613 and any manufacturer-specific components (e.g., anchoring).

To meet the minimum required factor of safety for global stability at the southern slopes at the Abutment 1 approach (L-R), GZA recommends sheet piles located generally at the toe of slope between Sta. 3+10 to 4+61 with a minimum section modulus of 62 in^3/ft. The recommended sheet pile length is 70 feet. The top of pile should be embedded a minimum of 1-foot below the final grade.

To meet the minimum factor of safety at the Sta. 4+00 northern slope (R-L), a toe berm is required. The toe berm should match the existing ground surface contour at approximately 130 feet Left of the baseline, just north of an existing marshy area, then rise at a 2H:1V slope up to El. 35, then remain flat until reaching the base of the highway embankment slope (approximately 50 feet Left).

6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO design guide maps (3.10.2.1-1 through -21 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectra (Site Class E) for a 7 percent probability of exceedance in 75 years are summarized for the site are as follows:

SITE CLASS E SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	2.5
Fa	2.5
Fv	3.5
As (Period = 0.0 sec)	0.21 g
SDs (Period = 0.2 sec)	0.42 g
SD1 (Period = 1.0 sec)	0.15 g

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil, except in areas where ULFGA is proposed. Recommended soil properties for backfill material are as follows:
  - Internal Friction Angle of Soil (Type 4) = 32°
  - Soil Total Unit Weight (Type 4) = 125 pcf
  - Internal Friction Angle of Soil (ULFGA) = 40°
  - Soil Total Unit Weight (ULFGA) = 20 pcf
  - Coefficient of Passive Earth Pressure, K_p = 4.15 for granular fills and 4.15 – 5.0 for ULFGA (use for design of backwalls and wingwalls)
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with AASHTO LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center. Alternatively,

prefabricated drainage geocomposite material can be placed against the uphill side of abutments, after holes have been created through the backing material at the weep hole locations.

6.4 PILE DESIGN

6.4.1 Abutments

- The proposed abutments may be supported on HP 14x117 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles. Piles should be driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on or near the bedrock surface.
- The abutment piles should be driven to a nominal resistance of 829 kips, calculated by dividing the maximum factored pile load of 539 kips (410 kips Strength I plus 129 kips downdrag) by a resistance factor of 0.65.
- The piles should be oriented for weak-axis bending consistent with the lateral pile analyses presented herein.
- Preliminary wave equation analyses indicate that the Abutment 1 piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 127,000 ft-lbs for the anticipated 115-foot-long, ASTM A572 Grade 50 HP14x117 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance within the MaineDOT range of 6 to 15 blows per inch.
- Preliminary wave equation analyses indicate that the Abutment 2 piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 58,000 ft-lbs for the anticipated 30-foot-long, ASTM A572 Grade 50 HP14x117 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance within the MaineDOT range of 6 to 15 blows per inch.

6.4.2 Pier

- The proposed pier substructure may be supported on HP 14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles. Piles should be driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on or near the bedrock surface.
- The pier piles should be driven to a nominal resistance of 552 kips, calculated by dividing the maximum factored pile load of 358 kips (Strength I) by a resistance factor of 0.65.
- The pier piles should be oriented consistent with the lateral pile analyses presented herein.
- Preliminary wave equation analyses indicate that the pier piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 58,000 ft-lbs for the anticipated 48-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance within the MaineDOT range of 6 to 15 blows per inch.

6.4.3 All Piles

- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. -73 at Abutment 1, approximately El. -20 at Pier 1, and approximately El. 23

at Abutment 2). A provision is recommended in the drawings for extra pile length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into the bedrock.

- To limit pile damage during driving, cast steel pile points should be provided in accordance with *MaineDOT Standard Specification Section 501.12 – Materials and Section 711.10 H-Beam Piles, Splices and Tips.*
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- We recommend that one pile at each abutment and one pier pile be dynamically tested at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a restrike test on each test pile.
- Piles may be installed through the marine clay stratum by vibratory methods. Vibratory installation should be terminated when an increase in resistance is detected, and the pile should be monitored dynamically for throughout the remainder of the driving.
- Piles shall be spliced in accordance with MaineDOT Section 501.047. Considering the predicted formation of a plastic hinge at the base of the abutment, splices should not be allowed within 15 feet of the bottom of the abutment.
- Integral abutment piles should be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per AASHTO LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.7 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1). GZA completed this analysis for the integral abutment piles as documented herein.
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankment to the bridge. The slabs should be positively connected to the backwalls.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection. These items are discussed in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

We recommend that the H-pile installation be controlled using wave equation analysis of the contractor's proposed driving system, field logging of the pile installation, and determination of final penetration resistance based on dynamic pile testing with signal matching analysis. As previously noted, the piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO LRFD Table 10.5.5.2.3-1.

AASHTO LRFD Table 10.5.5.2.3-1 requires that at least one load test with signal matching be performed per substructure in conjunction with the resistance factor of 0.65. We recommended that one PDA test with Signal Matching be completed at each abutment and at a pier pile, including a restrike test at least 24 hours after initial drive for each tested pile, resulting in a total of three PDA tests.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

We anticipate that temporary support of excavation may be necessary to maintain portions of the existing roadway adjacent to the pier. It is anticipated that a sheetpile cofferdam will likely be required for construction of the pier in the I-295 median.

Temporary dewatering is not anticipated to be necessary to control groundwater inflow in excavations at the abutments but is anticipated at the pier excavation. It is anticipated that any inflow of surface water or runoff to excavations can be handled by open pumping from sumps installed at the bottoms of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, six of the seven upper fill samples tested had less than 10 percent passing the No. 200 sieve, indicating a significant proportion of the fill may meet MaineDOT gradation requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill and possibly structural backfill. The material is considered suitable for use as Common Borrow without further testing.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

7.4 ULTRA-LIGHT WEIGHT FILL PLACEMENT

Based on our experience, the ULFGA material can be handled and placed similarly to granular borrow, but a unique aspect that affects construction is that densification must take place in a controlled fashion with careful attention to avoid crushing of the particles. This involves utilization of vibratory plate compaction equipment to place and compact the material in lifts and avoiding direct traffic on the material with heavy construction equipment. We recommend that GZA provide submittal review and field engineering to observe and provide input into material-specific aspects of construction sequencing, placement methodology, compaction equipment and compaction procedures.

We recommend that a nonwoven geotextile fabric be placed as a separator between the ULFGA and all other differing materials above, below or adjacent to the material. This will involve a combination of placing the geotextile along prepared subgrade surfaces, wrapping it up around the edges of placed ULFGA, and wrapping it around irregularities in the surface as required due to the presence of underdrain piping, catch basins, and guardrails.

Equipment travel on placed ULFGA should be strictly prohibited. Construction equipment other than for placement and compaction of ULFGA should not operate on the exposed ULFGA surface until a minimum 12-inch layer of granular cover material is placed over the ULFGA. The cover material should be placed and compacted within 48 hours of placing and compacting the final lift(s) of ULFGA in an area. Until that time, operation of construction equipment directly on the ULFGA shall be limited to light- duty equipment with rubber tires. The contractor should be required to submit a proposed plan for protecting exposed ULFGA.

The ULFGA will be placed in two stages consistent with the planned staged bridge construction. The contractor should be required to submit a description of their methodology to provide a continuous ULFGA fill layer considering phased bridge construction.

The ULFGA should be transported, placed and compacted in accordance with the project-specific Special Provision 203.

TABLES

TABLE 1
Summary of Subsurface Explorations
Johnson Road Bridge, over I-295
Falmouth, ME
WIN 21721.00

Boring ID	Station	Offset (ft)	Ground Surface El. (ft)	Topsoil	Pavement	Top of Stratum Elevation							Stratum Thickness (ft)								Depth to Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
						Fill	Upper Marine Sands	Marine Clay Crust	Marine Clay	Lower Marine Sands	Glacial Till	Bedrock	Top Soil	Pavement	Fill	Upper Marine Sands	Marine Clay Crust	Marine Clay	Lower Marine Sands	Glacial Till				El. (ft)	Depth (ft)
BB-FJR-101	2+48.0	44.1 R	28.9	28.9	NE	28.6	NE	16.9	11.9	-24.1	NE	NE	0.3	NE	11.7	NE	5.0	36.0	**>33**	NE	NE	86.0	-57.1	21.4	7.5
BB-FJR-102*	4+25.7	50.2 L	36.9	36.9	NE	36.4	24.3	14.8	9.3	NE	NE	NE	0.5	NE	12.1	9.5	5.5	**>27.9**	NE	NE	NE	55.5	-18.6	24.6	12.3
BB-FJR-102A*	4+20.4	51.2 L	36.7	NE	NE	36.7	24.3	14.8	9.3	NE	NE	NE	NE	NE	12.4	15.0	NE	**>24.6**	NE	NE	NE	52.0	-15.3	23.7	13.0
BB-FJR-103	4+10.8	4.4 R	47.8	NE	47.8	46.3	21.6	17.8	9.8	-24.2	-50.2	-72.6	NE	1.5	24.7	3.8	8.0	34.0	26.0	22.4	120.4	130.6	-82.8	31.4	16.4
BB-FJR-104	4+53.0	63.4 R	29.5	NE	NE	29.5	21.5	17.5	12.5	-24.5	NE	NE	NE	NE	8.0	4.0	5.0	37.0	17.5	NE	NE	71.5	-42.0	25.9	3.6
BB-FJR-105	5+33.1	6.7 L	33.4	NE	33.4	31.4	28.9	22.9	14.4	-15.9	NE	-19.9	NE	2.0	2.5	6.0	8.5	30.3	4.0	NE	53.3	64.1	-30.7	25.4	8.0
BB-FJR-106	6+77.3	4.5 L	62.6	NE	62.6	61.6	39.1	34.6	NE	NE	NE	23.3	NE	1.0	22.5	4.5	11.3	NE	NE	NE	39.3	49.3	13.3	50.4	12.2
BB-FJR-107*	7+19.6	69.4 R	46.9	NE	NE	46.9	41.0	35.6	32.6	27.6	NE	NE	NE	NE	NE	5.4	3.0	5.0	**>1.5**	NE	NE	20.8	26.1	34.5	12.4
CPT-FJR-101	3+80.8	52.3 L	34.3	NE	NE	34.3	22.8	14.8	8.8	-24.7	NE	NE	NE	NE	11.5	8.0	6.0	33.5	**>0.5**	NE	NE	59.5	-25.2	24.1	10.2
CPT-FJR-102	4+14.3	103.5 L	29	NE	NE	29.0	23.0	14.0	9.0	-22.5	NE	NE	NE	NE	6.0	9.0	5.0	31.5	**>2.3**	NE	NE	53.8	-24.8	24.0	5.0
CPT-FJR-103	4+18.3	46.9 L	36.8	NE	NE	36.8	24.3	14.8	9.3	-21.7	NE	NE	NE	NE	12.5	9.5	5.5	31.0	**>6.6**	NE	NE	65.1	-28.3	24.3	12.5
CPT-FJR-104	4+0.7	3.5 L	47.1	NE	47.1	45.6	25.1	15.1	6.1	-27.9	NE	NE	NE	1.5	20.5	10.0	9.0	34.0	**>30.4**	NE	NE	105.4	-58.3	32.1	15.0
CPT-FJR-105	7+13.8	73.7 R	46.1	NE	NE	46.1	41.0	35.6	32.6	27.6	NE	NE	NE	NE	5.1	5.4	3.0	5.0	**>2.8**	NE	NE	21.3	24.8	39.1	7.0

El. = Elevation, NE = Not Encountered, > = Boring Terminated in Stratum, * = Stratification taken from adjacent CPT

Notes:
1. Refer to the boring logs in Appendix B and CPT reports in Appendix C for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled locations were surveyed by MaineDOT and provided to GZA.
4. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.
5. Borings BB-FJR-102 and -107 were completed By Summit Geoengineering Services.

FIGURES

N
W E
S

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

Data Supplied by :

esri

USGS science for a changing world

USGS QUADRANGLE LOCATION

0 1,000 2,000 4,000 6,000

SCALE IN FEET

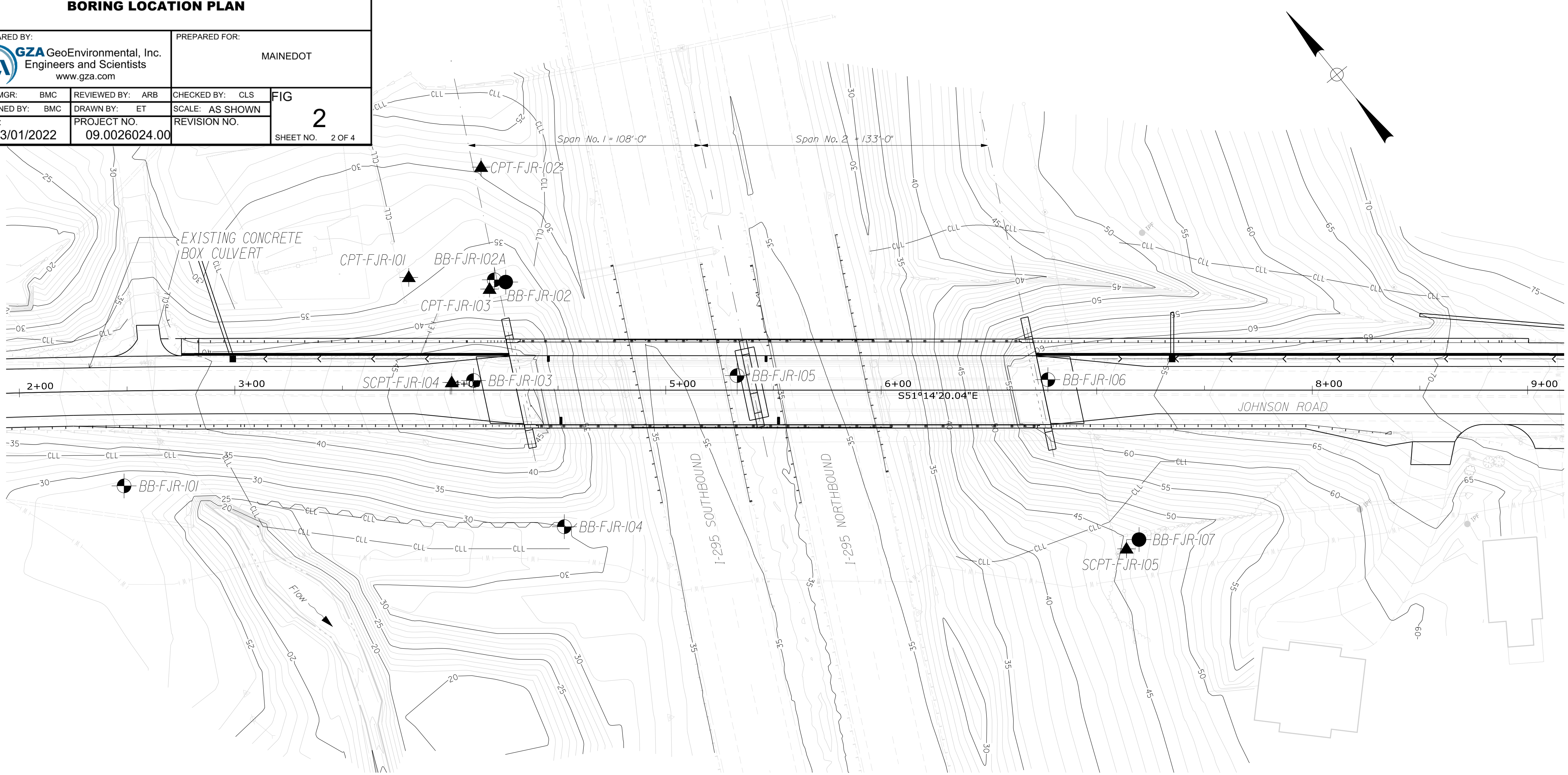
GZA	PROJ. MGR.: BMCD ESIGNED BY: LCN REVIEWED BY: ARB OPERATOR: LCN DATE: 11-15-2021	**LOCUS PLAN** JOHNSON ROAD BRIDGE OVER I-295 FALMOUTH, ME	JOB NO. 09.0026024.00 FIGURE NO. 1

JOHNSON ROAD BRIDGE
MAINEDOT WIN 21721.00
FALMOUTH, ME

BORING LOCATION PLAN

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com	PREPARED FOR: MAINEDOT

PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: CLS	FIG 2
DESIGNED BY: BMC	DRAWN BY: ET	SCALE: AS SHOWN	
DATE: 3/01/2022	PROJECT NO. 09.0026024.00	REVISION NO.	SHEET NO. 2 OF 4



NOTES

1) Base map developed from electronic files (Files included alignments.dgn, Bridge.dgn, Topo.dgn, and contours.dgn) provided by Hoyle, Tanner & Associates, Inc February 28, 2022.

2) The as-drilled boring locations were surveyed by a MaineDOT survey crew and provided to GZA in an electronic file (Borings.dgn).

LEGEND

Symbol	Description
BB-FJR-I06	Indicates borings performed by New England Boring Contractors of Hermon, Maine between June 3 and August 26, 2019 and observed by GZA personnel.
BB-FJR-I07	Indicates borings performed by Summit Geoengineering Services, Inc. of Rockland, Maine between May 28, and May 30, 2019 and observed by GZA personnel.
CPT-FBR-I05	Indicates cone penetration tests (CPTs) performed by Summit Geoengineering Services, Inc. of Rockland, Maine between May 28, and June 3, 2019 ("S" indicates seismic testing was performed).

PLAN

25 0 25 50
Scale

Division: Bridge
Filename: ...\BLP\Johnson BLP(2).dgn

Date: 3/2/2022
Username: BMC

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021721.00

WIN 21721.00

BRIDGE PLANS

BRIDGE NO. 5792

SIGNATURE | P.E. NUMBER | DATE

PROJ. MANAGER | DESIGN-DETAILED | CHECKED-REVIEWED | DESIGN2-DETAILED2 | DESIGN3-DETAILED3 | REVISIONS 1 | REVISIONS 2 | REVISIONS 3 | REVISIONS 4 | FIELD CHANGES

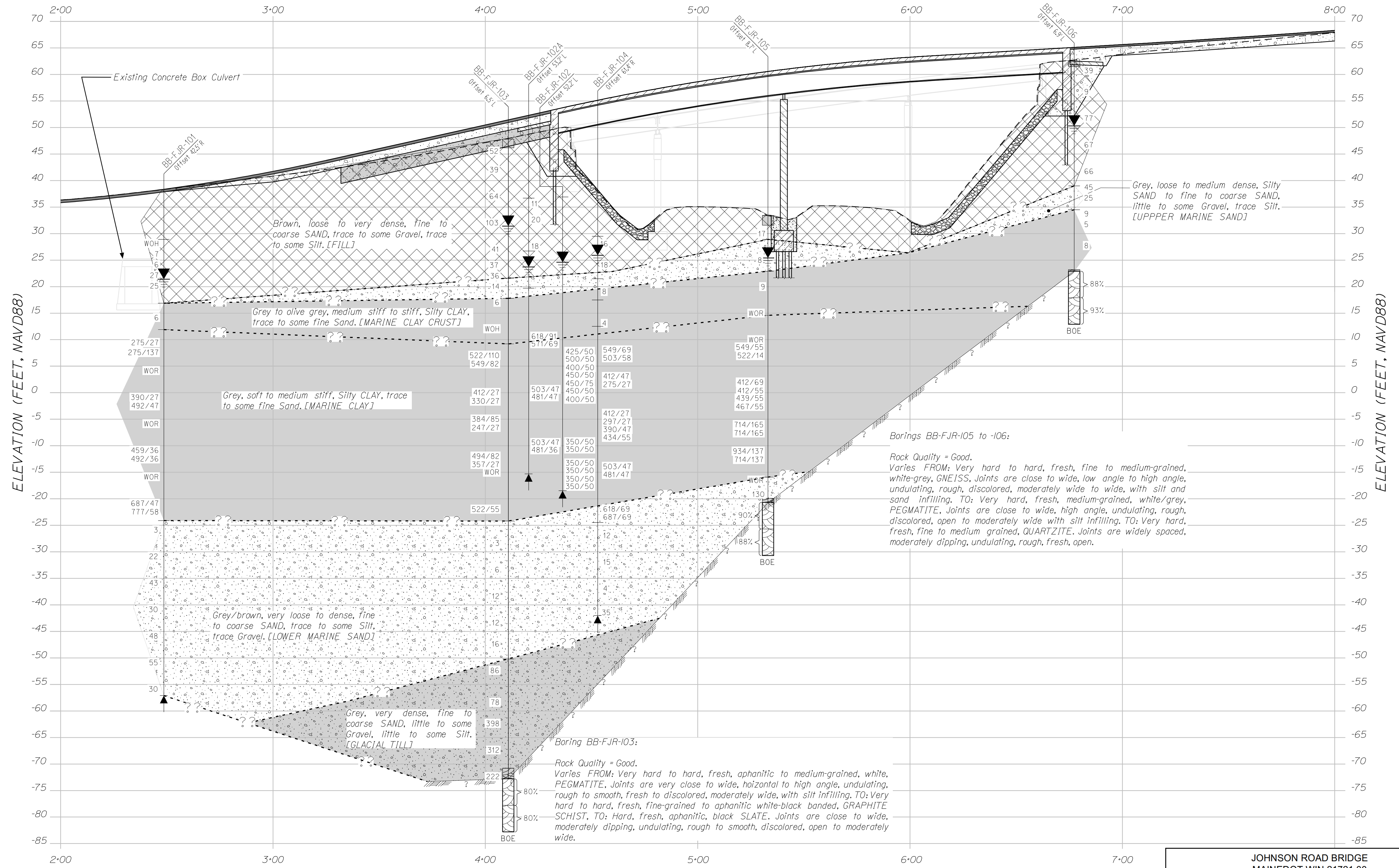
BY | DATE

JOHNSON ROAD BRIDGE
INTERSTATE 295
FALMOUTH CUMBERLAND COUNTY

BORING LOCATION PLAN

PREPARED BY: GZA

SHEET NUMBER
8
OF 58



NOTES

1) Base map developed from electronic files provided by Hoyle, Tanner & Associates, Inc. on January 28, 2022 (File included Profile.dgn)

2) The as-drilled boring locations were surveyed by a MaineDOT survey crew and provided to GZA in an electronic file (Borings.dgn).

3) BB-FJR-100 series bridge borings were performed by New England Boring Contractors and Summit Geoengineering Services, Inc and observed by GZA personnel between May 28 and August 26, 2019.

4) This generalized interpretive soil and rock 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 transitions may vary and are probably more erratic. Boring data are shown for borings drilled off alignment, but interpreted strata are based on the three borings drilled closest to the project baseline. For more specific information refer to the exploration logs.

LEGEND

- Boring No., Offset, if shown
- Pavement Thickness if applicable
- 6 — Energy-Corrected SPT N60 Value (blows/foot)
- WOH — Indicates weight of hammer
- WOR — Indicates weight of rod
- Encountered Groundwater Level
- Strata Interface
- 879/137 — Indicates in-situ field vane strength (peak/residual) in pounds per square foot.
- R — Split Spoon Refusal(>50 blows for 1" penetration)
- Advancement of rollerbit prior to coring
- Interpreted Top of Bedrock (See Note 4)
- RQD = Rock Quality Designation for Rock Core Sample
- BOE — Bottom of Exploration
- Bottom of Exploration, No Refusal

Horiz. 25 0 25 50
Vert. 10 0 10 20
Scale of Feet

JOHNSON ROAD BRIDGE
MAINEDOT WIN 21721.00
FALMOUTH, ME

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINEDOT	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG
DESIGNED BY: ENT	DRAWN BY: ENT	SCALE: AS SHOWN	3
DATE: 2/1/2022	PROJECT NO. 09.0026024.00	REVISION NO. 0	SHEET NO. 3 OF 4

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021721.00

WIN 21721.00

BRIDGE PLANS

Bridge No. 5792

SIGNATURE | P.E. NUMBER | DATE

PROJ. MANAGER | BY | DATE
DESIGN-DETAILED
CHECKED-REVIEWED
DESIGN2-DETAILED2
DESIGN3-DETAILED3
REVISIONS 1
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES

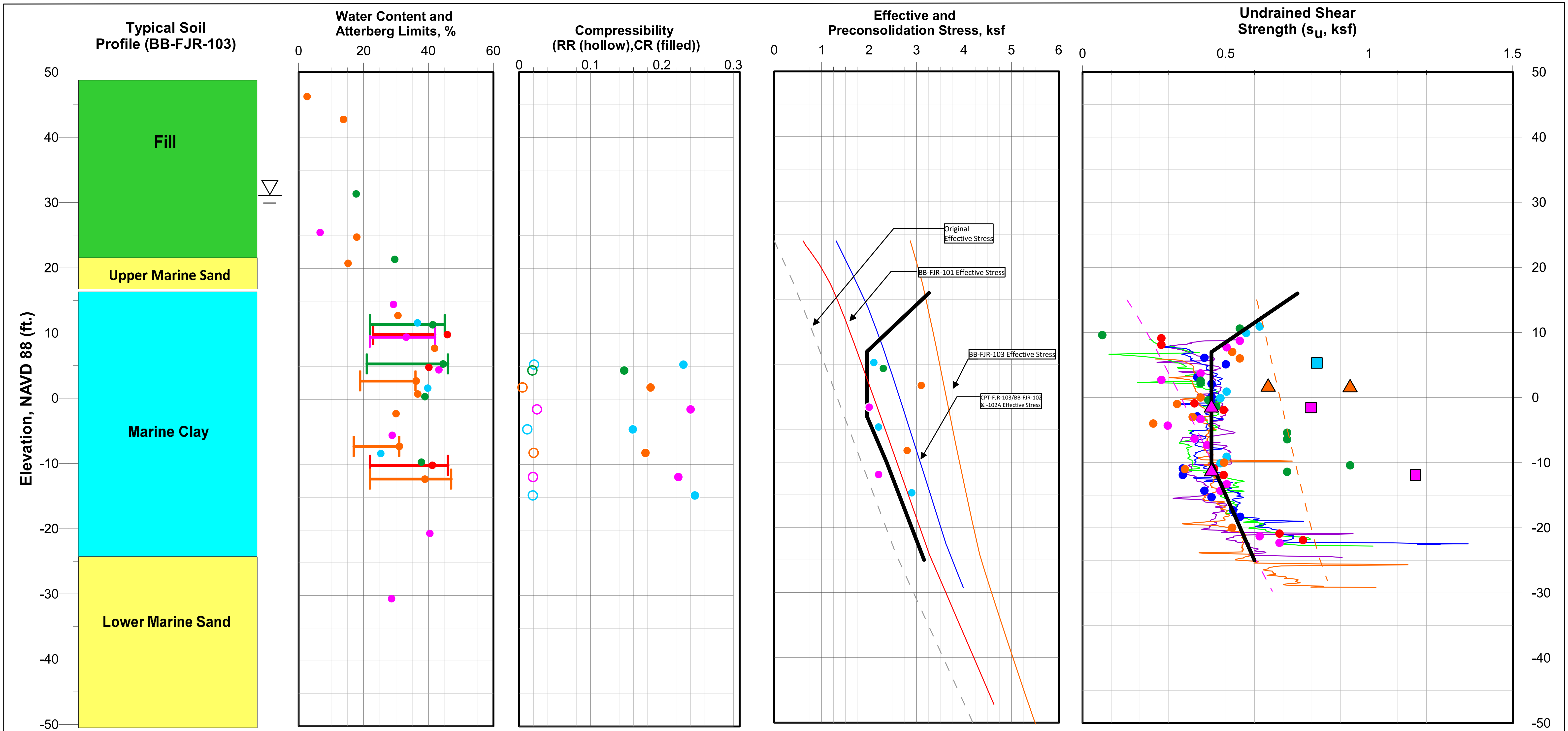
JOHNSON ROAD BRIDGE
INTERSTATE 295
FALMOUTH CUMBERLAND COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
9
OF 58

Division: Bridge
Filename: ...\ISP\Johnson_ISP(2).dgn

Date:1/31/2022
Username: BMC



NOTES:

1. DATA BASED ON TEST BORINGS (BB-FJR-101, -102A -103, -104, -105 AND -106) PERFORMED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE BETWEEN JUNE 3 AND AUGUST 21, 2019 AND TEST BORING BB-FJR-101 AND -107 PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE BETWEEN MAY 28 AND MAY 30, 2019. BORINGS PERFORMED BY NEW ENGLAND BORING CONTRACOTRS WERE OBSERVED AND LOGGED BY GZA PERSONNEL.
2. CPT EXPLORATIONS (CPT-FJR-101, -102, -103, -104 AND -105) PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE BETWEEN MAY 28 AND 30, 2019.
3. TYPICAL SOIL PROFILE BASED ON BORING BB-FJR-103.
4. WATER CONTENTS BASED ON LABORATORY TESTS PERFORMED ON SAMPLES TAKEN FROM RECENT BORINGS.
5. EFFECTIVE STRESS BASED ON INITIAL EFFECTIVE STRESS CALCULATED BY SETTLE3D BY ROCSCIENCE. EXISTING EMBANKMENT (FILL) MODELED AS AN EMBANKMENT LOAD OVER ORIGINAL GRADE. THEREFORE, EFFECTIVE STRESS ONLY CALCULATED BELOW EL. 18'.
6. PRECONSOLIDATION PRESSURE CALCULATED FROM CONSOLIDATION TESTS USING THE WORK METHOD .
7. CORRELATED UNDRAINED SHEAR STRENGTH FROM CPT DATA IS BASED ON NDu=15 FROM EL. 15 TO -5 AND N_{kt}=22 BELOW (CPT-FJR-101, -102, AND -103) and NDu=13 FROM EL. 15 TO -5 AND N_{kt}=18 BELOW (CPT-FJR-104).
8. IN LEGEND, FV=UNDRAINED SHEAR STRENGTH FROM IN-SITU FIELD VANE, CONSOL=LAB DATA FROM CONSOLIDATION TEST.

LEGEND

Plastic Limit, PL — Water Content, Wn — Liquid Limit, LL

BB-FJR-101 (FV)	BB-FJR-104 (Su/P=.21)
BB-FJR-102 (FV,Consol)	BB-FJR-103 (Su/P=.21)
BB-FJR-102A (FV,Consol)	DESIGN Su PROFILE
BB-FJR-103 (FV, Consol)	CPT-FJR-103 (Nkt 22 NDu 15)
BB-FJR-104 (FV, Consol)	CPT-FJR-104 (Nkt 18 NDu 13)
BB-FJR-105 (FV, Consol)	CKoUC - BB-FJR-102A
CPT-FJR-101 (Nkt 22 NDu 15)	CKoUC - BB-FJR-104
CPT-FJR-102 (Nkt 22 NDu 15)	DSS (BB-FJR-104)
DSS (BB-FJR-103)	

JOHNSON ROAD BRIDGE REPLACEMENT
FALMOUTH, ME

IN-SITU SOIL CONDITIONS VS. ELEVATION
Abutment 1 Approach

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers & Scientists www.gza.com		PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: --	FIGURE 4
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: N/A	
DATE: 2/23/2022	PROJECT NUMBER: 09.0026024.00	REVISION NUMBER: 0	

APPENDIX A – LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have

been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

APPENDIX B –TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

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

Rock Quality Based on RQD

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

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

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Johnson Road Bridge #5792 over I-295 **Location:** Falmouth, Maine	**Boring No.:** BB-FJR-101 **WIN:** 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	28.9	**Auger ID/OD:**	4.25" SSA
Operator:	B. Enos/J. Maynard	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/26/19-6/26/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 2+48.0, 44.1' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	7.5' bgs
Hammer Efficiency Factor:	0.895	**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	24/16	0.0 - 2.0	WOH-WOH-WOH-WOH	-		SSA	28.6		Top 3": Topsoil (0.3) Bottom 13": Brown, dry, very loose, fine to medium SAND, little silt, with organics, (Fill).	
	2D	24/13	2.0 - 4.0	3-2-3-2	5	7				Brown, dry, loose, fine to medium SAND, trace silt, (Fill).	
	3D	24/18	4.0 - 6.0	1-2-2-5	4	6				Brown, moist, loose, fine to medium SAND, trace silt, (Fill).	
5											
	4D	24/24	6.0 - 8.0	6-8-10-13	18	27				Brown, wet, medium dense, fine to medium SAND, trace silt, (Fill).	
	5D	24/24	8.0 - 10.0	6-7-10-8	17	25				Brown, wet, medium dense, fine to medium SAND, little silt, (Fill).	
10							35				
							33				
							39	16.9		12.0	
							41				
	6D	24/2	14.0 - 16.0	5-3-1-2	4	6	PUSH			Grey, wet, medium stiff, Silty CLAY, with wood fragments, (Marine Clay Crust).	
15							35				
							37				
							32	11.9		17.0	
							30				
	7D	24/14	19.0 - 21.0	Push thru vane	-		RC			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm vane raw torque readings: V1: 120/10 in-lbs V2: 120/60 in-lbs	
20	V1		19.6 - 20.0	S_u=275/27 psf							
	V2		20.6 - 21.0	S_u=275/137 psf							
25	8D	24/12	24.0 - 26.0	WOR-WOR-WOR-WOR	-					Grey, wet, Silty CLAY, (Marine Clay).	

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based on percentages passing specific grain sizes.
2. Automatic hammer NEBC #D19 Energy Transfer Ratio = 0.895
3. Water level measured immediately after removal of casing.
4. 4" casing advanced to 19' bgs, then boring was advanced using open hole techniques utilizing bentonite slurry.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine
Boring No.: BB-FJR-101
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	28.9	**Auger ID/OD:**	4.25" SSA
Operator:	B. Enos/J. Maynard	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/26/19-6/26/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 2+48.0, 44.1' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	7.5' bgs
Hammer Efficiency Factor:	0.895	**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
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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											
	9D	24/24	29.0 - 31.0	Push thru vane	-					Grey, wet, Silty CLAY, (Marine Clay). 65x130mm vane raw torque readings: V3: 170/10 in-lbs V4: 215/20 in-lbs	
30	V3		29.6 - 30.0	S_u=390/27 psf							
	V4		30.6 - 31.0	S_u=492/47 psf							
	10D	24/7	34.0 - 36.0	WOR-WOR-WOR-WOR	-					Grey, wet, soft, Silty CLAY, trace gravel, (Marine Clay).	
35											
	11D	24/24	39.0 - 41.0	Push thru vane	-					Grey, wet, Silty CLAY, (Marine Clay). 65x130mm vane raw torque readings: V5: 200/15 in-lbs V6: 215/15 in-lbs	
40	V5		39.6 - 40.0	S_u=459/36 psf							
	V6		40.6 - 41.0	S_u=492/36 psf							
	12D	24/6	44.0 - 46.0	WOR-WOR-WOR-WOR	-					G rey, wet, Silty CLAY, trace fine sand, (Marine Clay).	
45											
	13D	24/24	49.0 - 51.0	Push thru vane	-					Grey, wet, medium stiff, Silty CLAY, trace fine sand, (Marine Clay). 65x130mm vane raw torque readings:	
50											

Remarks:

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2. Automatic hammer NEBC #D19 Energy Transfer Ratio = 0.895
3. Water level measured immediately after removal of casing.
4. 4" casing advanced to 19' bgs, then boring was advanced using open hole techniques utilizing bentonite slurry.

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Maine Department of Transportation

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Date Start/Finish:	6/26/19-6/26/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 2+48.0, 44.1' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	7.5' bgs
Hammer Efficiency Factor:	0.895	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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50	V7 V8		49.6 - 50.0 50.6 - 51.0	S_u=687/47 psf S_u=777/58 psf						V7: 300/20 in-lbs V8: 340/25 in-lbs	
								-24.1		53.0	
	14D	24/24	54.0 - 56.0	6-1-1-2	2	3				Grey, wet, very loose, fine Silty SAND, (Lower Marine Sand).	
55											
	15D	24/14	59.0 - 61.0	4-6-9-11	15	22				Grey, wet, medium dense, fine SAND, little silt, (Lower Marine Sand).	
60											
	16D	24/18	64.0 - 66.0	9-13-16-17	29	43				Grey/brown, wet, dense, fine SAND, little silt, (Lower Marine Sand).	
65											
	17D	24/11	69.0 - 71.0	6-8-12-15	20	30				Grey/brown, wet, medium dense, fine SAND, little silt, (Lower Marine Sand).	
70											
	18D	24/15	74.0 - 76.0	12-15-17-16	32	48				Grey/brown, wet, dense, fine to medium SAND, little silt, (Lower Marine Sand).	
75											

Remarks:

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75											
	19D	24/12	79.0 - 81.0	10-17-20-19	37	55				Grey/brown, wet, very dense, fine to coarse SAND, little silt, trace gravel, (Lower Marine Sand).	
80											
	20D	24/6	84.0 - 86.0	9-9-11-13	20	30				Grey/brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel, (Lower Marine Sand).	
85											
								-57.1		86.0 **Bottom of Exploration at 86.0 feet below ground surface.**	
90											
95											
100											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based on percentages passing specific grain sizes.
2. Automatic hammer NEBC #D19 Energy Transfer Ratio = 0.895
3. Water level measured immediately after removal of casing.
4. 4" casing advanced to 19' bgs, then boring was advanced using open hole techniques utilizing bentonite slurry.

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:	Johnson Road Bridge #5792 over I-295	**Location:**	Falmouth, Maine	**Boring No.:**	BB-FJR-102
				WIN:	21721.00
Driller:	Summit Geoengineering Services, Inc.	**Elevation (ft.)**	36.9	**Auger ID/OD:**	1.5"/2"
Operator:	Craig Coolidge, P.E.	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	C. Sullivan	**Rig Type:**	AMS 9500 VTR	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/19-5/30/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	Sta. 4+25.7, 50.2' Lt	**Casing ID/OD:**	3/3.5"	**Water Level*:**	12.3' bgs
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HYD			Advanced boring to 30' and began conducting field vanes in clay, based on results of CPT-FJR-102. Summit collected splitspoon samples and are not shown on this log, additional information can be found in the report provided to GZA by Summit Engineering.	
5											
10											
15											
20											
25											

Remarks:

1. Boring was completed by Summit Geoengineering. GZA prepared log using information included in Summit's report dated 6/10/2019.
2. 3" casing advanced by pushing with central plug.
3. Field vanes consisted of Geonor vanes with rod diameter of 0.5". Raw torque readings were converted to undrained shear strength and were rounded to nearest 25 psf by Summit.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-102
WIN: 21721.00

Driller:	Summit Geoengineering Services, Inc.	**Elevation (ft.)**	36.9	**Auger ID/OD:**	1.5"/2"
Operator:	Craig Coolidge, P.E.	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	C. Sullivan	**Rig Type:**	AMS 9500 VTR	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/19-5/30/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	Sta. 4+25.7, 50.2' Lt	**Casing ID/OD:**	3/3.5"	**Water Level*:**	12.3' bgs
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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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											
30	V1		30.6 - 31.0	S_u=425/50 psf						55x110mm vane raw torque reading: V1: 8.5/1 ft-lbs	
	V2		31.6 - 32.0	S_u=500/50 psf						V2: 10/1 ft-lbs	
	V3		33.6 - 34.0	S_u=400/50 psf						V3: 8/1 ft-lbs	
	V4		34.6 - 35.0	S_u=450/50 psf						V4: 9/1 ft-lbs	
35											
	V5		36.6 - 37.0	S_u=450/75 psf						V5: 9/1.5 ft-lbs	
	V6		37.6 - 38.0	S_u=450/50 psf						V6: 9/1 ft-lbs	
	V7		39.6 - 40.0	S_u=400/50 psf						V7: 8/1 ft-lbs	
40											
45							PUSH				
	V8		47.6 - 48.0	S_u=350/50 psf						55x110 mm vane raw torque reading: V8: 7/1 ft-lbs	
	V9		48.6 - 49.0	S_u=350/50 psf						V9: 7/1 ft-lbs	
50											

Remarks:

1. Boring was completed by Summit Geoengineering. GZA prepared log using information included in Summit's report dated 6/10/2019.
2. 3" casing advanced by pushing with central plug.
3. Field vanes consisted of Geonor vanes with rod diameter of 0.5". Raw torque readings were converted to undrained shear strength and were rounded to nearest 25 psf by Summit.

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:	Johnson Road Bridge #5792 over I-295	**Boring No.:**	BB-FJR-102		
Location:	Falmouth, Maine	**WIN:**	21721.00		
Driller:	Summit Geoengineering Services, Inc.	**Elevation (ft.)**	36.9	**Auger ID/OD:**	1.5"/2"
Operator:	Craig Coolidge, P.E.	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	C. Sullivan	**Rig Type:**	AMS 9500 VTR	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/19-5/30/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	--
Boring Location:	Sta. 4+25.7, 50.2' Lt	**Casing ID/OD:**	3/3.5"	**Water Level*:**	12.3' bgs
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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											
	V10		51.0 - 51.4	S_u=425/50 psf						V10: 8.5/1 ft-lbs	
	V11		52.0 - 52.4	S_u=450/50 psf						V11: 9/1 ft-lbs	
	V12		54.0 - 54.4	S_u=525/75 psf						V12: 11/1.5 ft-lbs	
55	V13		55.0 - 55.4	S_u=550/75 psf				-18.6		V13: 11.5/1.5 ft-lbs 55.5 **Bottom of Exploration at 55.5 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

1. Boring was completed by Summit Geoengineering. GZA prepared log using information included in Summit's report dated 6/10/2019.
2. 3" casing advanced by pushing with central plug.
3. Field vanes consisted of Geonor vanes with rod diameter of 0.5". Raw torque readings were converted to undrained shear strength and were rounded to nearest 25 psf by Summit.

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:** Johnson Road Bridge #5792 over I-295 **Location:** Falmouth, Maine	**Boring No.:** BB-FJR-102A **WIN:** 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	36.7	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/20/19-8/21/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+20.4, 51.2' Lt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	13' bgs
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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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	24/18	0.0 - 2.0	2-4-7-10	11	11	SSA			Brown, dry, medium dense, fine to medium SAND, little silt, with rootlets, (Fill).	
	2D	24/22	3.0 - 5.0	10-10-10-27	20	20				Brown, dry, medium dense, fine to medium SAND, little silt, (Fill).	
5											
	3D	24/24	8.0 - 10.0	12-10-8-9	18	18	78			Brown, dry, medium dense, fine to medium SAND, trace silt, (Fill).	
							38				
10											
							9				
							10 8	24.3		12.4 Upper Marine Sand interface assumed from CPT-FJR-103.	
							13				
							29				
15							14				
							21				
							25				
							35				
20							64				
							46 21	14.8		21.9 Marine Clay interface assumed from CPT-FJR-103.	
							42				
							36				
25							33				

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. 4" casing advanced 25' bgs.
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-102A
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	36.7	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/20/19-8/21/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+20.4, 51.2' Lt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	13' bgs
Hammer Efficiency Factor:	0.6	**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
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	4D	24/13	25.0 - 27.0	Push thru vane	-		RC			Grey, wet, medium stiff, Silty CLAY, little fine sand, (Marine Clay). 65x130 mm vane raw torque readings: V1: 270/40 in-lbs V2: 250/30 in-lbs	
	V1		25.6 - 26.0	S_u=618/91 psf							
	V2		26.6 - 27.0	S_u=571/69 psf							
30	1U	24/24	30.0 - 32.0	PUSH							
35	5D	24/24	35.0 - 37.0	Push thru vane	-					Grey, wet, medium stiff to soft, Silty CLAY, (Marine Clay). 65x130 mm vane raw torque readings: V3: 220/20 in-lbs V4: 210/20 in-lbs	
	V3		35.6 - 36.0	S_u=503/47 psf							
	V4		36.6 - 37.0	S_u=481/47 psf							
40	2U	24/23	40.0 - 42.0	PUSH							
45	6D	24/24	45.0 - 47.0	Push thru vane	-					Grey, wet, medium stiff to soft, Silty CLAY, (Marine Clay). 65x130 mm vane raw torque readings: V5: 220/20 in-lbs V6: 210/15 in-lbs	
	V5		45.6 - 46.0	S_u=503/47 psf							
	V6		46.6 - 47.0	S_u=481/36 psf							
50											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. 4" casing advanced 25' bgs.
3. Water level measured immediately after removal of casing.

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:** Johnson Road Bridge #5792 over I-295 **Location:** Falmouth, Maine	**Boring No.:** BB-FJR-102A **WIN:** 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	36.7	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/20/19-8/21/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+20.4, 51.2' Lt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	13' bgs
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
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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.
50	3U	24/23	50.0 - 52.0	PUSH							
								-15.3		52.0 **Bottom of Exploration at 52.0 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. 4" casing advanced 25' bgs.
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-103
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	47.8	**Auger ID/OD:**	4.25" OD SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/12/19-6/13/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	16.4' bgs
Hammer Efficiency Factor:	.937	**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			-ASPHALT-	
	1D	24/15	1.5 - 3.5	17-18-15-18	33	52		46.3		1.5 Brown, dry, very dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#S-4 A-1-b, SP-SM WC=2.6
5	2D	24/20	5.0 - 7.0	11-12-13-22	25	39	63			Top 12": Brown, dry, dense, fine to medium SAND, trace silt, (Fill). Bottom 8": Brown/black, moist, dense, fine Silty SAND, (Fill).	G#S-5 A-3, SP-SM WC=13.8
							96				
							145				
							183				
							272				
10	3D	24/15	10.0 - 12.0	14-22-19-34	41	64	82			Brown, wet, very dense, fine SAND, trace silt, (Fill).	
							140				
							221				
							320				
							70				
15	4D	24/19	15.0 - 17.0	13-33-33-43	66	103	RC			Brown, wet, very dense, fine SAND, some silt, (Fill).	
20	5D	24/13	20.0 - 22.0	9-12-14-16	26	41				Brown, wet, dense, fine to medium SAND, trace silt, (Fill).	
	6D	24/10	23.0 - 25.0	7-10-14-15	24	37				Brown, wet, dense, fine to medium SAND, trace silt, (Fill).	G#S-6 A-3, SP-SM
25											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295 Location: Falmouth, Maine	Boring No.: BB-FJR-103 WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	47.8	**Auger ID/OD:**	4.25" OD SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/12/19-6/13/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	16.4' bgs
Hammer Efficiency Factor:	.937	**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	24/14	25.0 - 27.0	6-10-13-20	23	36				Top 12": Brown, wet, dense, fine to medium SAND, trace silt, (Fill).	
								21.6		26.2 Bottom 2": Grey, wet, Silty fine to medium SAND, little gravel, (Upper Marine Sand).	
	8D	24/11	27.0 - 29.0	10-6-3-1	9	14				Grey, wet, medium dense Silty fine to medium SAND, (Upper Marine Sand).	G#S-7 A-4, SM WC=15.2
30	9D	24/15	30.0 - 32.0	1-1-3-6	4	6	110	17.8		30.0 Grey, moist, medium stiff, SILT, little fine sand, (Marine Clay Crust).	
							112				
							131				
							145				
							161				
35	10D	24/24	35.0 - 37.0	2-WOH-1-WOH			83			Grey, wet, very loose, Sandy SILT, (Marine Clay Crust).	G#S-8 A-5(1), MH WC=30.6
							83				
							116				
							130	9.8		38.0	
							119				
40	11D V1	24/15	40.0 - 42.0 40.6 - 41.0	Push thru vane S_u=522/110 psf	-		103			Grey, wet, medium stiff, Silty CLAY, (Marine Clay). 65x130mm vane raw torque readings: V1: 19/4 ft-lbs V2: 20/3 ft-lbs	
	V2		41.6 - 42.0	S_u=549/82 psf			92				
							100				
							83				
							90				
45	1U	24/23	45.0 - 47.0	PUSH			68			Grey, wet, Silty CLAY, (Marine Clay).	
							61				
	12D V3	24/20	47.0 - 49.0 47.6 - 48.0	Push thru vane S_u=412/27 psf			85			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm vane raw torque reading: V3: 15/1 ft-lbs V4: 12/1 ft-lbs	
	V4		48.6 - 49.0	S_u=330/27 psf			93				
50							90				

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-103
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	47.8	**Auger ID/OD:**	4.25" OD SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/12/19-6/13/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	16.4' bgs
Hammer Efficiency Factor:	.937	**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	13D	24/24	50.0 - 52.0	Push thru vane			80			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm vane raw torque reading: V5: 14/2 ft-lbs V6: 9/1 ft-lbs	
	~~V5~~		~~50.6 - 51.0~~	~~S_u=384/85 psf~~							
	V6		51.6 - 52.0	S_u=247/27 psf			78				
							85				
							93				
							115				
55	2U	24/24	55.0 - 57.0	Push			100				
							104				
	14D	24/24	57.0 - 59.0	Push thru vane			77			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm vane new torque reading: V7: 18/3 ft-lbs V8: 13/1 ft-lbs	
	~~V7~~		~~57.6 - 58.0~~	~~S_u=494/82 psf~~							
	V8		58.6 - 59.0	S_u=357/27 psf			86				
							79				
60	15D	24/24	60.0 - 62.0	WOR-WOR-WOR-WOR			73			Grey, wet, Silty CLAY, (Marine Clay).	
							82				
							75				
							68				
							85				
65	3U	24/24	65.0 - 67.0	PUSH			110			Similar to above.	
							95				
	16D	24/24	67.0 - 69.0	Push thru vane			85			Grey, wet, medium stiff, Silty CLAY, little fine sand, (Marine Clay). 65x130mm vanes raw torque reading: V9: 19/2 ft-lbs Vane advancement refusal at 68.5'.	
	~~V9~~		~~67.6 - 68.0~~	~~S_u=522/55 psf~~							
	MV	4/4	68.2 - 68.5				90				
							89				
70							91				
							115	-24.2		72.0	
							100				
							102				
75							98				

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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:	Johnson Road Bridge #5792 over I-295			**Boring No.:**	BB-FJR-103
Location:	Falmouth, Maine			**WIN:**	21721.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	47.8	**Auger ID/OD:**	4.25" OD SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/12/19-6/13/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	16.4' bgs
Hammer Efficiency Factor:	.937	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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
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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	17D	24/24	75.0 - 77.0	1-WOH-2-5	2	3	117			Grey, wet, very loose, Silty fine SAND (Lower Marine Sand).	
							96				
							88				
							96				
							94				
80	18D	24/24	80.0 - 82.0	1-1-3-5	4	6	93			Grey, wet, loose, fine SAND, some silt, (Lower Marine Sand).	G#S-13 A-2-4(0), SM
							88				
							70				
							85				
							132				
85							88				
	19D	24/12	86.0 - 88.0	1-4-4-5	8	12	96			Grey/orange, wet, medium dense, fine SAND, trace silt, (Lower Marine Sand).	
							107				
							126				
							116				
90							111				
	20D	24/5	91.0 - 93.0	WOH-2-6-10	8	12	108			Grey, wet, medium dense, fine to medium SAND, trace silt, (Lower Marine Sand).	
							110				
							127				
							122				
95	21D	24/11	95.0 - 97.0	3-4-6-7	10	16	115			Black, wet, medium dense, fine to coarse SAND, trace silt, (Lower Marine Sand).	
							120				
							105				
							195	-50.2		98.0	
100							289				

Remarks:

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3. Water level measured immediately after removal of casing.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-103
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	47.8	**Auger ID/OD:**	4.25" OD SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/12/19-6/13/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	16.4' bgs
Hammer Efficiency Factor:	.937	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
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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.
100	22D	5/5	100.0 - 100.4	55	55/5"	86	RC	-52.6		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt. Note: Pieces of gravel mostly in spoon tip, (Glacial Till).	
										100.4	
										Rolled through/cored probable boulder from 100.4'-106.1'.	
105											
								-58.3		106.1	
	23D	24/12	106.1 - 108.1	26-25-25-28	50	78				Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, (Glacial Till).	G#S-14 A-1-b, SM
110	24D	14/9	110.0 - 111.2	18-153-102/2"	255/8"	398				Grey, wet, very dense, fine to medium SAND, some silt, little gravel, (Glacial Till).	
115	25D	18/8	115.0 - 116.5	42-105-95	200	312				Grey, wet, very dense, fine to medium SAND, some silt, little gravel, trace coarse sand, (Glacial Till).	
120	26D	5/4	120.0 - 120.4	142	142/5"	222	NX	-72.6		Grey, wet, very dense, fine to medium SAND, some silt, little gravel, (Glacial Till).	
	R1	60/59	120.4 - 125.4	RQD = 80%						120.4	
										Advance roller cone to 120.6'. Consistent, grey rock fragments and metal filings in wash return. Switch to 3" casing. Set up to core at 120.6'.	
								-74.7		R1: 120.6'-122.5': Very hard to hard, fresh, aphanitic to medium grained, white-black, banded to green, GRAPHITE SCHIST.	
										122.5	
										R1: 122.5'-125.6': Very hard to hard, fresh, aphanitic to medium grained, white, PEGMATITE. Joints are very close to wide, horizontal	
125											

Remarks:

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2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Johnson Road Bridge #5792 over I-295 Location: Falmouth, Maine	Boring No.: BB-FJR-103 WIN: 21721.00

Driller:	New England Boring Contractors	Elevation (ft.)	47.8	Auger ID/OD:	4.25" OD SSA
Operator:	Mike Porter	Datum:	NAVD 88	Sampler:	Standard Splitspoon
Logged By:	L. Navarrete	Rig Type:	Truck Mobile B-53	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/12/19-6/13/19	Drilling Method:	Drive & Wash	Core Barrel:	NX
Boring Location:	Sta. 4+10.8, 4.4' Rt	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	16.4' bgs
Hammer Efficiency Factor:	.937	Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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.
125	R2	60/60	125.6 - 130.6	RQD = 80%				-77.8		to high angle, undulating, rough to smooth, fresh to discolored, moderately wide, with silt infilling. -VASSALBORO FORMATION- Rock Quality = Good Recovery = 78% Rock Core Times (min:sec): 120.6-121.6' (2:12), 121.6-122.6' (2:26), 122.6-123.6' (2:04), 123.6-124.6' (1:54), 124.6-125.6' (2:10) 125.6	
								-81.1		R2: 125.6'-128.9': Very hard to hard, fresh, fine grained to aphanitic, white-black banded, GRAPHITE SCHIST. 128.9	
130								-82.8		R2: 128.9'-130.6': Hard, fresh, aphanitic, black, SLATE. Joints are close to wide, moderately dipping, undulating, rough to smooth, discolored, open to moderately wide, no infilling. -VASSALBORO FORMATION- Rock Quality = Good Recovery = 100% Rock Core Times (min:sec): 125.6-126.6' (2:11), 126.6-127.6' (2:18), 127.6-128.6' (2:16), 128.6-129.6' (3:52), 129.6-130.6' (3:15) 130.6 **Bottom of Exploration at 130.6 feet below ground surface.**	
135											
140											
145											
150											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-104
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	29.5	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/19-8/20/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+53.0, 63.4' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	3.6' bgs/0.6' bgs
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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T_v = Pocket Torvane Shear Strength (psf)
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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	24/14	0.0 - 2.0	3-4-12-14	16	16	SSA			Brown, dry, medium dense, fine to medium SAND, trace silt, with rootlets, (Fill).	
	2D	24/18	4.0 - 6.0	3-8-10-8	18	18				Brown, moist, medium dense, fine to medium SAND, trace silt, trace gravel, (Fill).	G#S-27 A-3, SP-SM WC=6.6
5											
								21.5		8.0	
							11				
	3D	24/24	9.0 - 11.0	WOR-3-5-5	8	8	37			Grey, wet, loose, fine to medium SAND and Silty Clay, trace organics and rootlets, (Upper Marine Sand).	
10							38				
							50	17.5		12.0	
							42				
							35				
							51				
15	4D ~~MV1~~	24/24	15.0 - 17.0 ~~15.6 - 16.0~~	2-1-3-3	4	4	127			Grey, wet, medium stiff, Silty CLAY, some fine sand, (Marine Clay Crust).	
							72				
							54	12.5		17.0	
							44				
							36				
20	5D V1	24/24	20.0 - 22.0 20.6 - 21.0	Push thru vane S_u=549/69 psf	-		77			Grey, wet, medium stiff, Silty CLAY, (Marine Clay). 65x130 mm vane raw torque readings: V1: 240/30 in-lbs V2: 220/25 in-lbs	
	V2		21.6 - 22.0	S_u=503/58 psf			53				
							44				
							34				
25							28				

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Well installed to 70 bgs, 10' of 2" ID PVC screen, 62' of 2" ID PVC riser, sand backfill up to 58.8' bgs, bentonite chips from 58.8' bgs to 43' bgs, sand backfill from 43' bgs to ground surface. 3' standpipe with cap at ground surface.
3. First water level measured 10 minutes after drilling within casing. Second water level measured in well 5 hours after completion of installation.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-104
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	29.5	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/19-8/20/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+53.0, 63.4' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	3.6' bgs/0.6' bgs
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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	6D	24/24	25.0 - 27.0	Push thru vane	-		103			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130 mm vane raw torque readings: V3: 180/20 in-lbs V4: 120/10 in-lbs	
	V3		25.6 - 26.0	S_u=412/47 psf							
	V4		26.6 - 27.0	S_u=275/27 psf			54				
							39				
							30				
							33				
30	1U	24/20	30.0 - 32.0	PUSH			28				
							36				
	V5		32.6 - 33.0	S_u=412/27 psf			31			65x130 mm vane raw torque readings: V5: 180/10 in-lbs	
	V6		33.6 - 34.0	S_u=297/27 psf			29			V6: 130/10 in-lbs	
							48				
35	7D	24/24	35.0 - 37.0	Push thru vane	-		35			Grey, wet, soft, Silty CLAY, little fine sand, (Marine Clay). 65x130 mm vane raw torque readings: V7: 170/20 in-lbs V8: 190/25 in-lbs	
	V7		35.6 - 36.0	S_u=390/47 psf							
	V8		36.6 - 37.0	S_u=434/55 psf			↓				
							35				
							39				
40	2U	24/24	40.0 - 42.0	PUSH			37				
							44				
	V9		42.6 - 43.0	S_u=503/47 psf			37			65x130 mm vane raw torque readings: V9: 220/20 in-lbs	
	V10		43.6 - 44.0	S_u=481/47 psf			32			V10: 210/20 in-lbs	
							38				
45							55				
							50				
							61				
							45				
							45				
50											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Well installed to 70 bgs, 10' of 2" ID PVC screen, 62' of 2" ID PVC riser, sand backfill up to 58.8' bgs, bentonite chips from 58.8' bgs to 43' bgs, sand backfill from 43' bgs to ground surface. 3' standpipe with cap at ground surface.
3. First water level measured 10 minutes after drilling within casing. Second water level measured in well 5 hours after completion of installation.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-104
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	29.5	**Auger ID/OD:**	4.25" SSA
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	M. Walsh	**Rig Type:**	ATV M-1	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/19/19-8/20/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+53.0, 63.4' Rt	**Casing ID/OD:**	4/4.5"	**Water Level*:**	3.6' bgs/0.6' bgs
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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	8D	24/24	50.0 - 52.0	Push thru vane	-		83			Grey, wet, medium stiff, Silty CLAY, some fine sand, (Marine Clay). 65x130 mm vane raw torque readings: V11: 270/30 in-lbs V12: 300/30 in-lbs	
	V11		50.6 - 51.0	S_u=618/69 psf							
	V12		51.6 - 52.0	S_u=687/69 psf			41				
							39				
							47	-24.5		54.0	
							49				
55	9D	24/24	55.0 - 57.0	WOR-5-7-14	12	12	77			Grey, wet, medium dense, Silty fine to medium SAND, (Lower Marine Sand).	
	MV2		55.6 - 56.0								
							58				
							43				
							43				
							42				
60	10D	24/24	60.0 - 62.0	WOR-7-8-9	15	15	57			Grey, wet, medium dense, Silty fine to medium SAND, (Lower Marine Sand).	G#S-33 A-4, SM WC=28.6
							44				
							55				
							59				
							59				
65	11D	24/22	65.0 - 67.0	WOR-WOR-4-9	4	4	129			Grey/brown, wet, very loose, fine to medium SAND, little silt, with orange bands, (Lower Marine Sand).	
							126				
							121				
							117				
							168				
70	12D	18/0	70.0 - 71.5	9-10-25	35	35				No recovery.	
								-42.0		71.5 **Bottom of Exploration at 71.5 feet below ground surface.**	
75											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterberg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Well installed to 70 bgs, 10' of 2" ID PVC screen, 62' of 2" ID PVC riser, sand backfill up to 58.8' bgs, bentonite chips from 58.8' bgs to 43' bgs, sand backfill from 43' bgs to ground surface. 3' standpipe with cap at ground surface.
3. First water level measured 10 minutes after drilling within casing. Second water level measured in well 5 hours after completion of installation.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-105
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	33.4	**Auger ID/OD:**	--
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/19-6/6/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 5+33.1, 6.7' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	8' bgs
Hammer Efficiency Factor: .937		**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							RC			-ASPHALT-	
	1D	24/10	2.0 - 4.0	7-5-6-5	11	17	21	31.4		2.0 Brown, wet, medium dense, fine to medium SAND, trace gravel, trace silt, (Fill).	G#S-15 A-3, SP-SM WC=17.7
							23				
							20	28.9		4.5	
5							26				
							25				
	2D	24/6	7.0 - 9.0	6-4-1-WOR	5	8	10			Grey, wet, loose, fine, Silty SAND, (Upper Marine Sand).	
							15				
							27				
10							48	22.9		10.5	
							66				
	3D	24/14	12.0 - 14.0	2-3-3-3	6	9	68			Grey, wet, stiff, Silty CLAY, (Marine Clay Crust).	
							63				
							63				
15	MV						65				
							52				
	4D	24/24	17.0 - 19.0	WOP-WOP-WOP-WOP			PUSH			Grey, wet, Silty CLAY. Small sand seams < 3/8" from sample, (Marine Clay Crust).	
								14.4		19.0	
20	1U	24/24	20.0 - 22.0							Grey, wet, Silty CLAY, (Marine Clay).	
	5D V1	24/24	22.0 - 24.0 22.6 - 23.0	WOR-WOR-WOR-WOR S_u=549/55 psf						Grey, wet, medium stiff, Silty CLAY. (Marine Clay). 65x130mm raw torque reading V1: 20/2.5 ft-lbs. V2: 19/0.5 ft-lbs.	
	V2		23.6 - 24.0	S_u=522/14 psf			19				
25							48				

Remarks:

1. Cored into pavement with 4" spin shoe. Pavement on bridge 0.8' thick. Ground level 22.3' below top of bridge deck.
2. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
3. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
4. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295 Location: Falmouth, Maine	Boring No.: BB-FJR-105 WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	33.4	**Auger ID/OD:**	--
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/19-6/6/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 5+33.1, 6.7' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	8' bgs
Hammer Efficiency Factor:	.937	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)	T_v = Pocket Torvane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample Attempt	SSA = Solid Stem Auger	$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)	WC = Water Content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample Attempt	RC = Roller Cone	N-uncorrected = Raw Field SPT N-value	PL = Plastic Limit
V = Field Vane Shear Test, PP = Pocket Penetrometer	WOH = Weight of 140 lb. Hammer	Hammer Efficiency Factor = Rig Specific Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Field Vane Shear Test Attempt	WOR/C = Weight of Rods or Casing	N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency	G = Grain Size Analysis
	WO1P = Weight of One Person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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							30				
							28				
							22				
	2U	24/24	28.0 - 30.0				39			Grey, wet, soft, Silty CLAY, (Marine Clay).	
							30				
30	6D V3	24/24	30.0 - 32.0 30.6 - 31.0	Push thru vane S_u=412/69 psf			31			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm raw torque reading V3: 15/2.5 ft-lbs. V4: 15/2.0 ft-lbs.	
	V4		31.6 - 32.0	S_u=412/55 psf			31				
							30				
	7D V5	24/24	33.0 - 35.0 33.6 - 34.0	Push thru vane S_u=439/55 psf			21			Grey, wet, soft, Silty CLAY, (Marine Clay). 65x130mm raw torque reading. V5: 16/2.0 ft-lbs. V6: 17/2.0 ft-lbs.	
	V6		34.6 - 35.0	S_u=467/55 psf			21				
35							23				
							22				
							21				
	8D V7	24/24	38.0 - 40.0 38.6 - 39.0	Push thru vane S_u=714/165 psf			19			Grey, wet, medium stiff, Silty CLAY, (Marine Clay). 65x130mm raw torque reading. V7: 26/6 ft-lbs. V8: 26/6 ft-lbs.	
	V8		39.6 - 40.0	S_u=714/165 psf			17				
40							19				
							22				
							22				
	9D V9	24/24	43.0 - 45.0 43.6 - 44.0	Push thru vane S_u=934/137 psf			26			Grey, wet, medium stiff, Silty CLAY, (Marine Clay). 65x130mm raw torque reading V9: 34/5 ft-lbs. V10: 26/5 ft-lbs.	
	V10		44.6 - 45.0	S_u=714/137 psf			24				
45							26				
							28				
							32				
	10D	18/18	48.5 - 50.0	WOR-WOR-WOR			31			Top 9": Grey, wet, Silty CLAY, (Marine Clay).	
50							38	-15.9		49.3 Bottom 9": Brown, wet, fine SAND, little silt, (Lower Marine Sand).	

Remarks:

1. Cored into pavement with 4" spin shoe. Pavement on bridge 0.8' thick. Ground level 22.3' below top of bridge deck.
2. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
3. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
4. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-105
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	33.4	**Auger ID/OD:**	--
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/4/19-6/6/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 5+33.1, 6.7' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	8' bgs
Hammer Efficiency Factor:	.937	**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							60				
							51				
							48				
	11D	3/3	53.0 - 53.3	83/3"	83/3"	130	RC	-19.9		Grey, wet, GRAVEL, little fine sand, (Lower Marine Sand). 53.3	
	R1	60/58	54.1 - 59.1	RQD = 90%			NX			Advance roller bit to 54.1'. Consistent grey rock fragments in wash return, probable top of rock. Set up to core at 54.1'. R1: Hard, fresh, fine to medium grained, white-grey, GNEISS. Joints are wide to close, high angle to moderately dipping, undulating, rough, discolored, open to wide, with silt and sand infilling. -VASSALBORO FORMATION- Rock Quality = Good Recovery = 97% Rock Core Times (min:sec): 54.1-55.1' (3:00), 55.1-56.1' (2:47), 56.1-57.1' (1:56), 57.1-58.1' (1:40), 58.1-59.1' (1:35)	
55											
	R2	60/59	59.1 - 64.1	RQD = 88%						R2: Hard to very hard, fresh, fine to medium grained, white-grey, GNEISS. Joints are close to wide, high angle to low angle, dipping, undulating, rough, discolored, moderately wide to open, with silt and sand infilling. -VASSALBORO FORMATION- Rock Quality = Good Recovery = 98% Rock Core Times (min:sec): 59.1-60.1' (3:05), 60.1-61.1' (2:35), 61.1-62.1' (2:10), 62.1-63.1' (1:30), 63.1-64.1' (1:24)	
60											
								-30.7		64.1 **Bottom of Exploration at 64.1 feet below ground surface.**	
65											
70											
75											

Remarks:

1. Cored into pavement with 4" spin shoe. Pavement on bridge 0.8' thick. Ground level 22.3' below top of bridge deck.
2. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
3. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
4. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-106
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	62.6	**Auger ID/OD:**	4.25" SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/3/19-6/4/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 6+77.3, 4.5' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	12.2' bgs
Hammer Efficiency Factor:	.937	**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			-ASPHALT-	
	1D	24/12	1.0 - 3.0	14-12-13-12	25	39		61.6		1.0 Brown, dry, dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#S-20 A-1-b, SP-SM
5	2D	24/24	5.0 - 7.0	3-3-3-3	6	9	29			Brown, dry, loose, fine to medium SAND, trace gravel, trace silt, (Fill).	G#S-21 A-3, SP-SM
							47				
							90				
							138				
							198				
10	3D	24/12	10.0 - 12.0	24-21-28-29	49	77	55			Brown, wet, very dense, fine to medium SAND, (Fill).	
							117				
							136				
							115				
							201				
15	4D	24/11	15.0 - 17.0	11-20-23-25	43	67	RC			Brown, wet, very dense, fine to medium SAND, (Fill).	
20	5D	24/11	20.0 - 22.0	8-26-16-20	42	66				Brown, wet, very dense, fine to medium SAND, (Fill).	
	6D	24/8	23.0 - 25.0	14-11-18-19	29	45	109	39.1		Top 6": Dark brown, wet, dense, fine to medium SAND, little silt, (Fill). 23.5	
25							148			Bottom 2": Grey, wet, Silty fine to coarse SAND, (Upper Marine	

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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:** Johnson Road Bridge #5792 over I-295 **Location:** Falmouth, Maine	**Boring No.:** BB-FJR-106 **WIN:** 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	62.6	**Auger ID/OD:**	4.25" SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/3/19-6/4/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 6+77.3, 4.5' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	12.2' bgs
Hammer Efficiency Factor:	.937	**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	24/3	25.0 - 27.0	9-8-8-6	16	25				Sand). Grey, wet, medium dense, fine to coarse SAND, some gravel, trace silt, (Upper Marine Sand).	
								34.6		28.0	
	8D	24/7	28.0 - 30.0	3-3-3-3	6	9	95			Grey, wet, stiff, Silty CLAY, (Marine Clay Crust).	
							105				
30	9D	24/19	30.0 - 32.0	2-1-2-WOR	3	5	RC			Olive to grey, wet, medium stiff, Silty CLAY, trace fine sand, (Marine Clay Crust).	
	U1	12/12	33.0 - 34.0	PUSH						Grey, wet, Silty CLAY, trace fine sand, (Marine Clay Crust).	
	10D	24/24	34.0 - 36.0	1-2-3-2	5	8				Grey, wet, medium stiff to stiff, Silty CLAY, some sand, with seams of fine sand, (Marine Clay Crust).	
35											
								23.3		39.3	
40	R1	60/59	39.6 - 44.6	RQD = 88%			NX			Advance roller bit to 39.3'; consistent grey rock fragments in wash return, probable top of rock. Set up to core at 39.6'. R1: Very hard, fresh, medium grained, white/grey, PEGMATITE. Joints are widely spaced to close, high angle, undulating, rough, discolored, open to moderately wide, with silt infilling. -VASSALBORO FORMATION- Rock Quality = Good Recovery = 90% Rock Core Times (min:sec): 39.6-40.6' (6:21), 40.6-41.6' (2:37), 41.6-42.6' (1:51), 42.6-43.6' (1:41), 43.6=44.6' (1:48)	
45	R2	60/60	44.6 - 49.6	RQD = 93%						R2: Top 4.3': Very hard, fresh, medium grained, PEGMATITE, similar to above.	
								13.7		48.9	
50								13.0		R2: Bottom 0.7': Very hard, fresh, fine to medium grained, QUARTZITE. Joints are widely spaced, moderately dipping,	

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-106
WIN: 21721.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	62.6	**Auger ID/OD:**	4.25" SSA
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	L. Navarrete	**Rig Type:**	Truck Mobile B-53	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/3/19-6/4/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 6+77.3, 4.5' Lt	**Casing ID/OD:**	4/4.5", 3/3.5"	**Water Level*:**	12.2' bgs
Hammer Efficiency Factor:	.937	**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										undulating, rough, fresh, open, no infilling. -VASSALBORO FORMATION- Rock Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 44.6-45.6' (2:32), 45.6-46.6' (1:38), 46.6-47.6' (1:40), 47.6-48.6' (2:01), 48.6-49.6' (1:22)	
										49.6	
										Bottom of Exploration at 49.6 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes.
2. Automatic hammer NEBC #B-24 Energy Transfer Ratio = 0.937
3. Water level measured immediately after removal of casing.

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: Johnson Road Bridge #5792 over I-295
Location: Falmouth, Maine

Boring No.: BB-FJR-107
WIN: 21721.00

Driller:	Summit Geoengineering Services, Inc.	**Elevation (ft.)**	46.9	**Auger ID/OD:**	1.5"/2"
Operator:	Craig Coolidge, P.E.	**Datum:**	NAVD 88	**Sampler:**	Standard Splitspoon
Logged By:	C. Sullivan	**Rig Type:**	AMS 9500 VTR	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/28/19-5/28/19	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NX
Boring Location:	Sta. 7+19.6, 69.4' Rt	**Casing ID/OD:**	3.5"	**Water Level*:**	12.4' bgs
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										Advanced boring to 19' and began conducting field vanes in clay, based on results of SCPT-FJR-105. Summit collected samples and they are not shown on this log. Additional information can be found in the report provided to GZA.	
5											PP = 5,000 to 6,000 psf
											PP = 3,000 to 4,000 psf
10	V1		10.0 - 10.4	S_u=1950/500 psf						55x110mm vane raw torque reading: V1: 40/10 ft-lbs	
15											PP = 1,500 to 2,000 psf
											PP = 500 psf +/-
								27.9		19.0 **Bottom of Exploration at 19.0 feet below ground surface.**	
20											
25											

Remarks:

1. Boring was completed by Summit Geoengineering. GZA prepared log using information included in Summit's report dated 6/10/2019.
2. 3" casing advanced by pushing with central plug.
3. Field vanes consisted of Geonor vanes with rod diameter of 0.5". Raw torque readings were converted to undrained shear strength and were rounded to nearest 25 psf by Summit.

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 C – CONE PENETRATION TEST REPORTS

APPENDIX C.1 – CPT DATA REPORT BY SUMMIT GEOENGINGEERING SERVICES

The key to success starts with a solid foundation.
ENGINEERING | EXPLORATION | EXPERIENCE

Exploration Report

MDOT I-295 Bridge

Johnson Road, Falmouth, Maine

Client

GZA GeoEnvironmntal, Inc.
477 Congress Street, Suite 700
Portland, Maine 04101

Project #: 19120
Date: 6/10/19

SUMMIT
GEOENGINEERING SERVICES

145 Lisbon Street (PO Box 7216) Lewiston, Maine 04243 | (207) 576-3313
173 Pleasant Street Rockland, Maine 04841 | (207) 318-7761
www.summitgeoeng.com

June 10, 2019
Summit #19120

Andrew R. Blaisdell, PE
GZA GeoEnvironmntal, Inc.
477 Congress Street, Suite 700
Portland, Maine 04101

Reference: Geotechnical Exploration Services
Piezocone Penetration Testing – Johnson Road Falmouth, Maine

Dear Mr. Blaisdell;

We have completed exploration services for a planned bridge project on Johnson Road in Falmouth, Maine. Summit Geoengineering Services (SGS) was asked to perform piezocone penetration testing (CPT) and prepare this data report summarizing results.

Work Description

The project site is located at Johnson Road Bridge over Interstate I-295 in Falmouth, Maine. Explorations were performed along the edge of the existing bridge abutments as shown on Figure 1 Exploration Location Plan. Work performed includes the following:

- 5 piezocone penetration tests with shear wave velocity
- 4 dissipation tests within soft clay
- 2 test borings with standard penetration tests and field vane shear tests

Explorations

Summit Geoengineering Services (SGS) performed 5 piezocone penetration tests (CPT) to include shear wave velocity on May 28 to 30, and June 3, 2019. CPT was performed in accordance with ASTM D5778. Shear wave velocity was performed at rod break intervals of 3.3 feet in accordance with ASTM D7400. Field vane shear tests (ASTM D2573) and standard penetration tests (ASTM D1586) were conducted within 2 test borings. Logs of the explorations are included at the end of this report.

CPT was advanced using a rubber track mounted PowerProbe 9500 VTR or a truck mounted PowerProbe 9630 Pro with a Vertek digital cone having a cross sectional area of 10 cm^2. CPT was performed to a depth of push refusal or anchor failure ranging from 21.3 to 105.4 feet below ground surface. Anchoring was conducting using a single point hollow anchor with start of test depth at 5 feet. Parameters obtained include cone resistance (q_c), sleeve friction (f_s), piezocone pore pressure (u_2), and shear wave velocity (V_s). Dissipation tests were conducted at CPT-FJR-101, CPT-FJR-102, and CPT-FJR-103.

Soil Behavior Type

Soil Behavior Type (SBT_N) profiling based on normalized cone penetration resistance (Robertson 1990, B_q method) indicates the following soil types generalized as follows:

CPT-FJR-101

- ***5 to 20 feet*** - Type 6 (clean sand to silty sand) to Type 7 (gravelly sand to sand)
- ***20 to 23 feet*** - Type 4 (clayey silt to silty clay) and Type 5 (silty sand to sandy silt)
- ***23 to 59.5 feet*** - Type 1 (Sensitive, fine grained) and Type 3 (clay to silty clay)

CPT-FJR-102

- ***5 to 18 feet*** - Type 4 (clayey silt to silty clay) to Type 7 (gravelly sand to sand)
- ***18 to 52 feet*** - Type 3 (clay to silty clay)
- ***52 to 53.8 feet*** - Type 6 (clean sand to silty sand)

CPT-FJR-103

- ***5 to 23 feet*** - Type 6 (clean sand to silty sand) and Type 7 (gravelly sand to sand)
- ***23 to 59 feet*** -Type 3 (clay to silty clay)
- ***59 to 65.1 feet*** - Type 4 (clayey silt to silty clay) to Type 6 (clean sand to silty sand)

CPT-FJR-104

- ***5 to 33 feet*** - Type 6 (clean sand to silty sand) and Type 7 (gravelly sand to sand)
- ***33 to 76 feet*** - Type 1 (Sensitive, fine grained) and Type 3 (clay to silty clay)
- ***76 to 105.4 feet*** - Type 4 (clayey silt to silty clay) to Type 6 (clean sand to silty sand)

CPT-FJR-105

- ***5 to 11 feet*** - Type 5 (silty sand to sandy silt) to Type 7 (gravelly sand to sand)
- ***11 to 13.5 feet*** - Type 4 (clayey silt to silty clay)
- ***13.5 to 18.5 feet*** - Type 3 (clay to silty clay)
- ***18.5 to 21.3 feet*** - Type 6 (clean sand to silty sand)

Soil behavior type 5 to type 7 are typical of granular fill and/or alluvium consisting of sand with variable silt and gravel. Type 1 to type 4 is typical of Presumpscot Formation consisting of silty clay with minor sand. Type 4 is commonly associated with upper firm clay and type 1 or 3 is commonly associated with lower soft clay. Details of the SBT_N profiling is shown on the attached CPT logs.

Shear Wave Velocity Tests

Shear wave velocity tests were conducted during performed of the cone penetration tests during rod break intervals. Waterfall plots showing details of the shear wave velocity results are included at the end of this report. Summary of the results are provided below:

SHEAR WAVE VELOCITY TEST SUMMARY									
CPT-FJR-101		CPT-FJR-102		CPT-FJR-103		CPT-FJR-104		CPT-FJR-105	
D (ft)	V_s (ft/s)	D (ft)	V_s (ft/s)	D (ft)	V_s (ft/s)	D (ft)	V_s (ft/s)	D (ft)	V_s (ft/s)
10.7	960	10.8	520	10.6	770	10.5	995	11.0	670
14.1	675	14.1	540	14.1	590	14.1	510	14.4	485
17.5	980	17.6	485	18.7	400	20.3	750	17.8	590
21.0	780	21.0	400	22.1	690	23.6	650	21.1	1005
24.5	695	24.4	425	25.6	550	27.0	715	--	--
27.8	570	28.2	570	29.0	475	30.3	630	--	--
31.3	525	31.5	450	32.4	515	33.8	725	--	--
34.7	485	34.8	505	35.7	465	37.1	615	--	--
38.0	535	38.2	515	39.1	580	40.6	550	--	--
41.1	500	41.5	625	42.5	490	43.9	485	--	--
44.6	500	44.8	500	45.9	595	47.2	475	--	--
48.0	625	48.2	570	49.2	595	50.7	505	--	--
51.0	480	51.4	555	51.9	450	54.1	545	--	--
54.4	630	53.7	1340	55.1	555	57.4	535	--	--
57.7	680	--	--	59.0	600	60.9	530	--	--
59.4	515	--	--	62.8	910	64.1	610	--	--
--	--	--	--	65.1	670	67.5	565	--	--
--	--	--	--	--	--	70.9	565	--	--
--	--	--	--	--	--	74.3	620	--	--
						77.7	650		
						81.1	790		
						84.5	745		
						87.9	760		
						91.0	715		
						94.4	880		
						97.5	1105		
						101.0	850		
						104.2	880		
--	--	--	--	--	--	105.4	1092	--	--
Ave	635	**Ave**	570	**Ave**	580	**Ave**	690	**Ave**	690

Dissipation Tests

Dissipation tests were performed at CPT-FJR-101, CPT-FJR-102, and CPT-FJR-103. The horizontal coefficient of consolidation (C_h) can be estimated using a T_{50} determined for time for piezocone pore pressure to dissipate to 50 percent (U_{50}). Piezocone pore pressure is measured at the u_2 position located behind the cone tip. Graphic results of the dissipation tests are included at the end of this report. Summary of the results are provided below:

145 Lisbon Street (PO Box 7216) Lewiston, Maine 04243 | (207) 576-3313
173 Pleasant Street Rockland, Maine 04841 | (207) 318-7761
www.summitgeoeng.com

DISSIPATION TEST SUMMARY						
Location	**Depth (ft)**	**U_0 (psi)**	**U_{100} (psi)**	**U_{50} (psi)**	**T_{50} (min)**	**C_h (cm²/min)**
CPT-FJR-101 GW = 10.2 ft	28.7	54.3	8.0	31.1	73.6	0.17
CPT-FJR-102 GW = 5.0 ft	26.5	55.5	9.3	32.4	41.2	0.30
	40.0	66.0	15.2	40.6	45.1	0.27
CPT-FJR-103 GW = 12.5 ft	37.7	63.0	10.9	37.3	65.0	0.19

Rigidity Index = 250, Cross Sectional Area = 10 cm², Pore Pressure Location = U_2

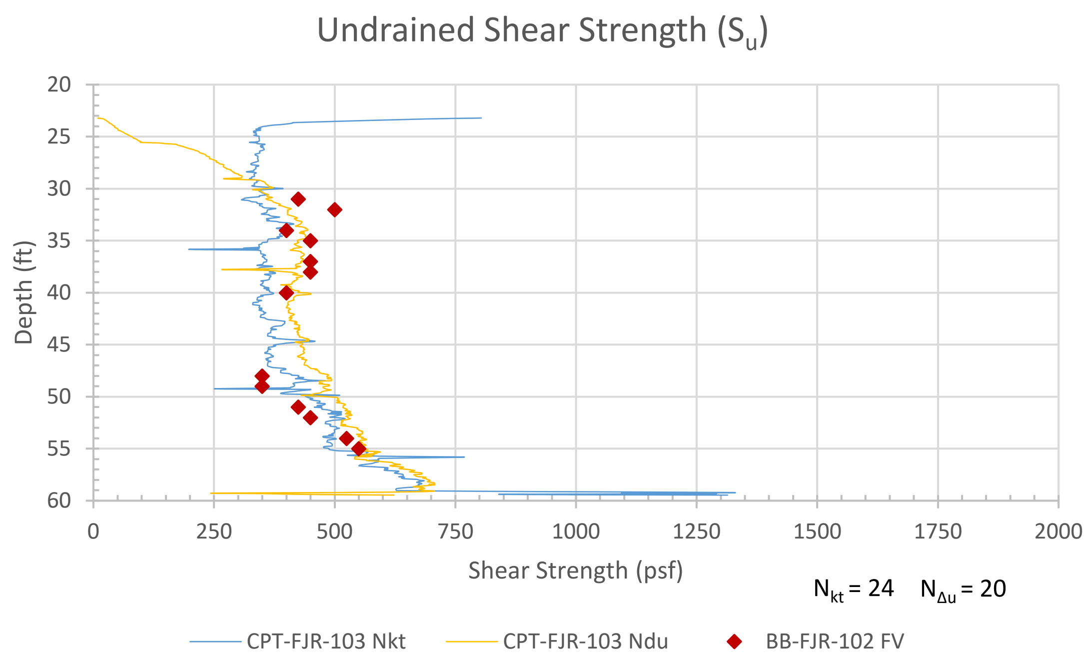
Groundwater was measured within open boreholes upon completion of explorations as follows:

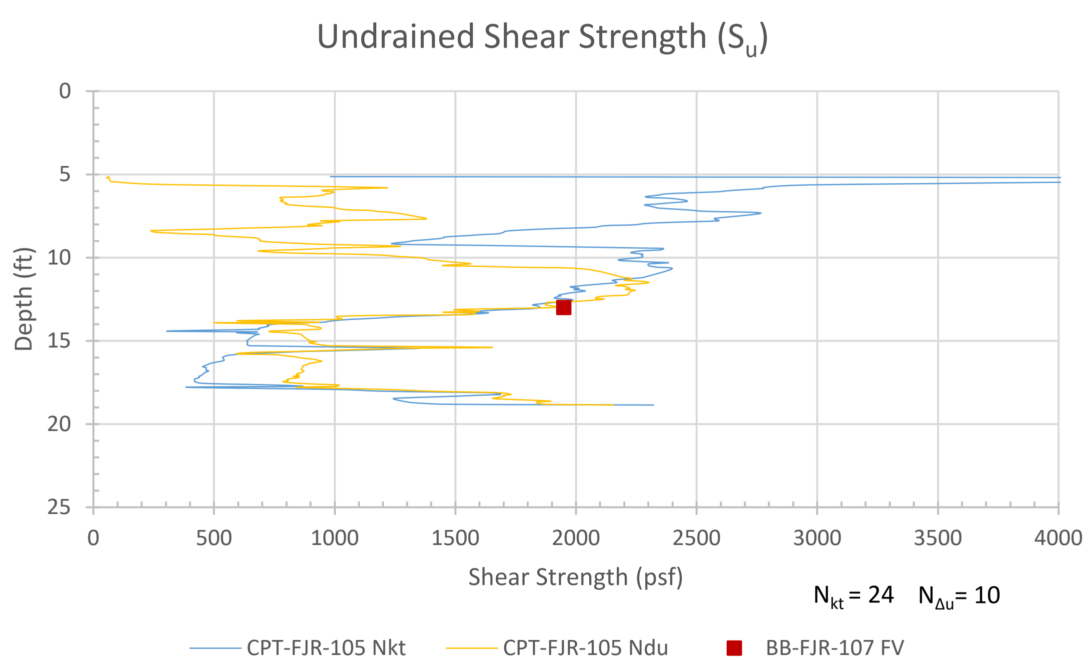
GROUNDWATER SUMMARY	
Location	**Depth (ft)**
CPT-FJR-101	10.2
CPT-FJR-102	5.0
CPT-FJR-103	12.5
CPT-FJR-104	15.0
CPT-FJR-105	7.0
BB-FJR-102	12.3
BB-FJR-107	7.0

Field Vane Shear Tests

The undrained shear strength (S_u) for soft clay is estimated from cone penetration resistance using N_{kt} and $N_{\Delta u}$ correction factors and field vane shear tests performed at adjacent test borings. Field vane shear tests were conducted at BB-FJR-102 from a depth of 31 to 55 feet below ground surface and at BB-FJR-107 at depth of 13 feet. Results of the field vane shear tests are shown on the test boring logs included at the end of this report.

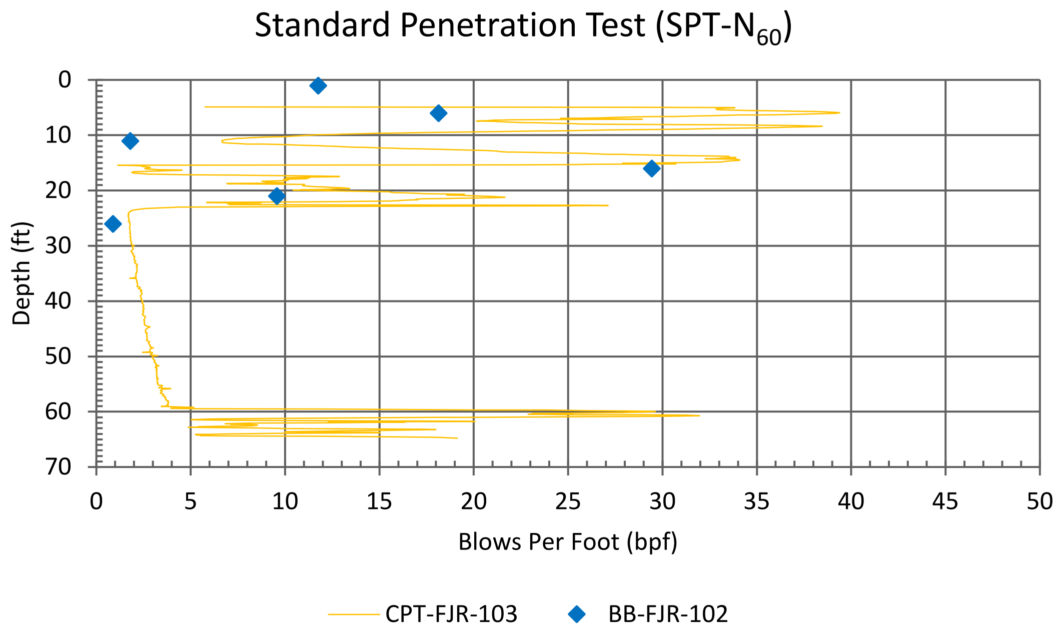
Results indicate an average undrained shear strength of approximately 420 psf with a range from 350 to 700 psf at CPT-FJR-103/BB-FJR-102. Results indicate an undrained shear strength of 1,950 psf at BB-FJR-107 at depth of 13 feet below ground surface. Spikes typically represent silt or sand seams or layers. Graphic results of undrained shear strength (S_u) for soft clay are presented below:

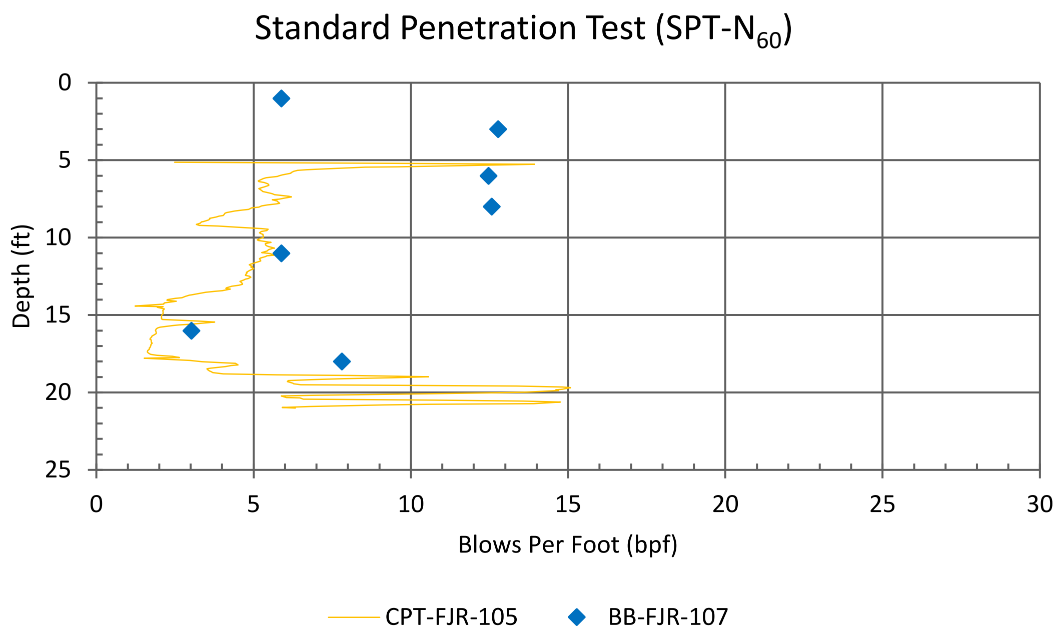




Standard Penetration Tests

Standard penetration tests (SPT-N_{60}) were performed for test borings BB-FJR-102 and BB-FJR-107 using a 24-inch split spoon sampler with an auto drop hammer (140 lbs). The auto drop hammer energy transfer rate is approximately 88 percent. Graphic representation of corrected SPT-N_{60} values for CPT-FJR-103/BB-FJR-102 and CPT-FJR-105/BB-FJR-107 are presented below:





Sand to Silty Clay (Boring BB-FJR-107, 2'-4')

145 Lisbon Street (PO Box 7216) Lewiston, Maine 04243 | (207) 576-3313
173 Pleasant Street Rockland, Maine 04841 | (207) 318-7761
www.summitgeoeng.com

Closure

Evaluation for soil profiling, shear wave velocity, coefficient of consolidation, undrained shear strength, and standard penetration tests is based on professional judgment and generally accepted principles of geotechnical engineering. Changes in subsurface conditions from those presented in this report may occur between the exploration locations.

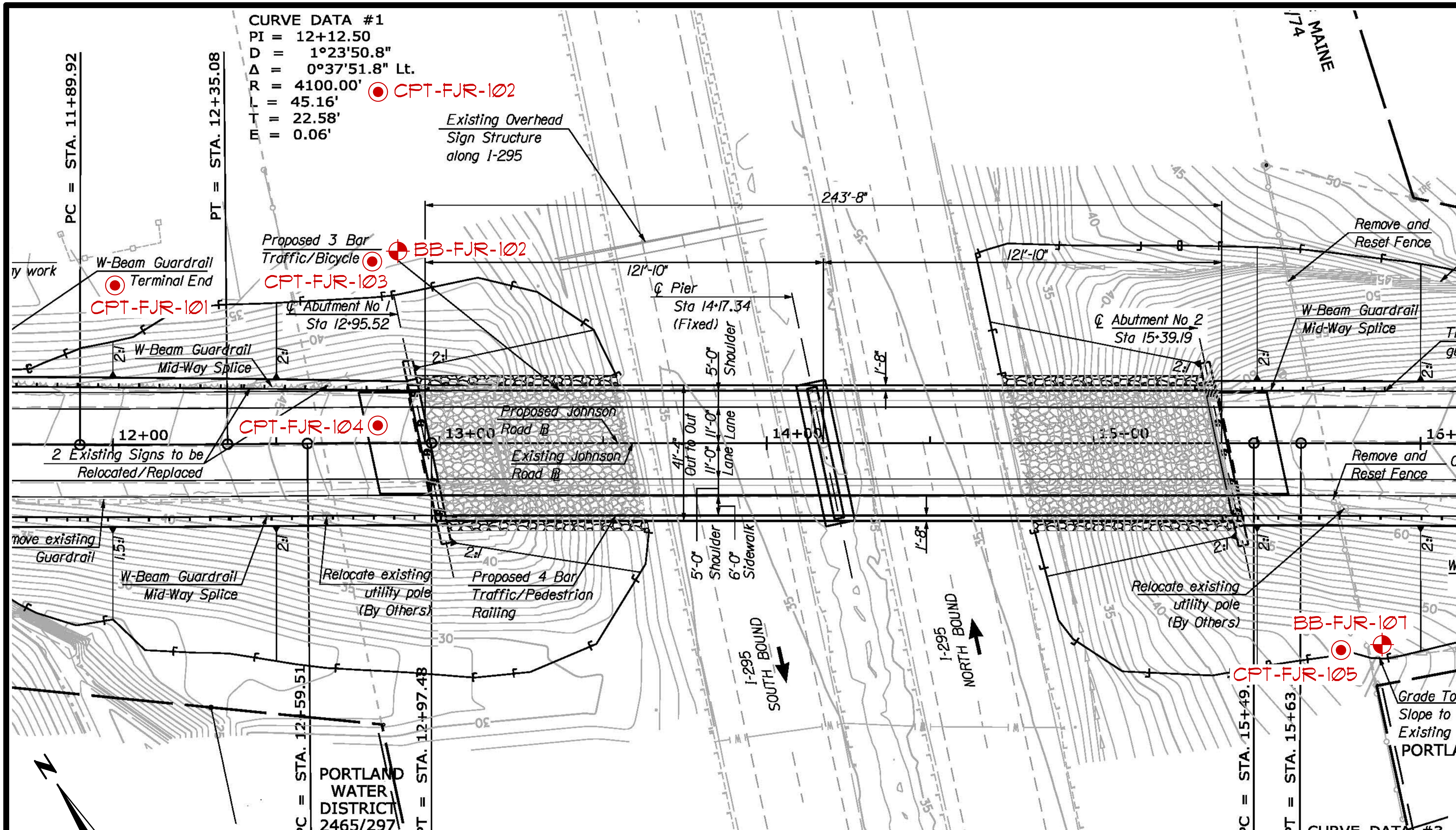
We appreciate the opportunity to serve you during this phase of your project. If there are any questions or additional information is required, please do not hesitate to call.

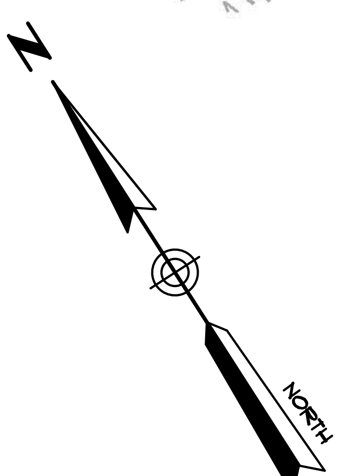
Sincerely yours,
Summit Geoengineering Services

Craig W. Coolidge, P.E.
Vice President
Principal Engineer

Attachments: EXPLORATION LOCATION PLAN, LOGS, WATERFALL PLOTS, DISSIPATION GRAPHS

145 Lisbon Street (PO Box 7216) Lewiston, Maine 04243 | (207) 576-3313
173 Pleasant Street Rockland, Maine 04841 | (207) 318-7761
www.summitgeoeng.com





LEGEND

BB-FBR-105	SUMMIT TEST BORING WITH FIELD VANES (MAY 21-23, 2019)
CPT-FBR-106	SUMMIT CONE PENETRATION TEST (MAY 21-23, 2019)

PLAN REFERENCE

"JOHNSON ROAD BRIDGE, INTERSTATE 295, PRELIMINARY PLAN", DATED DECEMBER 2018, PREPARED BY STATE OF MAINE, DEPARTMENT OF TRANSPORTATION".

SCALE: 1" = 30'

0 15 30 60 90

PROJECT: MAINE D.O.T. BRIDGE PROJECT
JOHNSON RD - INTERSTATE 295 - FALMOUTH, ME

CLIENT: GZA GEOENVIRONMENTAL, INC.

TITLE: EXPLORATION LOCATION PLAN

DRAWN BY: KRF

APPR. BY: CWC

SCALE: 1" = 30'

DATE: JUNE 5, 2019

SUMMIT GEOENGINEERING SERVICES

145 LISBON ST. - SUITE 101
LEWISTON, ME 04240
Tel: (207) 576-3313

113 PLEASANT STREET
ROCKLAND, ME 04841
Tel: (207) 318-7761

PROJ.#: 19120

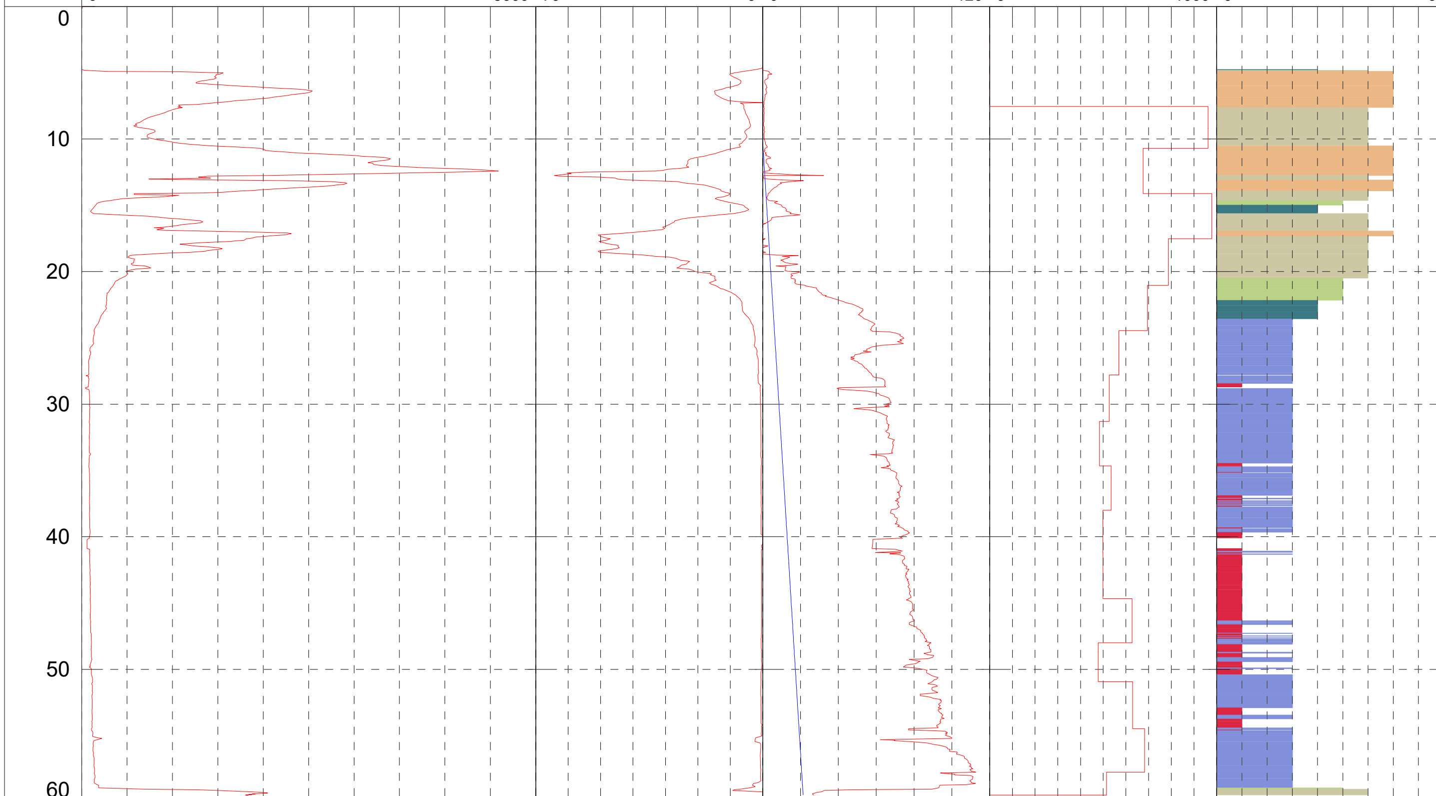
FIGURE: 1

CPT-FJR-101

COMPANY: Summit Geoengineering Services
OPERATOR: C. Coolidge, P.E.
CREW: C. Sullivan, S. Floyd
CLIENT: GZA
CLIENT REP: Blaine Cardali, P.E.

TEST DATE: Wed 29/May/2019
TEST ID: CPT-FJR-101
PROJECT: 19120
SITE: Johnson Road
LOCATION: West Abutment

Depth (ft)	Tip COR (psi) 0–5000	Sleeve Stress (psi) 70–0	Pore Pressure (psi) WT: 10.20(ft) 0–120	Seismic Velocity (ft/s) 0–1000	Class. PP (Rob. 1990) 0–9	REMARKS



REMARKS:
Start of CPT at 5 Ft
Groundwater at 10.2 Ft
Dissipation at 28.7 Ft
Push Refusal at 59.5 Ft

FINAL BASELINE: -9.32 (psi) FINAL BASELINE: -0.1419 (psi) FINAL BASELINE: 0.573 (psi)

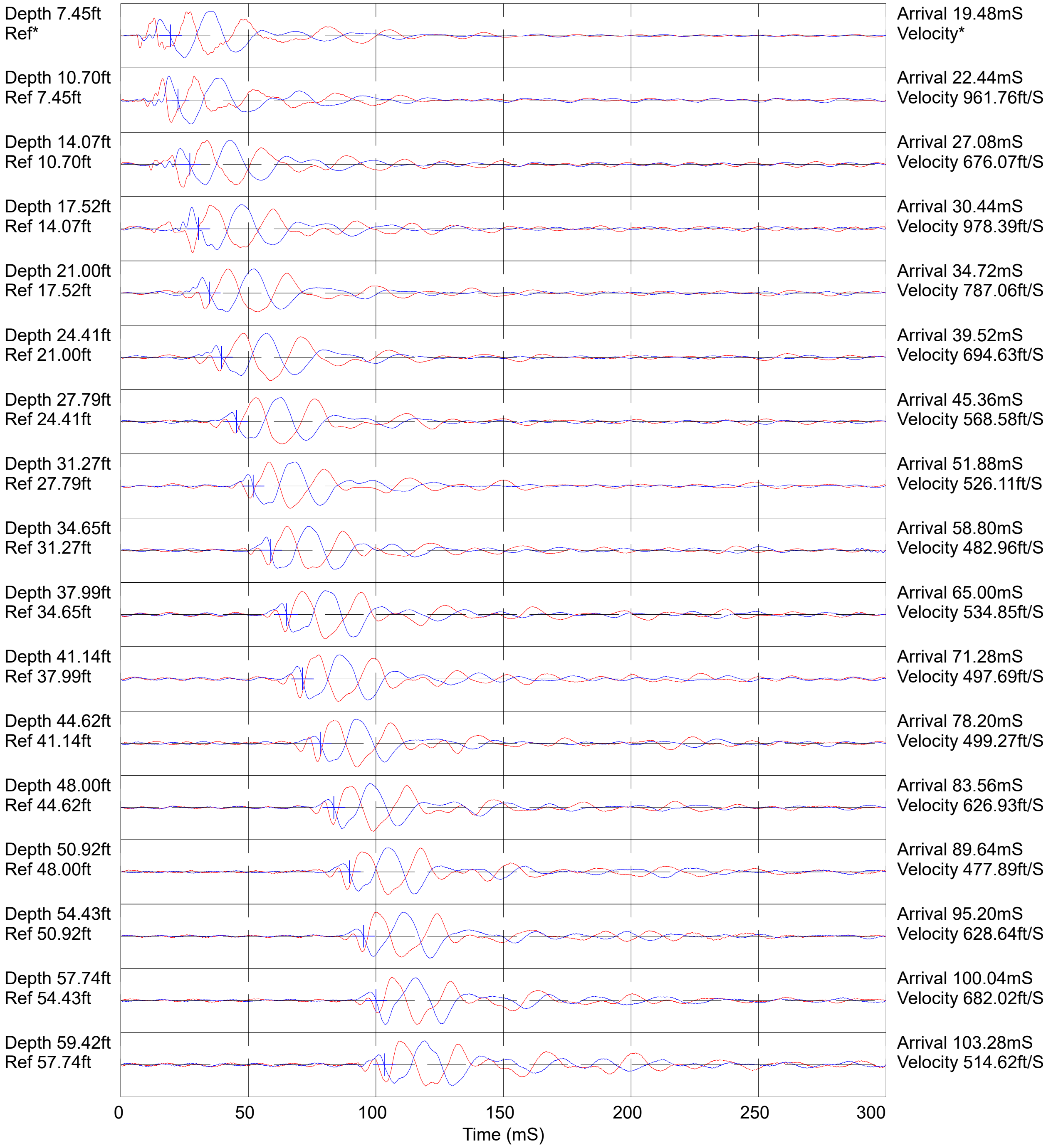
TOTAL DEPTH: 59.476 ft

PROBE ID: 4544.104.A

1 Sensitive, fine grained
2 Organic soils - peats
3 Clays - clay to silty clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to sand
8 Very stiff sand to clayey sand **
9 Very stiff, fine grained **

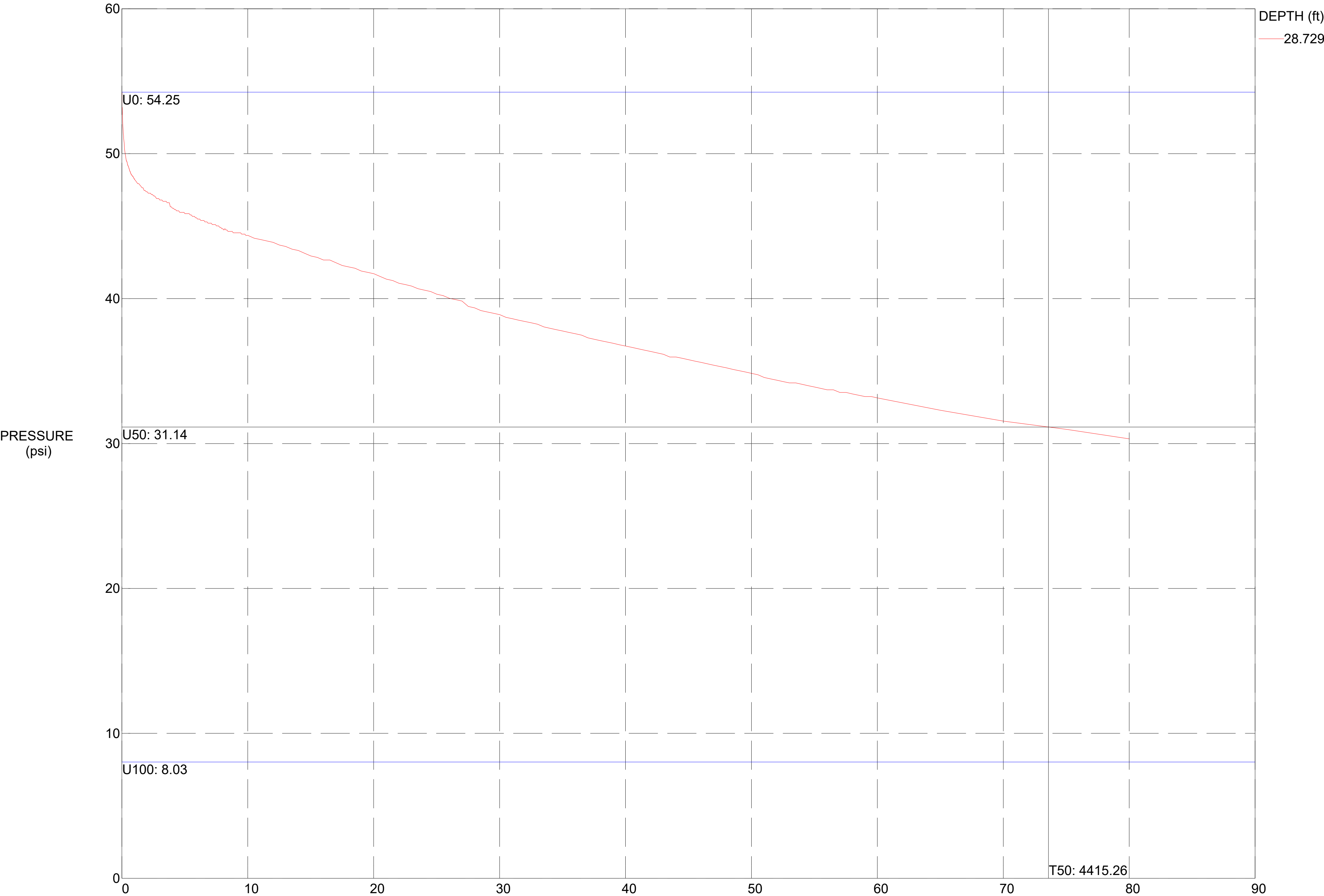
*SBT: Robertson 1990; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

TEST ID: CPT-FJR-101



Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A

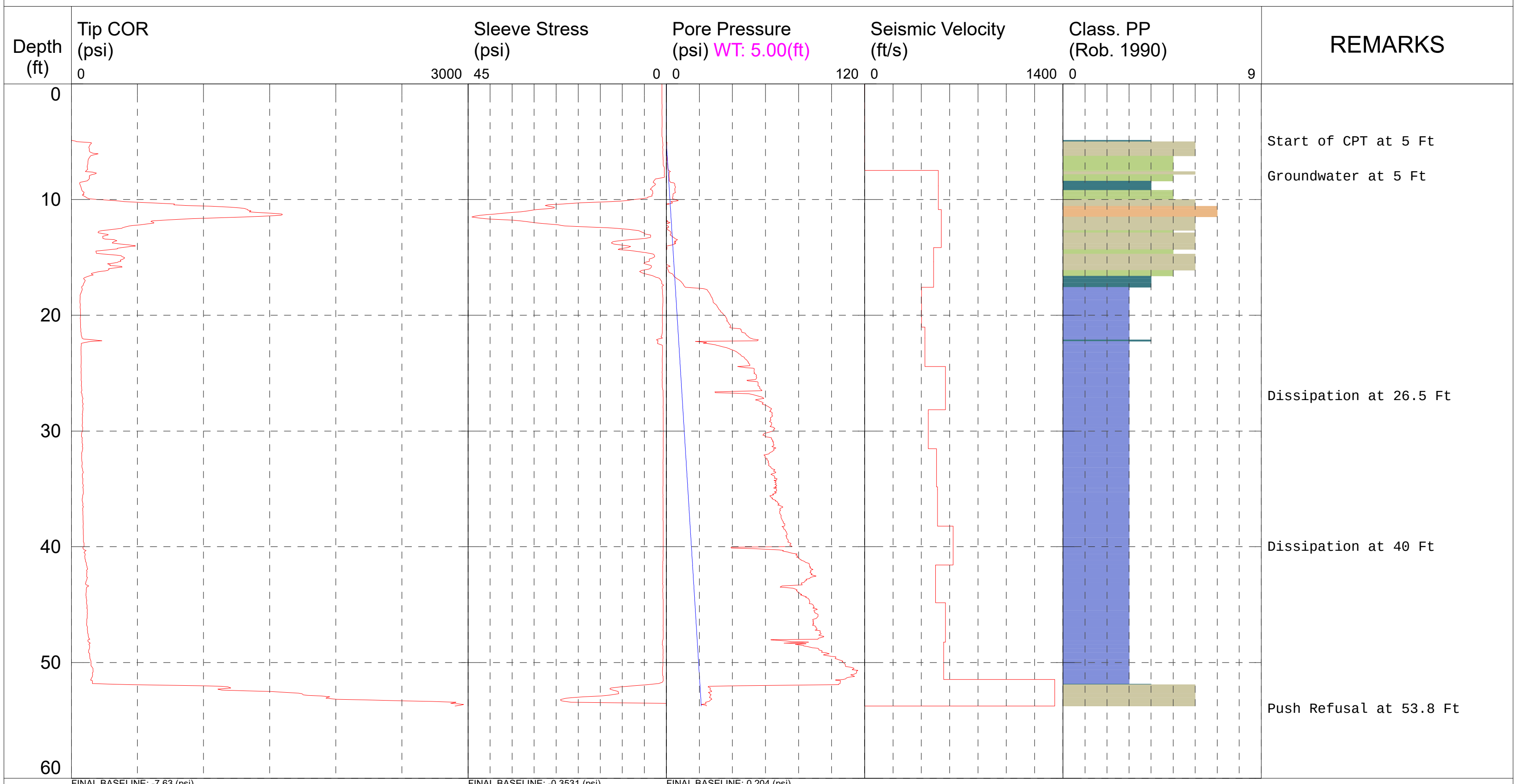


MAXIMUM PRESSURE = 54.246 (psi)
HYDROSTATIC PRESSURE = 8.032 (psi), WATER TABLE: 10.20 ft

CPT-FJR-102

COMPANY: Summit Geoengineering Services
OPERATOR: C. Coolidge, P.E.
CREW: C. Sullivan, S. Floyd
CLIENT: GZA
CLIENT REP: Blaine Cardali, P.E.

TEST DATE: Wed 29/May/2019
TEST ID: CPT-FJR-102
PROJECT: 19120
SITE: Johnson Road
LOCATION: West Abutment



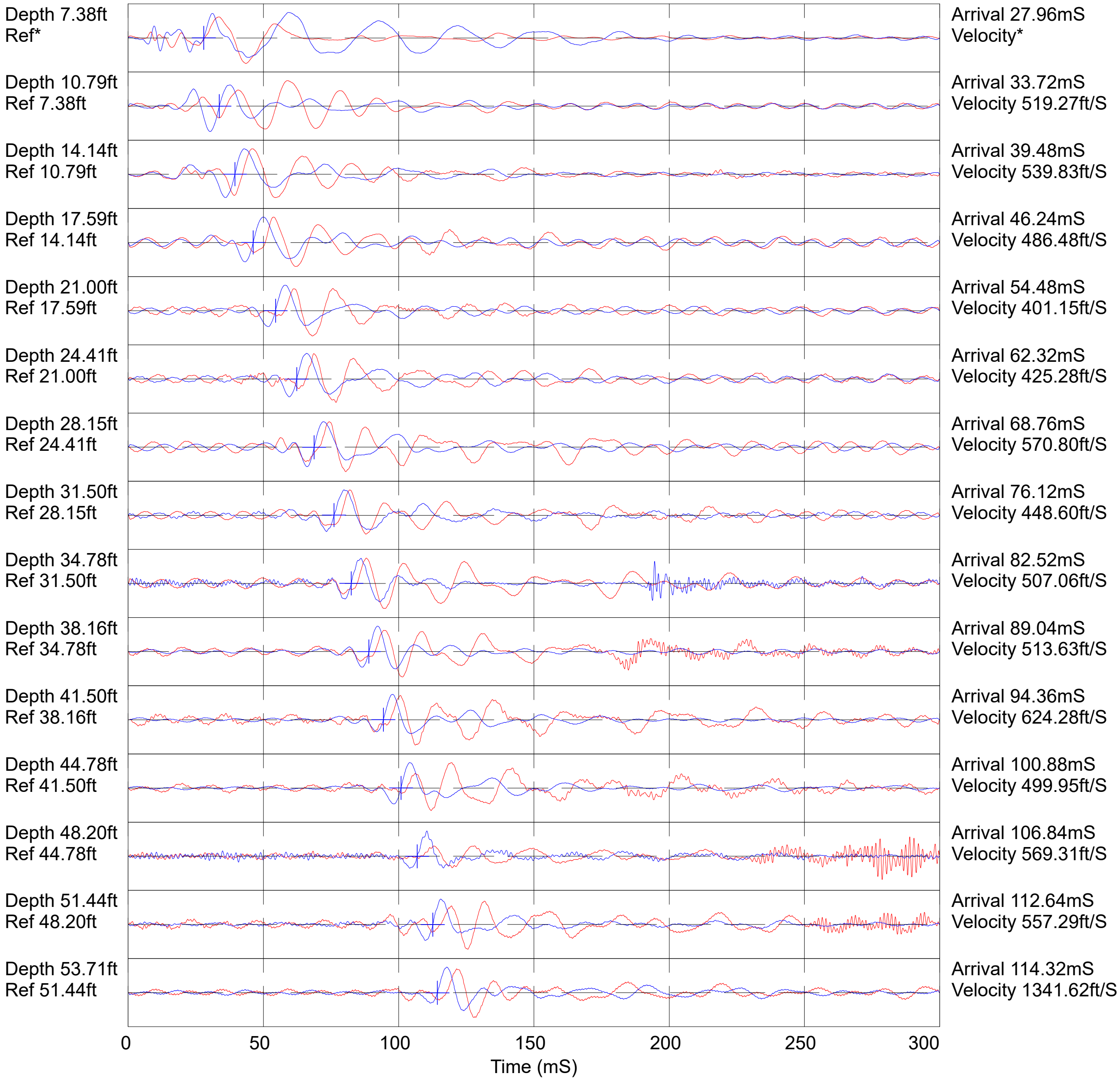
TOTAL DEPTH: 53.761 ft

PROBE ID: 4544.104.A

1 Sensitive, fine grained
2 Organic soils - peats
3 Clays - clay to silty clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to sand
8 Very stiff sand to clayey sand **
9 Very stiff, fine grained **

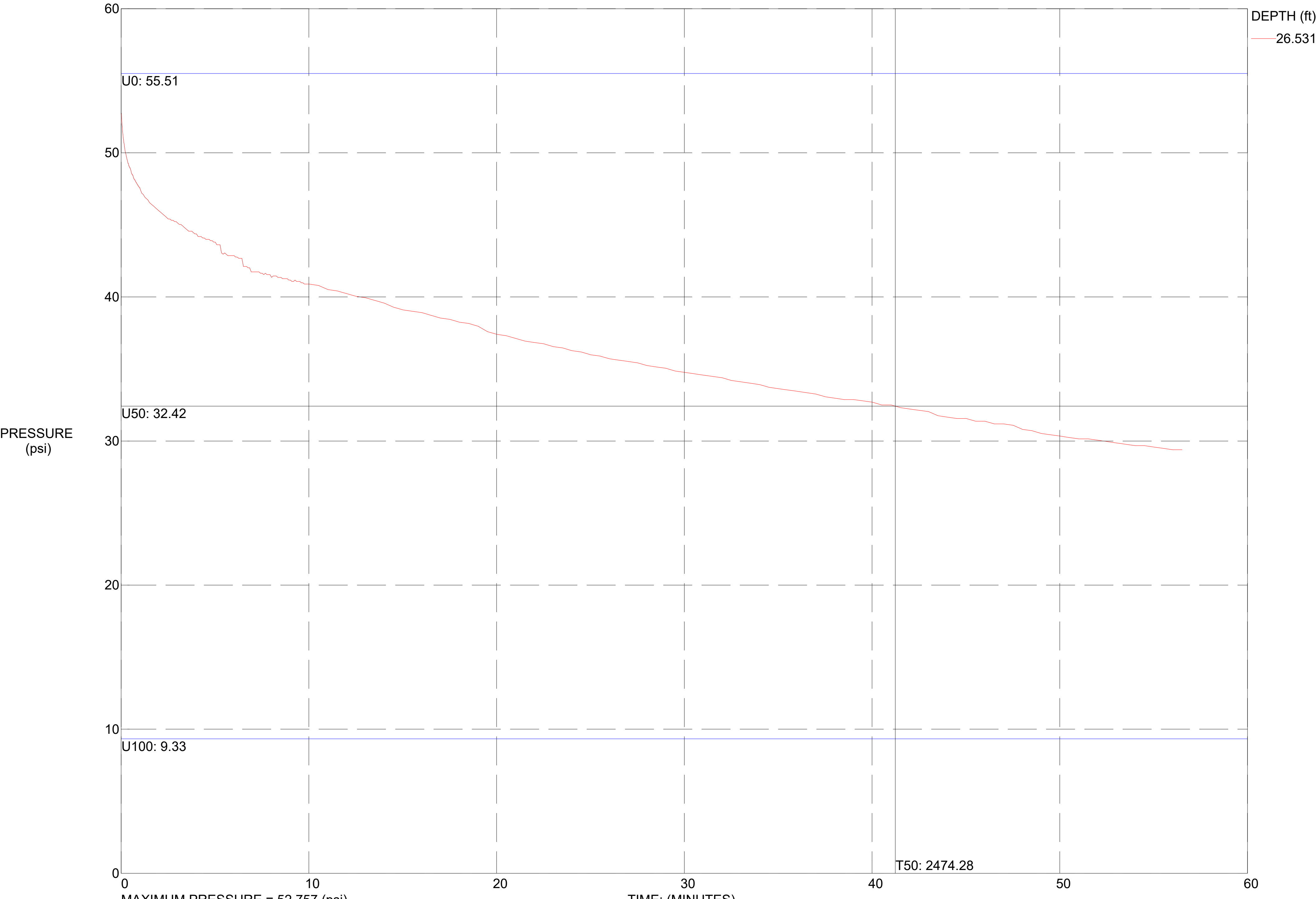
*SBT: Robertson 1990; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

TEST ID: CPT-FJR-102

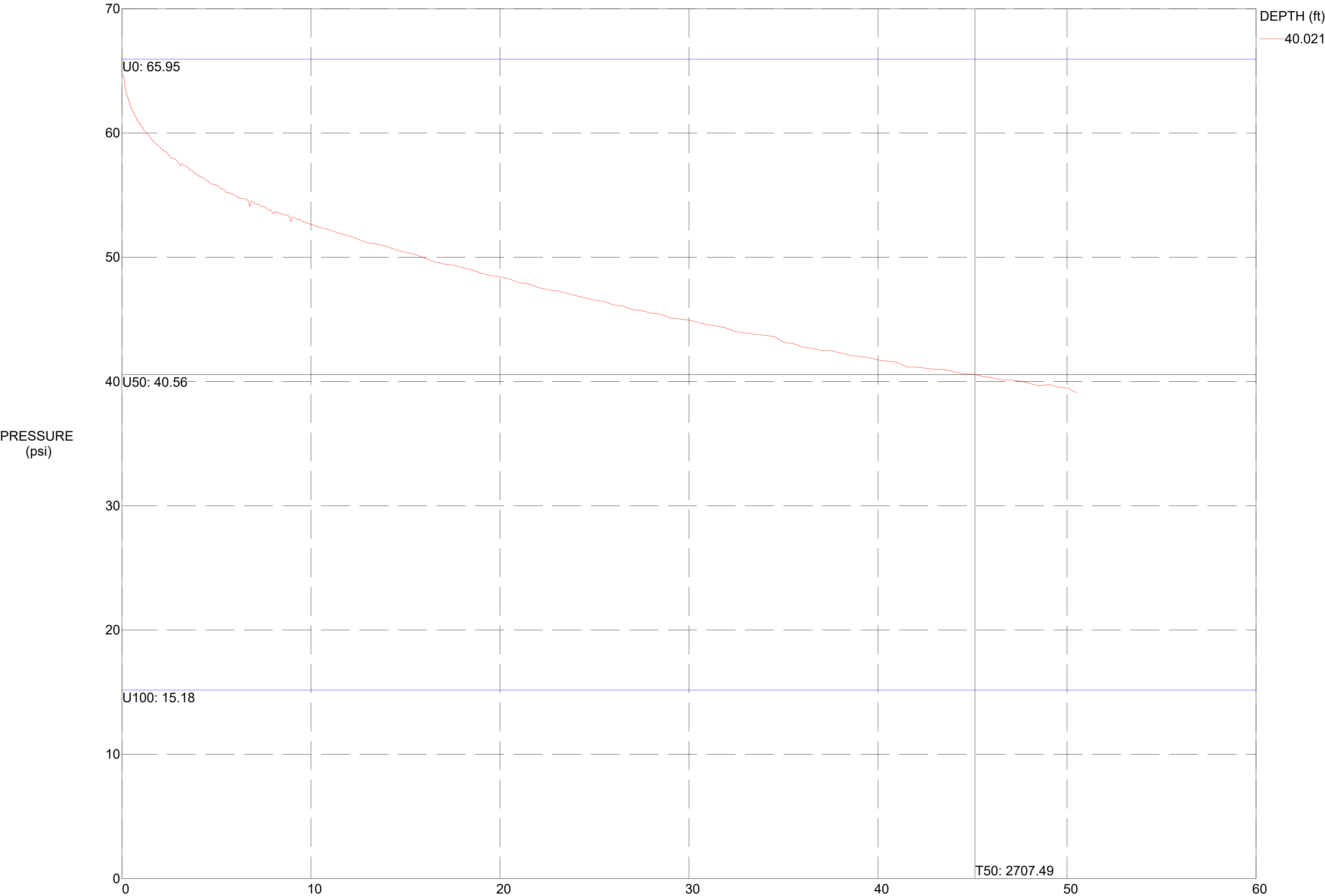


Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A



MAXIMUM PRESSURE = 52.757 (psi)
HYDROSTATIC PRESSURE = 9.334 (psi), WATER TABLE: 5.00 ft

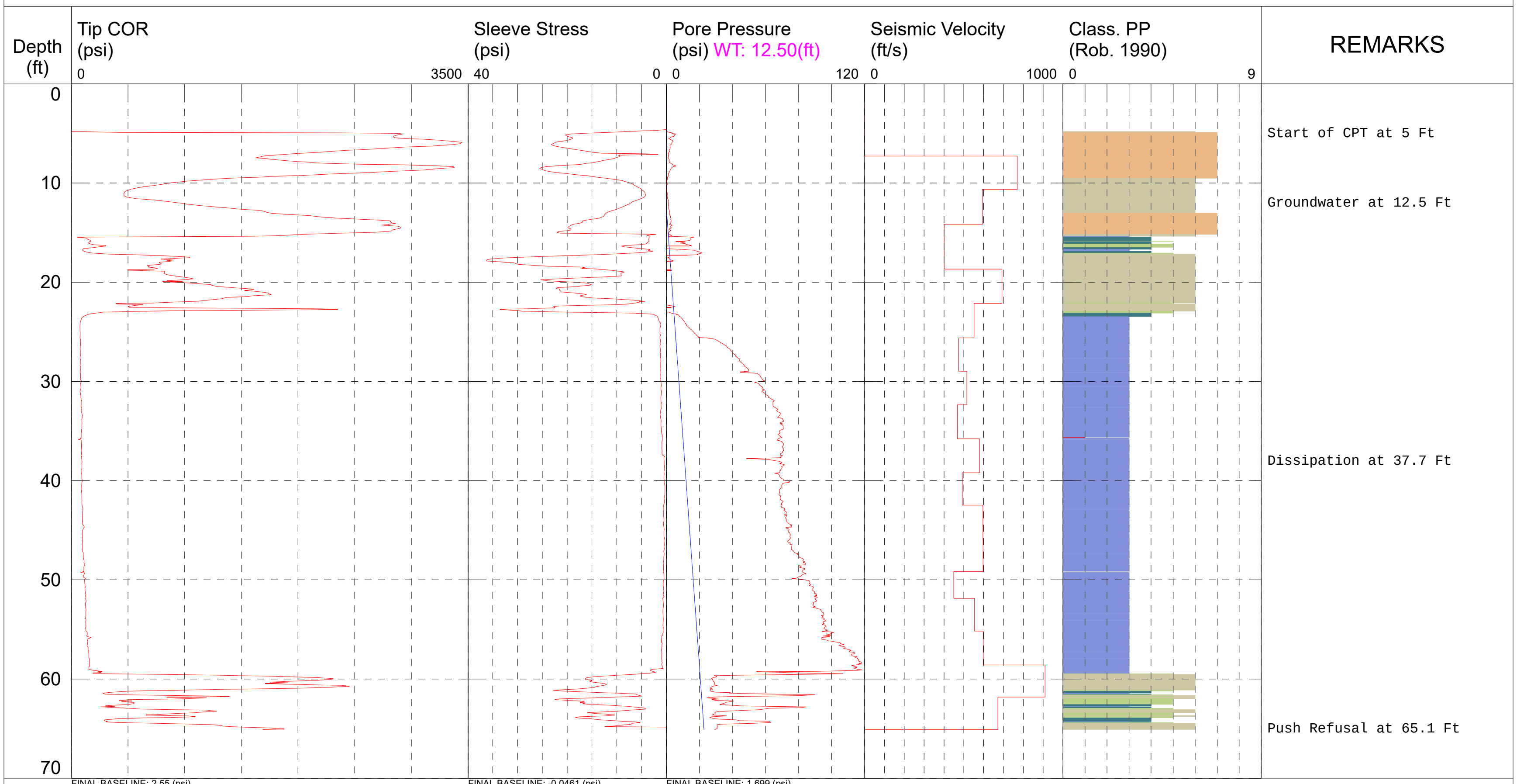


MAXIMUM PRESSURE = 65.946 (psi)
HYDROSTATIC PRESSURE = 15.182 (psi), WATER TABLE: 5.00 ft

CPT-FJR-103

COMPANY: Summit Geoengineering Services
OPERATOR: C. Coolidge, P.E.
CREW: C. Sullivan, S. Floyd
CLIENT: GZA
CLIENT REP: Blaine Cardali, P.E.

TEST DATE: Thu 30/May/2019
TEST ID: CPT-FJR-103
PROJECT: 19120
SITE: Johnson Road
LOCATION: West Abutment



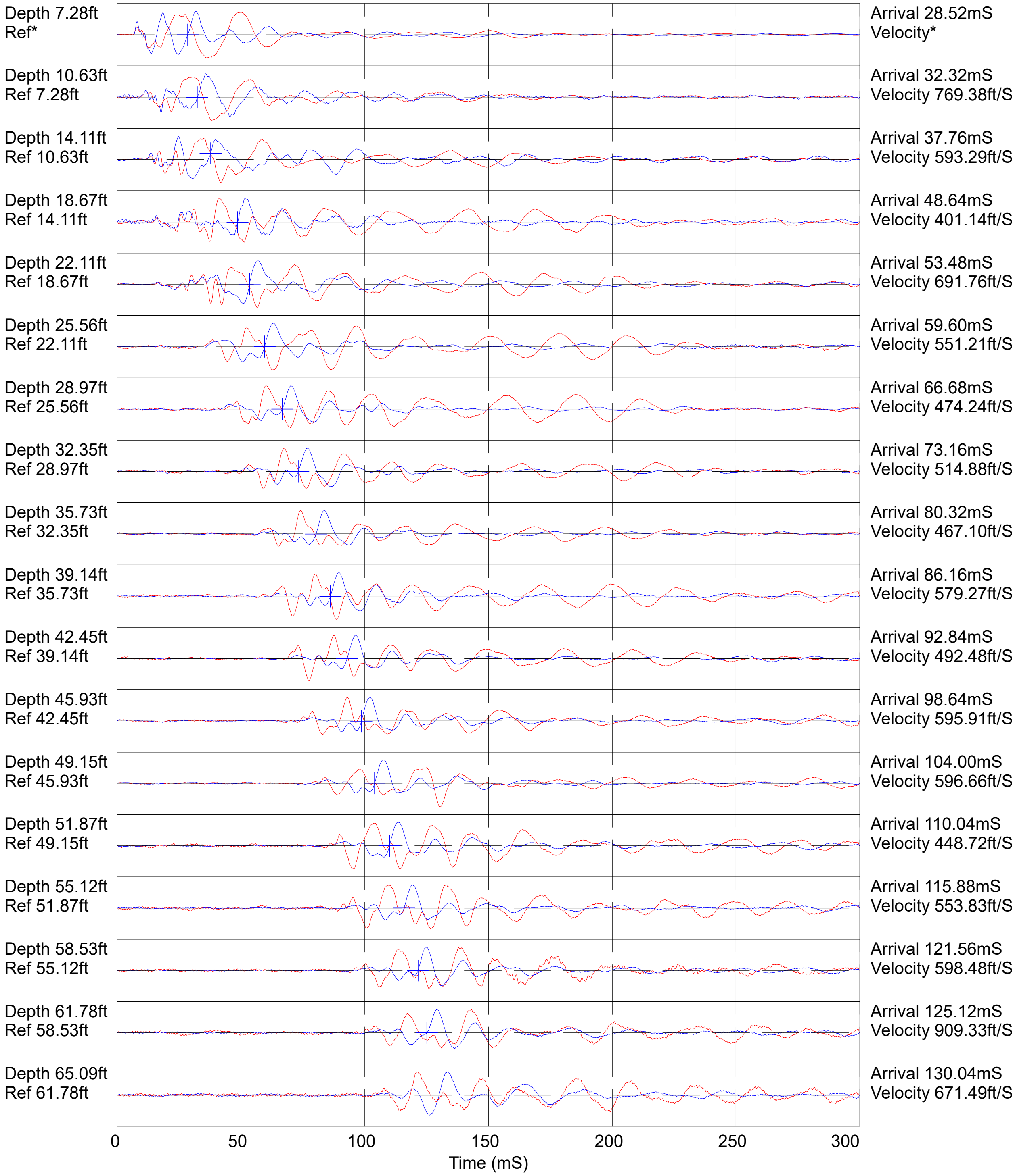
TOTAL DEPTH: 65.096 ft

PROBE ID: 4544.104.A

1 Sensitive, fine grained	4 Silt mixtures - clayey silt to silty clay	7 Gravelly sand to sand
2 Organic soils - peats	5 Sand mixtures - silty sand to sandy silt	8 Very stiff sand to clayey sand **
3 Clays - clay to silty clay	6 Sands - clean sand to silty sand	9 Very stiff, fine grained **

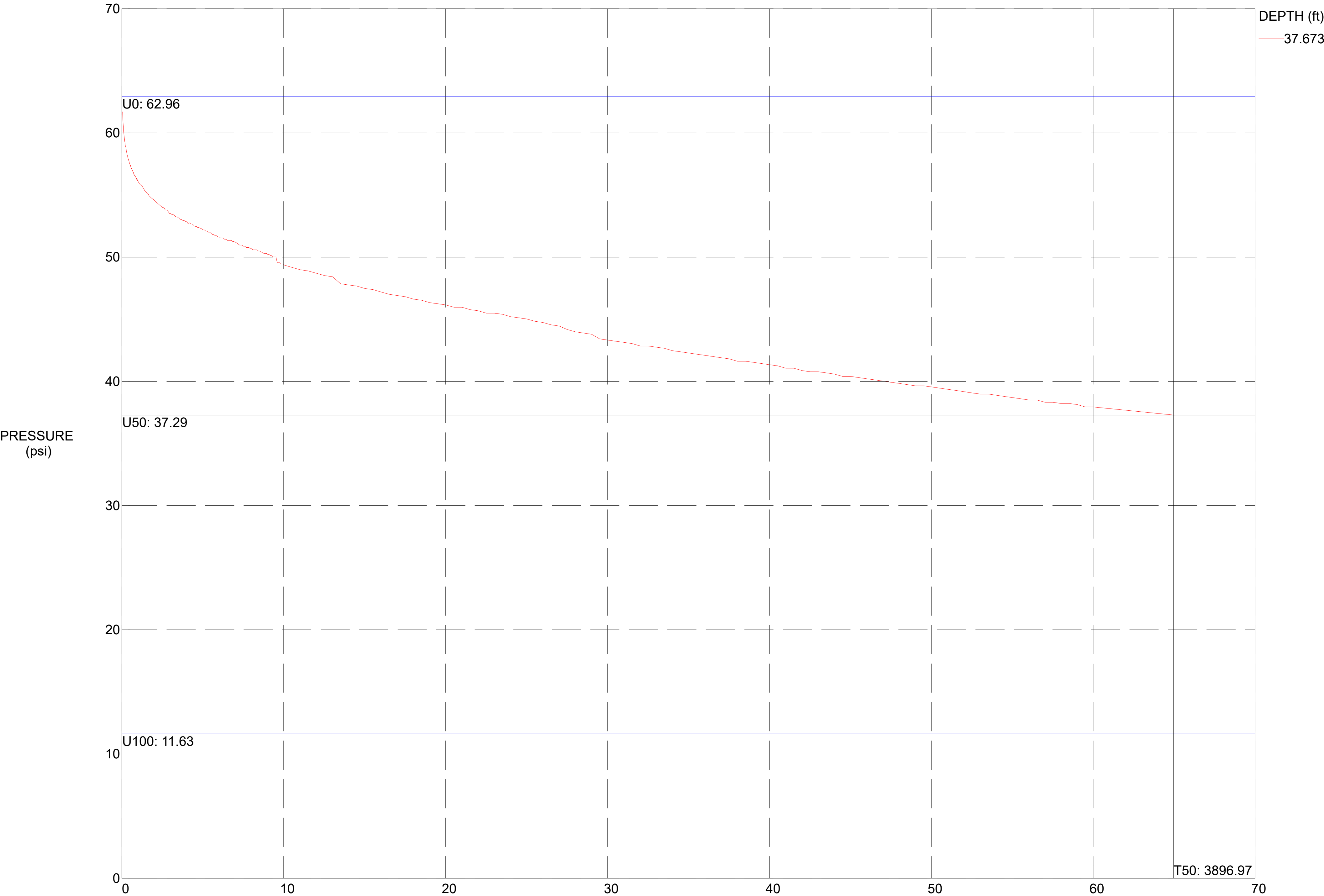
*SBT: Robertson 1990; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

TEST ID: CPT-FJR-103



Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A

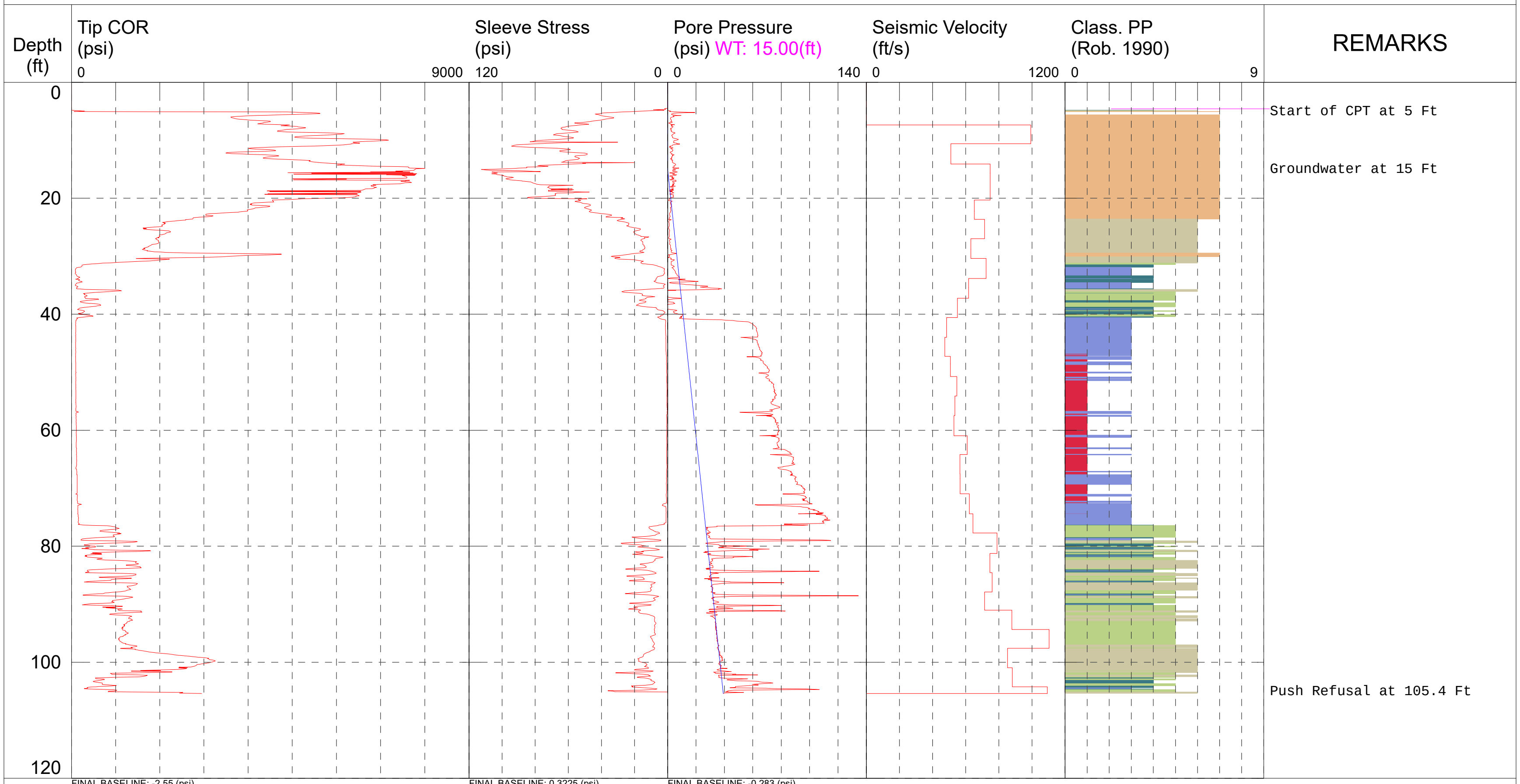


MAXIMUM PRESSURE = 62.96 (psi)
HYDROSTATIC PRESSURE = 10.913 (psi), WATER TABLE: 12.50 ft

CPT-FJR-104

COMPANY: Summit Geoengineering Services
OPERATOR: C. Coolidge, P.E.
CREW: C. Sullivan
CLIENT: GZA
CLIENT REP: Blaine Cardali, P.E.

TEST DATE: Mon 03/Jun/2019
TEST ID: CPT-FJR-104
PROJECT: 19120
SITE: Johnson Road
LOCATION: West Abutment



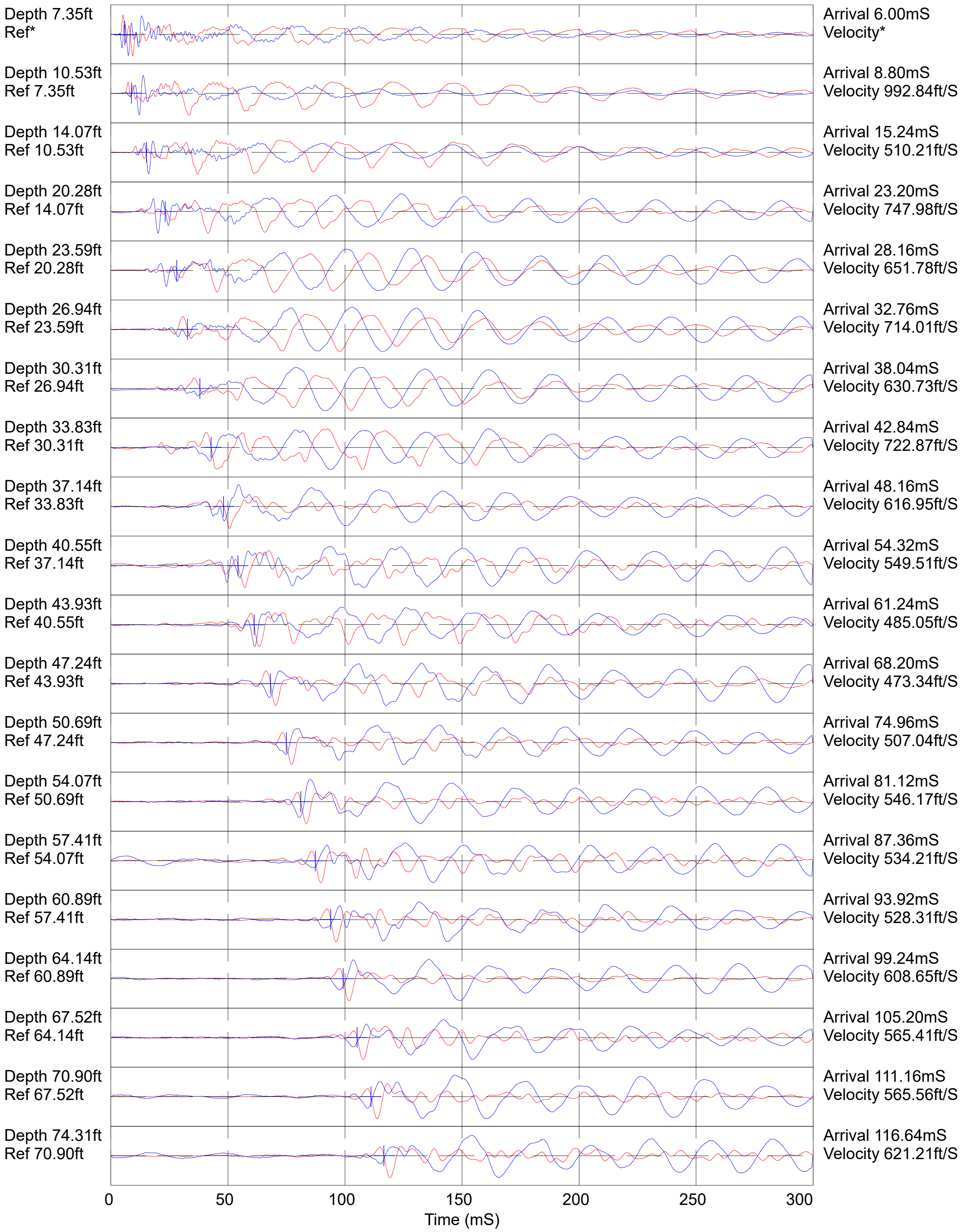
TOTAL DEPTH: 105.389 ft

PROBE ID: 4544.104.A

1 Sensitive, fine grained	4 Silt mixtures - clayey silt to silty clay	7 Gravelly sand to sand
2 Organic soils - peats	5 Sand mixtures - silty sand to sandy silt	8 Very stiff sand to clayey sand **
3 Clays - clay to silty clay	6 Sands - clean sand to silty sand	9 Very stiff, fine grained **

*SBT: Robertson 1990; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

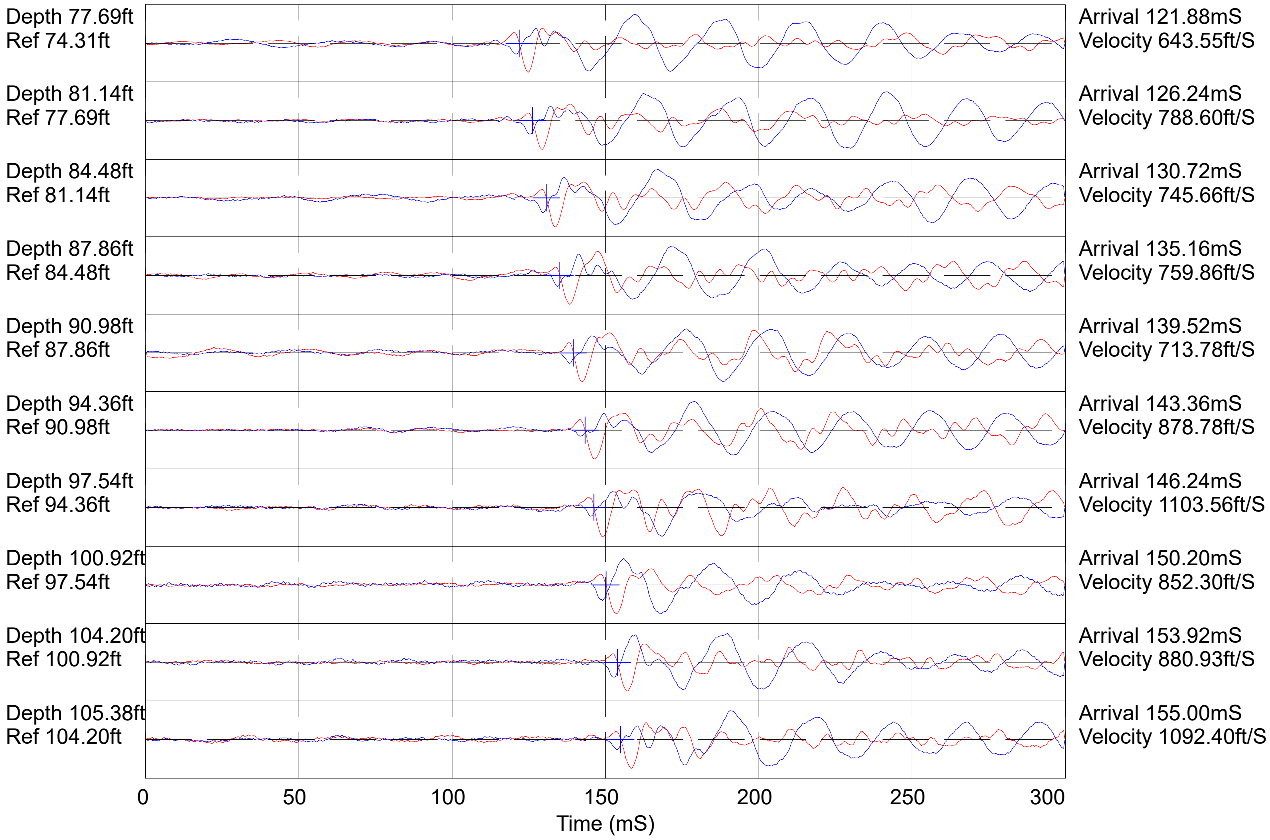
TEST ID: CPT-FJR-104



Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A

TEST ID: CPT-FJR-104



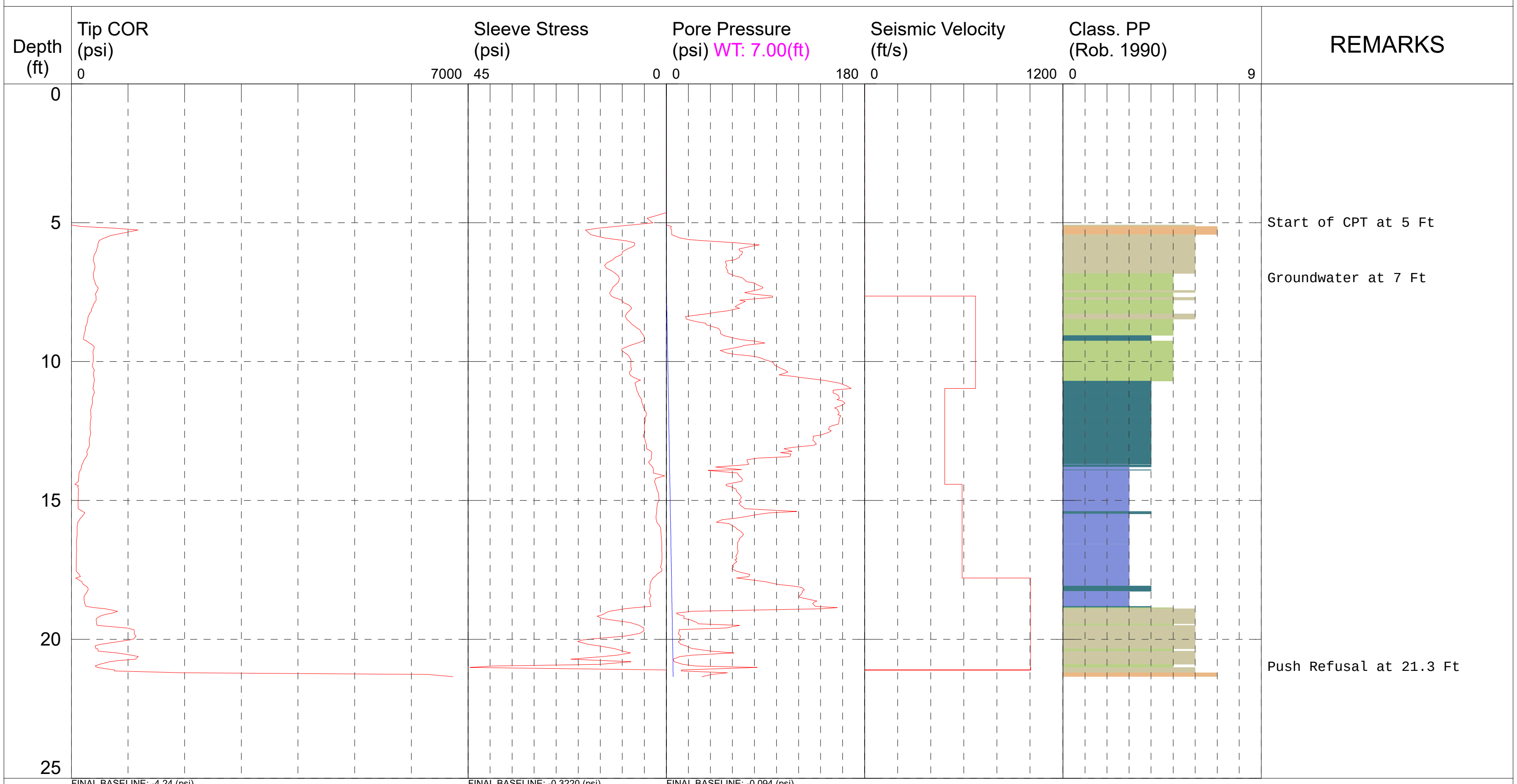
Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A

CPT-FJR-105

COMPANY: Summit Geoengineering Services
OPERATOR: C. Coolidge, P.E.
CREW: C. Sullivan, S. Floyd
CLIENT: GZA
CLIENT REP: Blaine Cardali, P.E.

TEST DATE: Tue 28/May/2019
TEST ID: CPT-FJR-105
PROJECT: 19120
SITE: Johnson Road
LOCATION: East Abutment



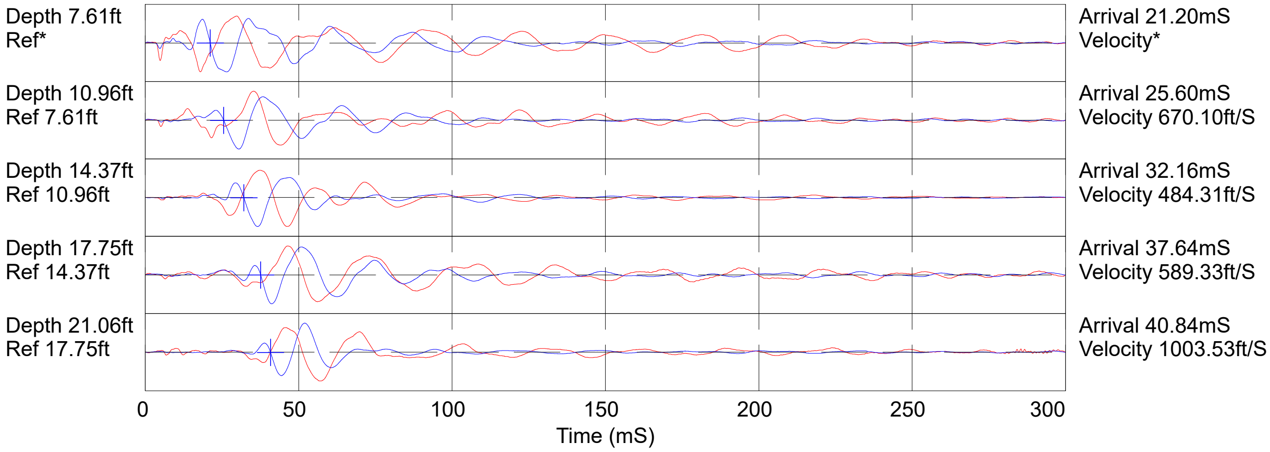
TOTAL DEPTH: 21.344 ft

PROBE ID: 4544.104.A

1	Sensitive, fine grained
2	Organic soils - peats
3	Clays - clay to silty clay
4	Silt mixtures - clayey silt to silty clay
5	Sand mixtures - silty sand to sandy silt
6	Sands - clean sand to silty sand
7	Gravelly sand to sand
8	Very stiff sand to clayey sand **
9	Very stiff, fine grained **

*SBT: Robertson 1990; **Overconsolidated or Cemented; *SBT/SPT CORRELATION: UBC-1983

TEST ID: CPT-FJR-105



Hammer to Rod String Distance (ft): 4.92
* = Not Determined

PROBE ID: 4544.104.A

SUMMIT GEOENGINEERING SERVICES

SOIL BORING LOG

Boring #: BB-FJR-102

Project:	Bridge Replacement	Project #: 19120	
Location:	Johnson Rd.	Sheet: 1 of 3	
City, State:	Falmouth, Maine	Chkd by: ELS	
Drilling Co:	Summit Geoengineering Services, Inc.	Boring Elevation:	36 ft +/-
Driller:	Craig Coolidge, P.E.	Reference:	Preliminary Plan, Sheet 2, Johnson Rd Bridge Provided by MDOT
Summit Staff:	Colleen Sullivan	Date started: 5/30/2019	Date Completed: 5/30/2019

DRILLING METHOD		SAMPLER	
Vehicle:	AMS	Length:	24" SS
Model:	9500 VTR	Diameter:	2"OD/1.5"ID
Method:	3" Casing	Hammer:	140 lb
Hammer Style:	Auto	Method:	ASTM D1586

ESTIMATED GROUND WATER DEPTH

Date	Depth	Elevation	Reference
5/30/2019	12.3 ft	N/A	Measured in open borehole after test

Depth (ft.)	No.	Pen/Rec (in)	Depth (ft)	blows/6"	Elev. (ft.)	SAMPLE DESCRIPTION	Geological/ Test Data	Geological Stratum
	S-1	24/12	0 - 2	1		Soft, dark brown, SILT, little Sand and rootlets,		
1				2		trace Clay, damp		TOPSOIL
				4	35.5' +/-	Loose, brown, fine to medium SAND, little Silt, damp,		0.5'
2				4		SP-SM		FILL
3								
4								
5								
	S-2	24/9	5 - 7	10		Compact, light brown, fine to medium SAND, little Silt,		
6				8		damp, SP-SM		
				8				
7				7				
8								
9								
10								
	S-3	24/12	10 - 12	2		Very loose, light brown, fine to medium SAND,		
11				1		little Silt, occasional mottling, moist, SP-SM		
				1				
12				2				
13								
14								
15								
	S-4	24/18	15 - 17	17		Dense, olive brown, fine SAND, with Silt and Clay		
16				18		lenses, moist to wet, SP-SM		
				14				
17				9				
					19' +/-			17' +/-
18								GLACIAL MARINE DEPOSIT
19								
20								
	S-5	24/24	20 - 22	3		Stiff, olive brown, SILT & CLAY, slight mottling,		
21				5		trace Organics, wet, CL		
				5				
22				5				

BURMISTER SOIL CLASSIFICATION

Granular Soils		Cohesive Soils		% Composition
Blows/ft.	Density	Blows/ft.	Consistency	Description
0-4	V. Loose	<2	V. soft	0-10% trace
5-10	Loose	2-4	Soft	10-20% little
11-30	Compact	5-8	Firm	20-35% some
31-50	Dense	9-15	Stiff	35-50% and
>50	V. Dense	16-30	V. Stiff	
		>30	Hard	

NOTES: PP = Pocket Penetrometer, MC = Moisture Content, S = Split spoon sample, LL = Liquid Limit, PI = Plasticity Index, UT = Shelby Tube Sample, S_U = Undrained Shear Strength, $S_{U(r)}$ = Remolded Shear Strength, WOH = Weight of Hammer

Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches
Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200

Soil Moisture Condition
Dry: S = 0%
Humid: S = 1 to 25%
Damp: S = 26 to 50%
Moist: S = 51 to 75%
Wet: S = 76 to 99%
Saturated: S = 100%

SUMMIT GEOENGINEERING SERVICES

SOIL BORING LOG

Boring #:	BB-FJR-102		
Project:	Bridge Replacement	Project #: 19120	
Location:	Johnson Rd.	Sheet: 2 of 3	
City, State:	Falmouth, Maine	Chkd by: ELS	
Drilling Co:	Summit Geoengineering Services, Inc.	Boring Elevation	36 ft +/-
Driller:	Craig Coolidge, P.E.	Reference:	Preliminary Plan, Sheet 2, Johnson Rd Bridge Provided by MDOT
Summit Staff:	Colleen Sullivan	Date started: 5/30/2019	Date Completed: 5/30/2019

DRILLING METHOD		SAMPLER	
Vehicle:	AMS	Length:	24" SS
Model:	9500 VTR	Diameter:	2"OD/1.5"ID
Method:	3" Casing	Hammer:	140 lb
Hammer Style:	Auto	Method:	ASTM D1586

ESTIMATED GROUND WATER DEPTH

Date	Depth	Elevation	Reference
5/30/2019	12.3 ft	N/A	Measured in open borehole after test

Depth (ft.)	No.	Pen/Rec (in)	Depth (ft)	blows/6"	Elev. (ft.)	SAMPLE DESCRIPTION	Geological/ Test Data	Geological Stratum
								GLACIAL MARINE DEPOSIT
23								
24								
25								
	S-6	24/24	25 - 27	WOH		Very soft, gray, CLAY & SILT, CL		
26				WOH				
				WOH				
27				WOH				
28								
29								
30	FIELD VANES							
			Tip of Vane					
31	FV-1		31		5' +/-	S_U = 425 psf, $S_{U(r)}$ = 50 psf (8.5 ft-lb, 1 ft-lb)	SMALL VANE↓	
32	FV-2		32		4' +/-	S_U = 500 psf, $S_{U(r)}$ = 50 psf (10 ft-lb, 1 ft-lb)		
33								
34	FV-3		34		2' +/-	S_U = 400 psf, $S_{U(r)}$ = 50 psf (8 ft-lb, 1 ft-lb)		
35	FV-4		35		1' +/-	S_U = 450 psf, $S_{U(r)}$ = 50 psf (9 ft-lb, 1 ft-lb)		
36								
37	FV-5		37		-1' +/-	S_U = 450 psf, $S_{U(r)}$ = 75 psf (9 ft-lb, 1.5 ft-lb)		
38	FV-6		38			S_U = 450 psf, $S_{U(r)}$ = 50 psf (9 ft-lb, 1 ft-lb)		
39								
40	FV-7		40		-4' +/-	S_U = 400 psf, $S_{U(r)}$ = 50 psf (8 ft-lb, 1 ft-lb)		
41								
42								
43								
44								

BURMISTER SOIL CLASSIFICATION

Granular Soils		Cohesive Soils		% Composition
Blows/ft.	Density	Blows/ft.	Consistency	Description
0-4	V. Loose	<2	V. soft	0-10% trace
5-10	Loose	2-4	Soft	10-20% little
11-30	Compact	5-8	Firm	20-35% some
31-50	Dense	9-15	Stiff	35-50% and
>50	V. Dense	16-30	V. Stiff	
		>30	Hard	

NOTES: PP = Pocket Penetrometer, MC = Moisture Content, S = Split spoon sample, LL = Liquid Limit, PI = Plasticity Index, UT = Shelby Tube Sample, S_U = Undrained Shear Strength, $S_{U(r)}$ = Remolded Shear Strength, WOH = Weight of Hammer

Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches
Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200

Soil Moisture Condition
Dry: S = 0%
Humid: S = 1 to 25%
Damp: S = 26 to 50%
Moist: S = 51 to 75%
Wet: S = 76 to 99%
Saturated: S = 100%

SOIL BORING LOG

SUMMIT GEOENGINEERING SERVICES

Boring #:	BB-FJR-102		
Project:	Bridge Replacement	Project #:	19120
Location:	Johnson Rd.	Sheet:	3 of 3
City, State:	Falmouth, Maine	Chkd by:	ELS
Drilling Co:	Summit Geoengineering Services, Inc.	Boring Elevation:	36 ft +/-
Driller:	Craig Coolidge, P.E.	Reference:	Preliminary Plan, Sheet 2, Johnson Rd Bridge Provided by MDOT
Summit Staff:	Colleen Sullivan	Date started:	5/30/2019 Date Completed: 5/30/2019

DRILLING METHOD		SAMPLER	
Vehicle:	AMS	Length:	24" SS
Model:	9500 VTR	Diameter:	2"OD/1.5"ID
Method:	3" Casing	Hammer:	140 lb
Hammer Style:	Auto	Method:	ASTM D1586

ESTIMATED GROUND WATER DEPTH

Date	Depth	Elevation	Reference
5/30/2019	12.3 ft	N/A	Measured in open borehole after test

Depth (ft.)	No.	Pen/Rec (in)	Depth (ft)	blows/6"	Elev. (ft.)	SAMPLE DESCRIPTION	Geological/ Test Data	Geological Stratum
								GLACIAL MARINE DEPOSIT
45								
	UT-1	30/27	45 - 47.5	PUSH		Very soft, CLAY & SILT, occasional black organic		
46				PUSH		streaks and fine Sand-Silt lenses, wet, CL		
				PUSH				
47	FIELD VANES			PUSH				
			Tip of Vane					
48	FV-8		48		-12' +/-	S_U = 350 psf, $S_{U(r)}$ = 50 psf	SMALL VANE↓	
						(7 ft-lb, 1 ft-lb)		
49	FV-9		49		-13' +/-	S_U = 350 psf, $S_{U(r)}$ = 50 psf		
						(7 ft-lb, 1 ft-lb)		
50								
51	FV-10		51		-15' +/-	S_U = 425 psf, $S_{U(r)}$ = 50 psf		
						(8.5 ft-lb, 1 ft-lb)		
52	FV-11		52		-16' +/-	S_U = 450 psf, $S_{U(r)}$ = 50 psf		
						(9 ft-lb, 1 ft-lb)		
53								
54	FV-12		54		-18' +/-	S_U = 525 psf, $S_{U(r)}$ = 75 psf		
						(11 ft-lb, 1.5 ft-lb)		
55	FV-13		55		-19' +/-	S_U = 550 psf, $S_{U(r)}$ = 75 psf		
						(11.5 ft-lb, 1.5 ft-lb)		
56						End of Exploration at 55.5', No Refusal		55.5'
								POSSIBLE SAND SEAM
57								
58								
59								
60								
61								
62								
63								
64								
65								
66								

BURMISTER SOIL CLASSIFICATION

Granular Soils		Cohesive Soils		% Composition Description
Blows/ft.	Density	Blows/ft.	Consistency	
0-4	V. Loose	<2	V. soft	0-10% trace
5-10	Loose	2-4	Soft	10-20% little
11-30	Compact	5-8	Firm	20-35% some
31-50	Dense	9-15	Stiff	35-50% and
>50	V. Dense	16-30	V. Stiff	
		>30	Hard	

NOTES: PP = Pocket Penetrometer, MC = Moisture Content, S = Split spoon sample, LL = Liquid Limit, PI = Plasticity Index, UT = Shelby Tube Sample, S_U = Undrained Shear Strength, $S_{U(r)}$ = Remolded Shear Strength, WOH = Weight of Hammer

Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches
Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200

Soil Moisture Condition
- Dry: S = 0%
- Humid: S = 1 to 25%
- Damp: S = 26 to 50%
- Moist: S = 51 to 75%
- Wet: S = 76 to 99%
- Saturated: S = 100%

SOIL BORING LOG

SUMMIT GEOENGINEERING SERVICES

Boring #:	**BB-FJR-107**		
Project:	Bridge Replacement	Project #:	19120
Location:	Johnson Rd.	Sheet:	1 of 1
City, State:	Falmouth, Maine	Chkd by:	ELS
Drilling Co:	Summit Geoengineering Services, Inc.	Boring Elevation	46 ft +/-
Driller:	Craig Coolidge, P.E.	Reference:	Preliminary Plan, Sheet 2, Johnson Rd Bridge Provided by MDOT
Summit Staff:	Colleen Sullivan	Date started: 5/28/2019	Date Completed: 5/28/2019

DRILLING METHOD		SAMPLER	
Vehicle:	AMS	Length:	24" SS
Model:	9500 VTR	Diameter:	2"OD/1.5"ID
Method:	3" Casing w/Wash	Hammer:	140 lb
Hammer Style:	Auto	Method:	ASTM D1586

ESTIMATED GROUND WATER DEPTH

Date	Depth	Elevation	Reference
5/28/2019	12.4 ft	34 ft +/-	Measured in open borehole after test
5/28/2019	7 ft	39 ft +/-	Observed moisture change in samples
5/28/2019	3 ft	43 ft +/-	Perched water in Fill Layer

Depth (ft.)	No.	Pen/Rec (in)	Depth (ft)	blows/6"	Elev. (ft.)	SAMPLE DESCRIPTION	Geological/ Test Data	Geological Stratum
	S-1	24/12	0 - 2	1		Very loose, brown, fine to medium SAND, little Silt,		
1				1		trace Gravel, moist, SP-SM		FILL
				2				
2				2				
	S-2	24/24	2 - 4	3		Loose, brown, fine to medium SAND, little Silt,		
3				4		trace Gravel, moist to wet, SP-SM		
				5	43' +/-	Stiff, olive brown, Silt & Clay, slight mottling, moist,		3'
4				7		CL		GLACIAL MARINE
								DEPOSIT
5								(PRESUMPSCOT
	S-3	24/24	5 - 7	4		Stiff, mottled olive brown, SILT & CLAY, moist, CL	PP = 5,000 to	FORMATION)
6				5			6,000 psf	
				6				
7				9				
	S-4	24/24	7 - 9	5		Same as above, wet, CL	PP = 3,000 to	
8				5			4,000 psf	
				7				
9				7				
10								
	S-5	24/24	10 - 12	2		Firm, olive gray, CLAY & SILT, wet, CL	PP = 2,000 to	
11				3			3,000 psf	
				3				
12	FIELD VANES			4				
			Tip of Vane					
13	FV - 1		13		33' +/-	S_U = 1950 psf, $S_{U(r)}$ = 500 psf	SMALL VANE↓	
						(40 ft-lb, 10 ft-lb)		
14								
15								
	S-6	24/24	15 - 17	1		Soft, gray, CLAY & SILT, wet, CL	PP = 1,500 to	
16				1			2,000 psf	
				2				
17				3				
	S-7	24/24	17 - 19	2		Same as above, occasional fine sand lense, CL	PP = 500 psf +/-	
18				3				
				5				
19				8				
20								
21								
					25' +/-	End of Exploration at 20.8', Refusal		20.8'
22								BEDROCK

BURMISTER SOIL CLASSIFICATION

Granular Soils		Cohesive Soils		% Composition
Blows/ft.	Density	Blows/ft.	Consistency	Description
0-4	V. Loose	<2	V. soft	0-10% trace
5-10	Loose	2-4	Soft	10-20% little
11-30	Compact	5-8	Firm	20-35% some
31-50	Dense	9-15	Stiff	35-50% and
>50	V. Dense	16-30	V. Stiff	
		>30	Hard	

NOTES: PP = Pocket Penetrometer, MC = Moisture Content,
S = Split spoon sample, LL = Liquid Limit, PI = Plasticity Index,
UT = Shelby Tube Sample, S_U = Undrained Shear Strength,
$S_{U(r)}$ = Remolded Shear Strength, WOH = Weight of Hammer

Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches
Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200

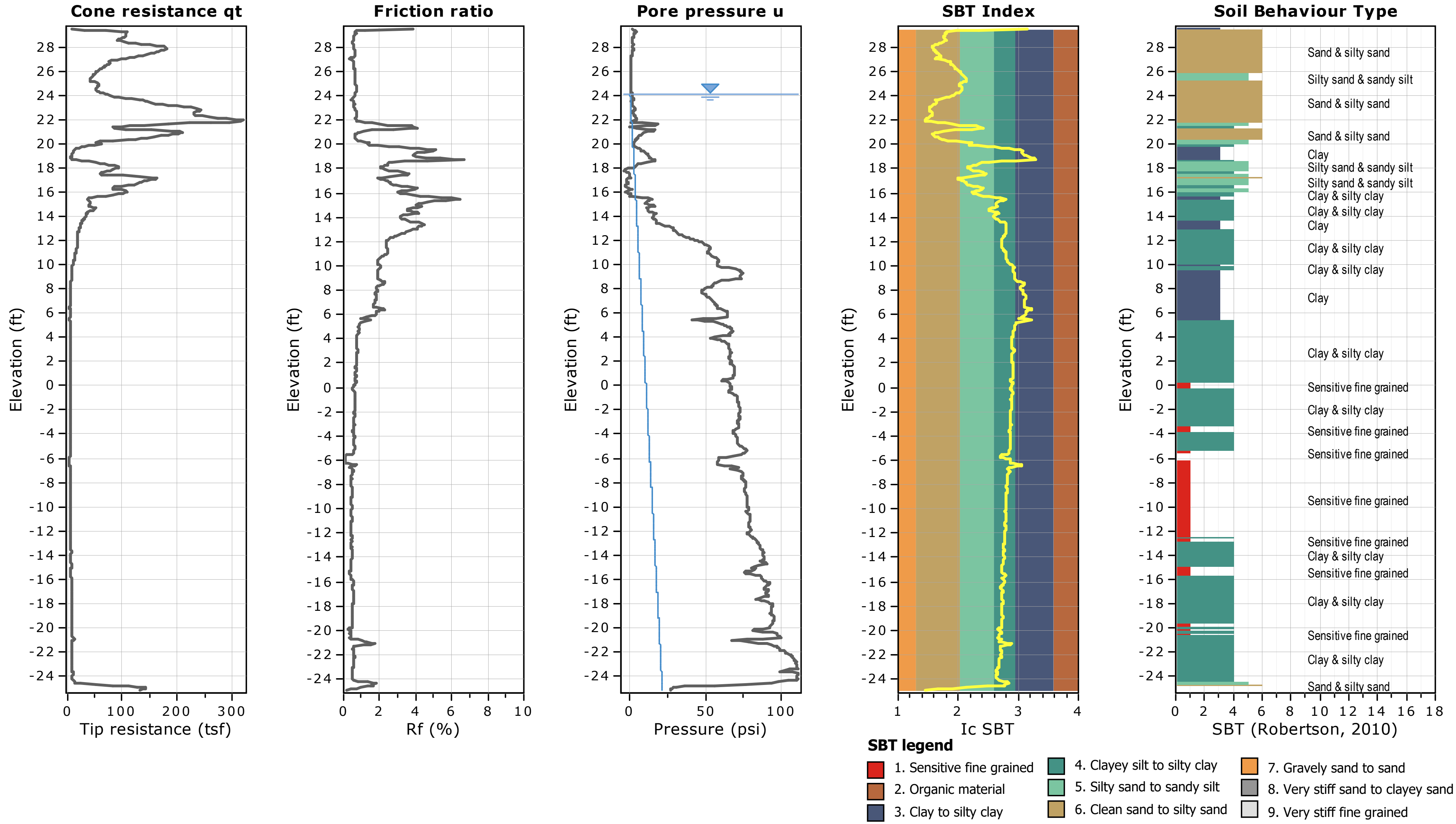
Soil Moisture Condition
- Dry: S = 0%
- Humid: S = 1 to 25%
- Damp: S = 26 to 50%
- Moist: S = 51 to 75%
- Wet: S = 76 to 99%
- Saturated: S = 100%

APPENDIX C.2 – CPT INTERPRETATION REPORT BY GZA

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

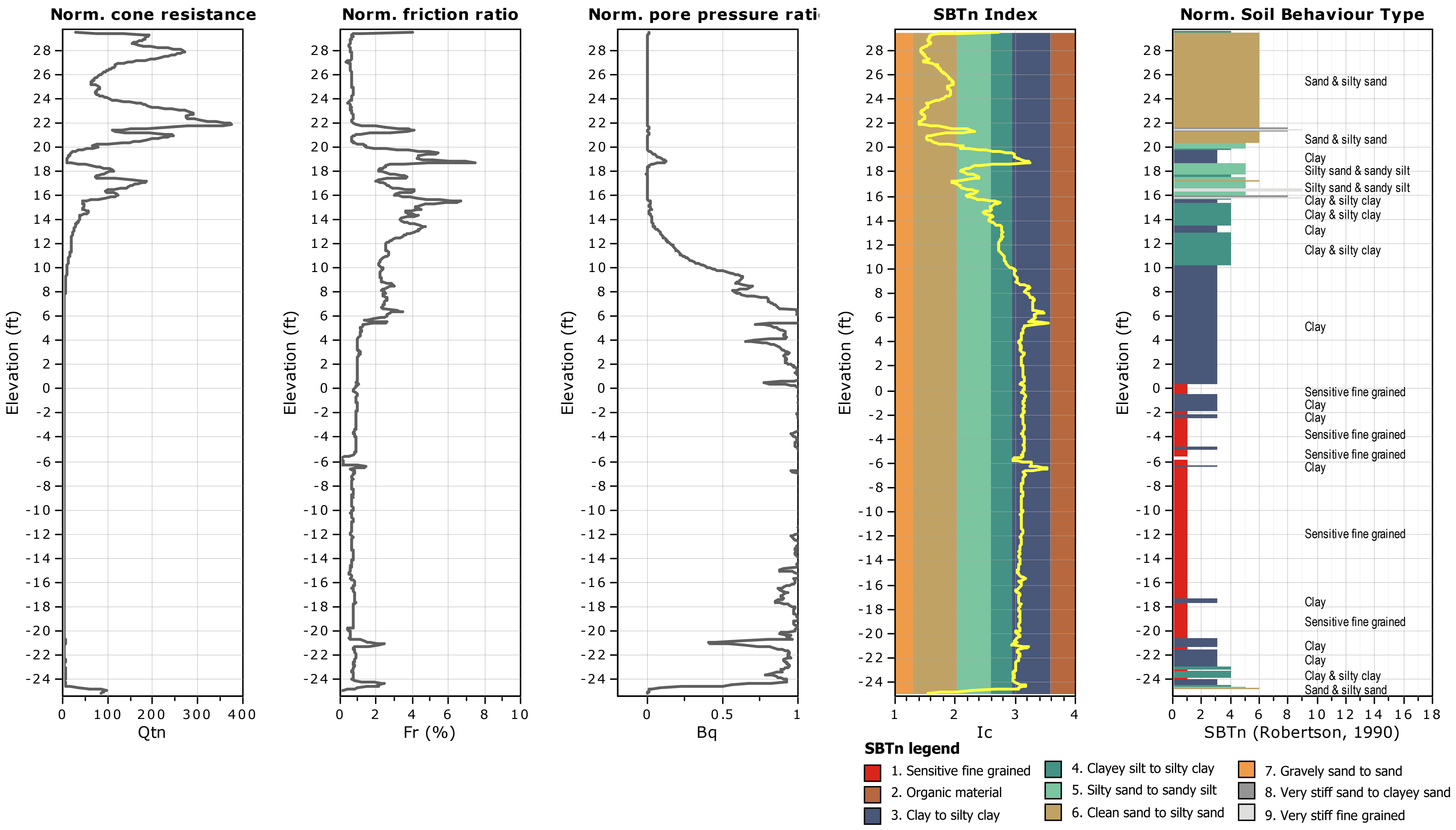
CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

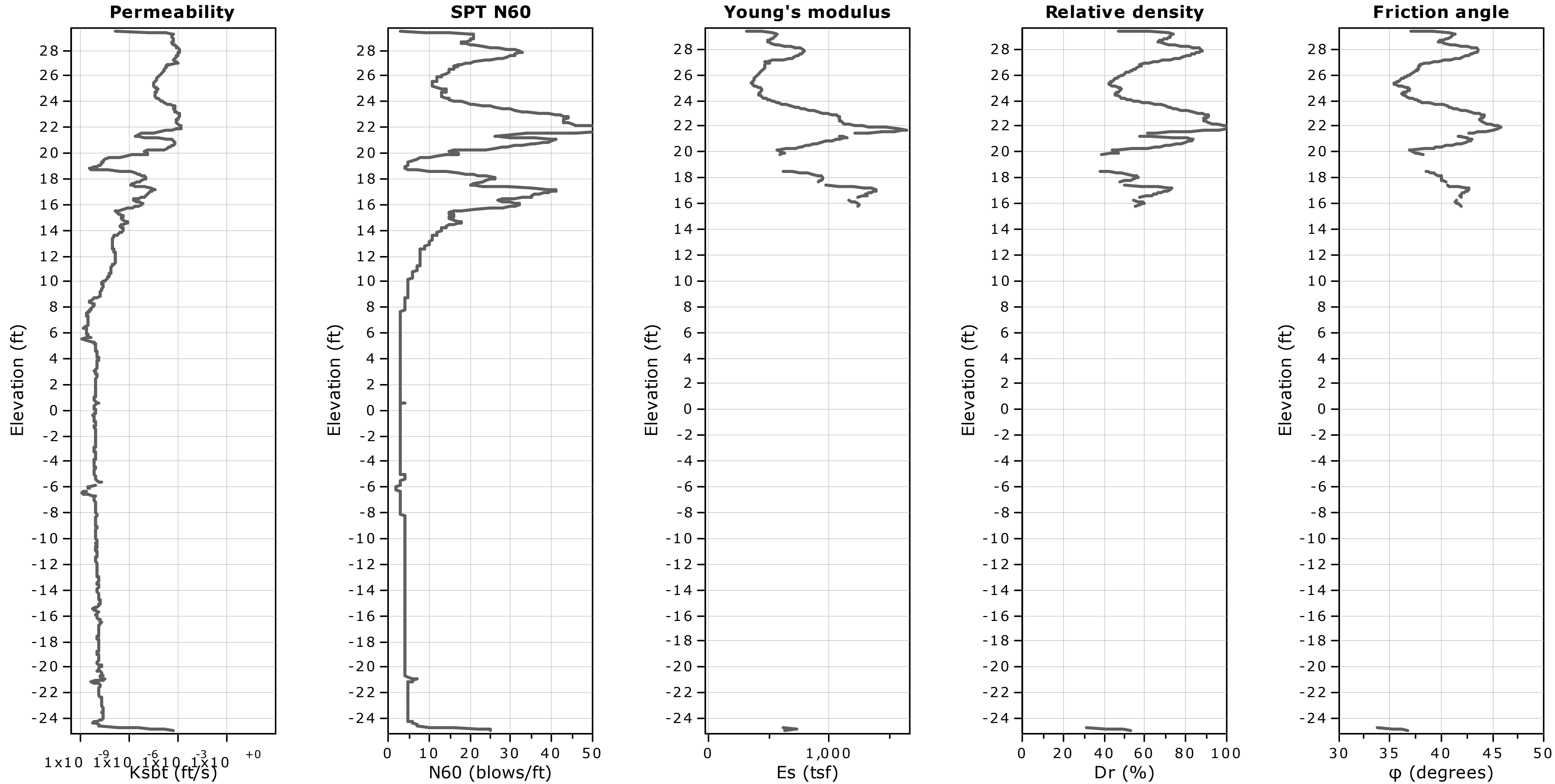
Project: Johnson Road Bridge
Location: Falmouth, Maine



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

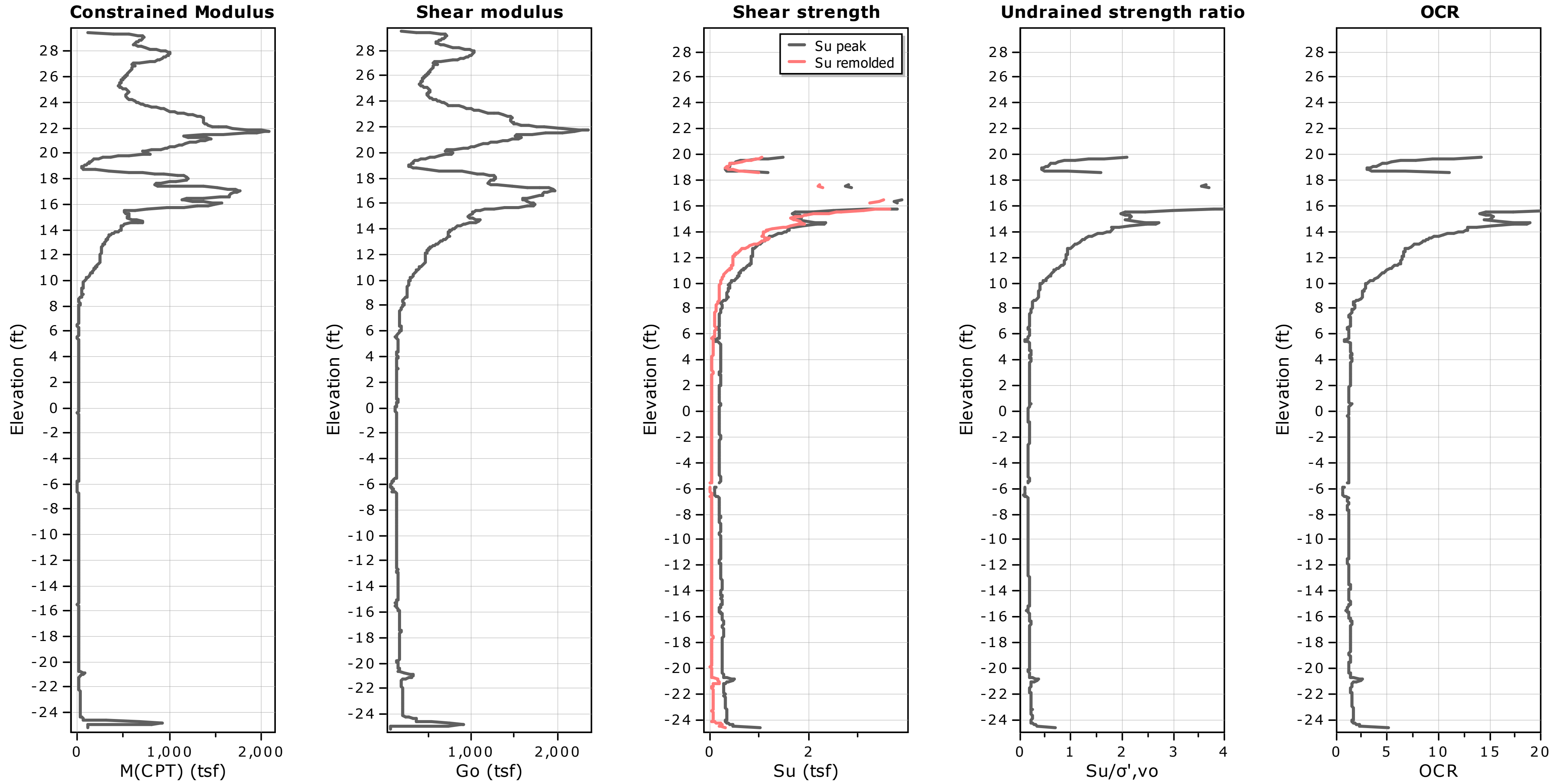


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

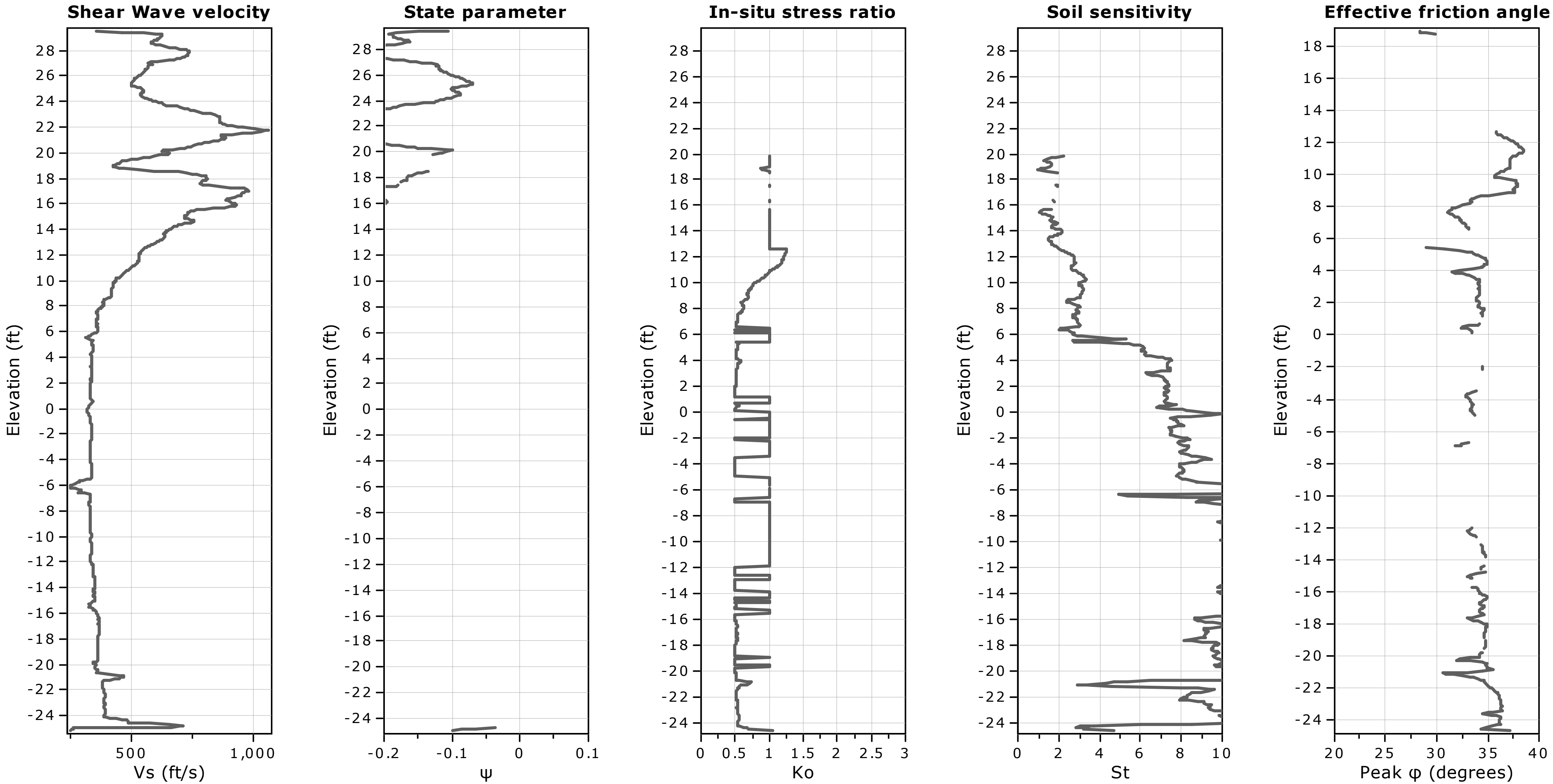


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 22
OCR factor for clays, N_{kt}: 0.33
Flat Dilatometer Test data

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



Calculation parameters

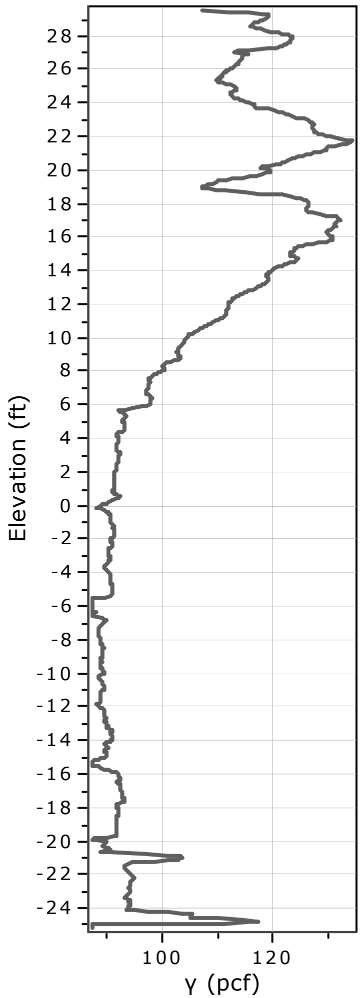
Soil Sensitivity factor, N_S: 7.00

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

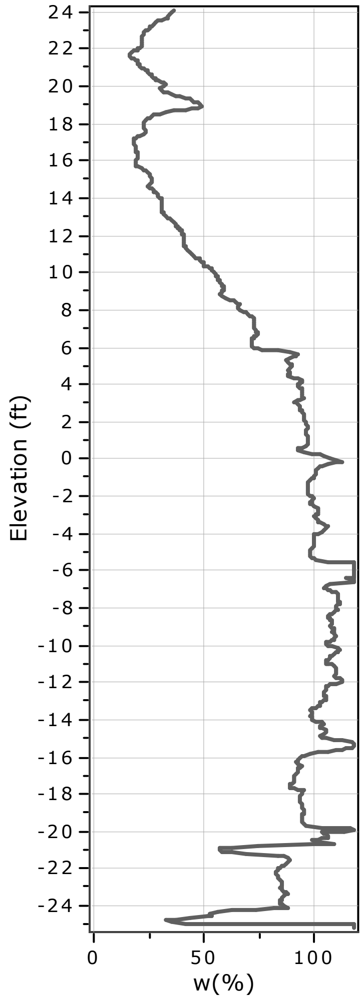
CPT-FJR-101
Total depth: 59.48 ft, Date: 6/17/2019
Surface Elevation: 34.30 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

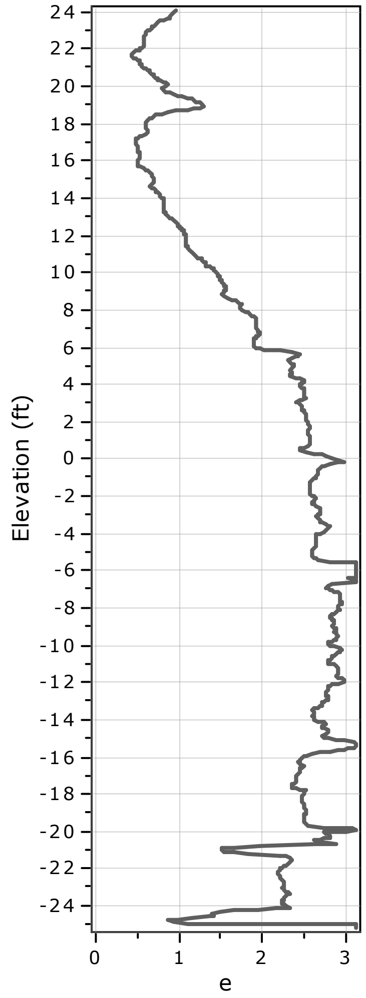
Bulk unit weight



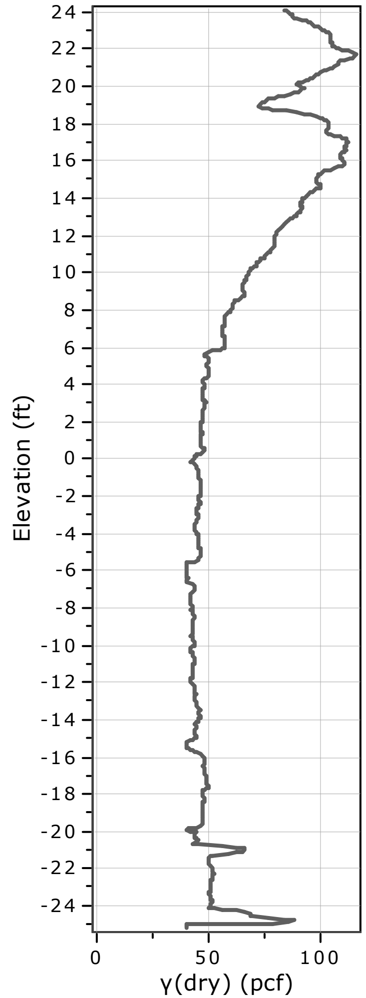
Water content



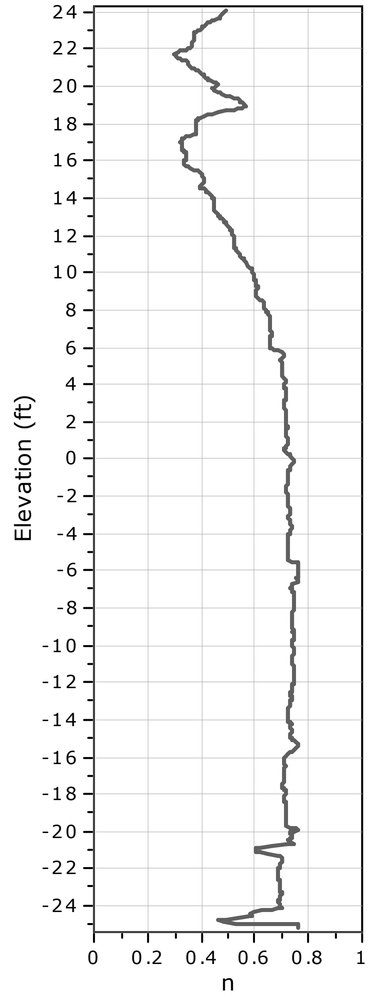
Void ratio



Dry unit weight



Porosity



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-101
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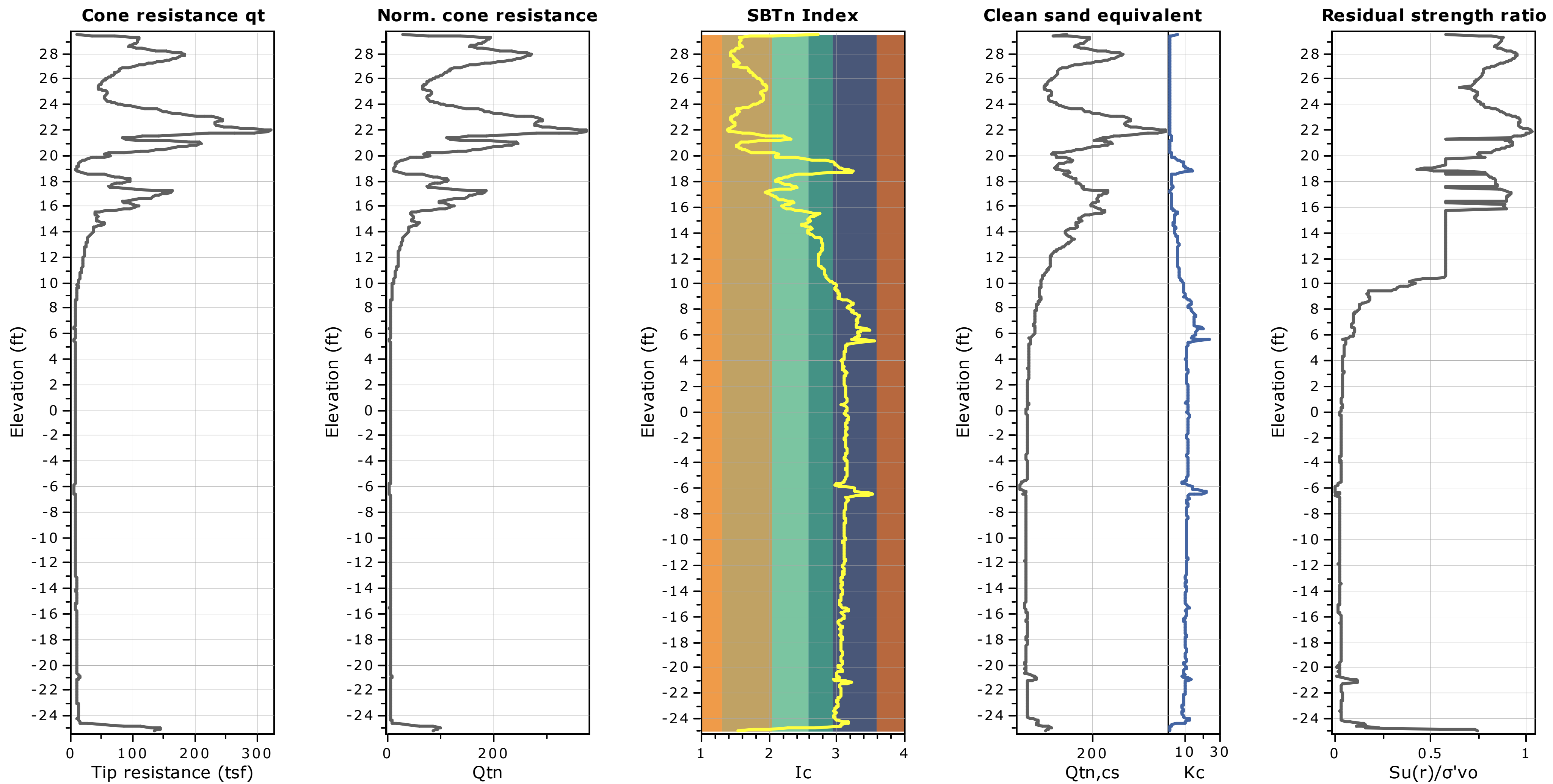
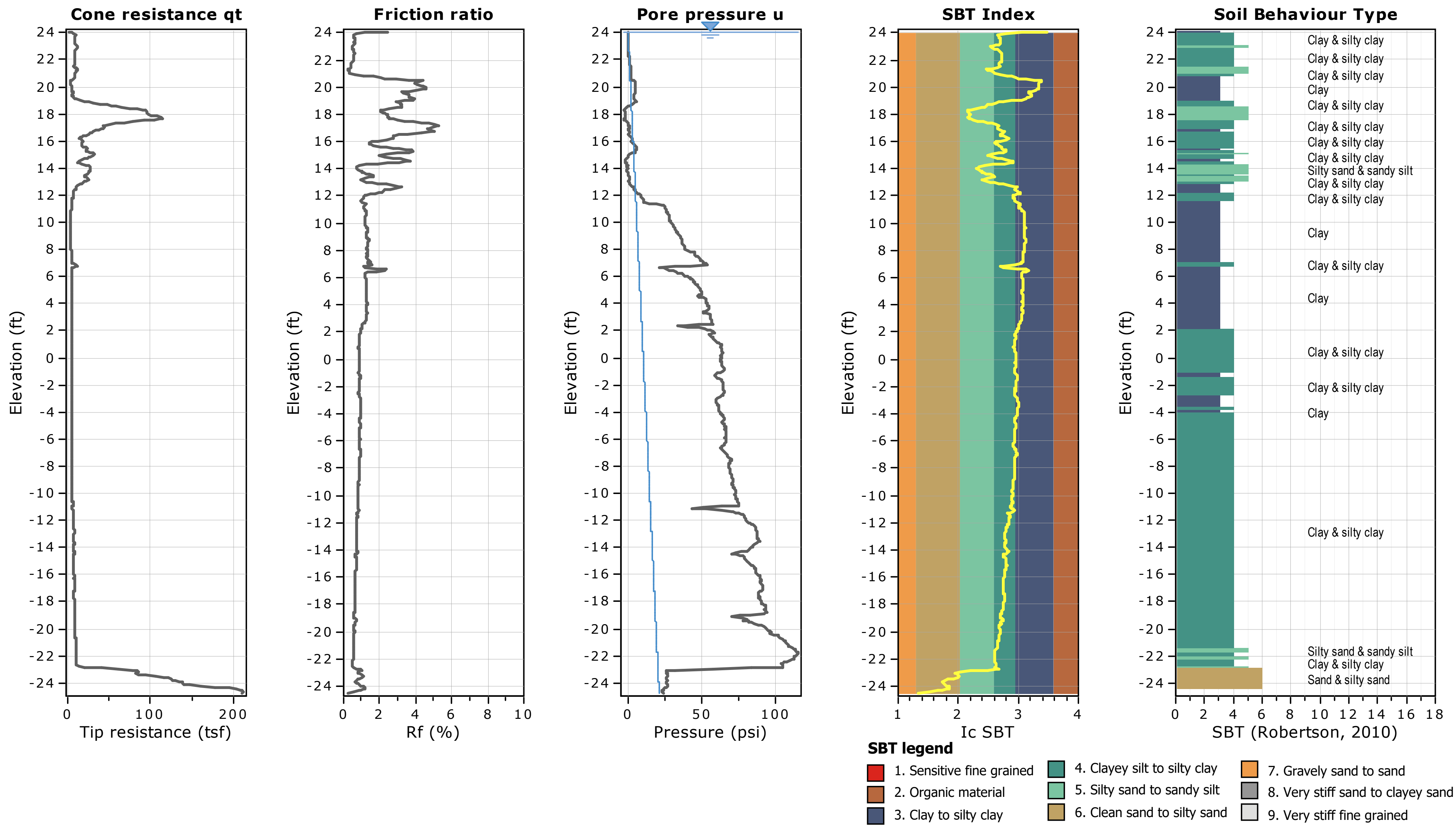
Location: **Falmouth, Maine**



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

CPT-FJR-102
Total depth: 53.76 ft, Date: 6/17/2019
Surface Elevation: 29.00 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



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Geotechnical Engineers
South Portland, Maine

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Location: **Falmouth, Maine**

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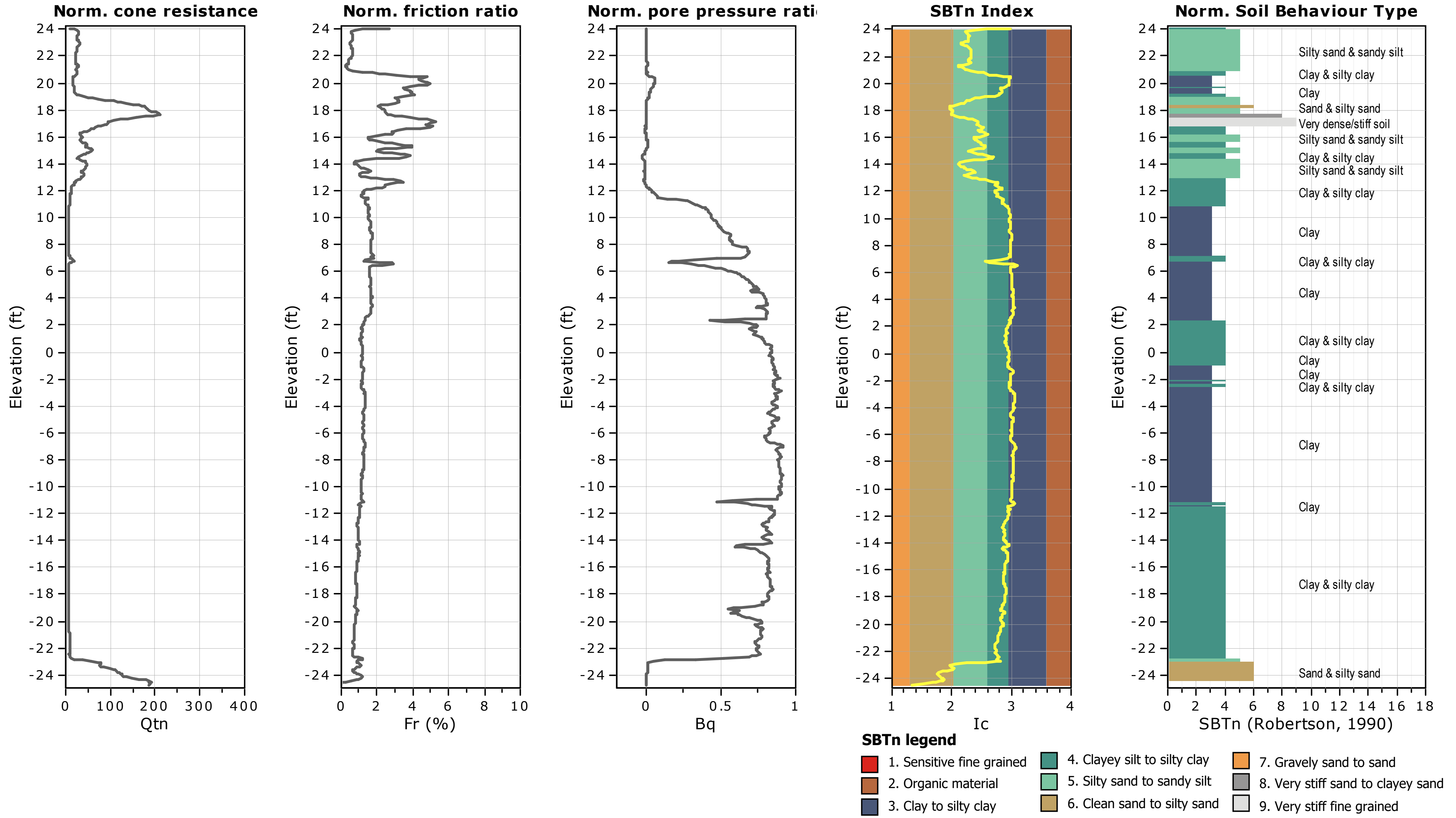

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South Portland, Maine

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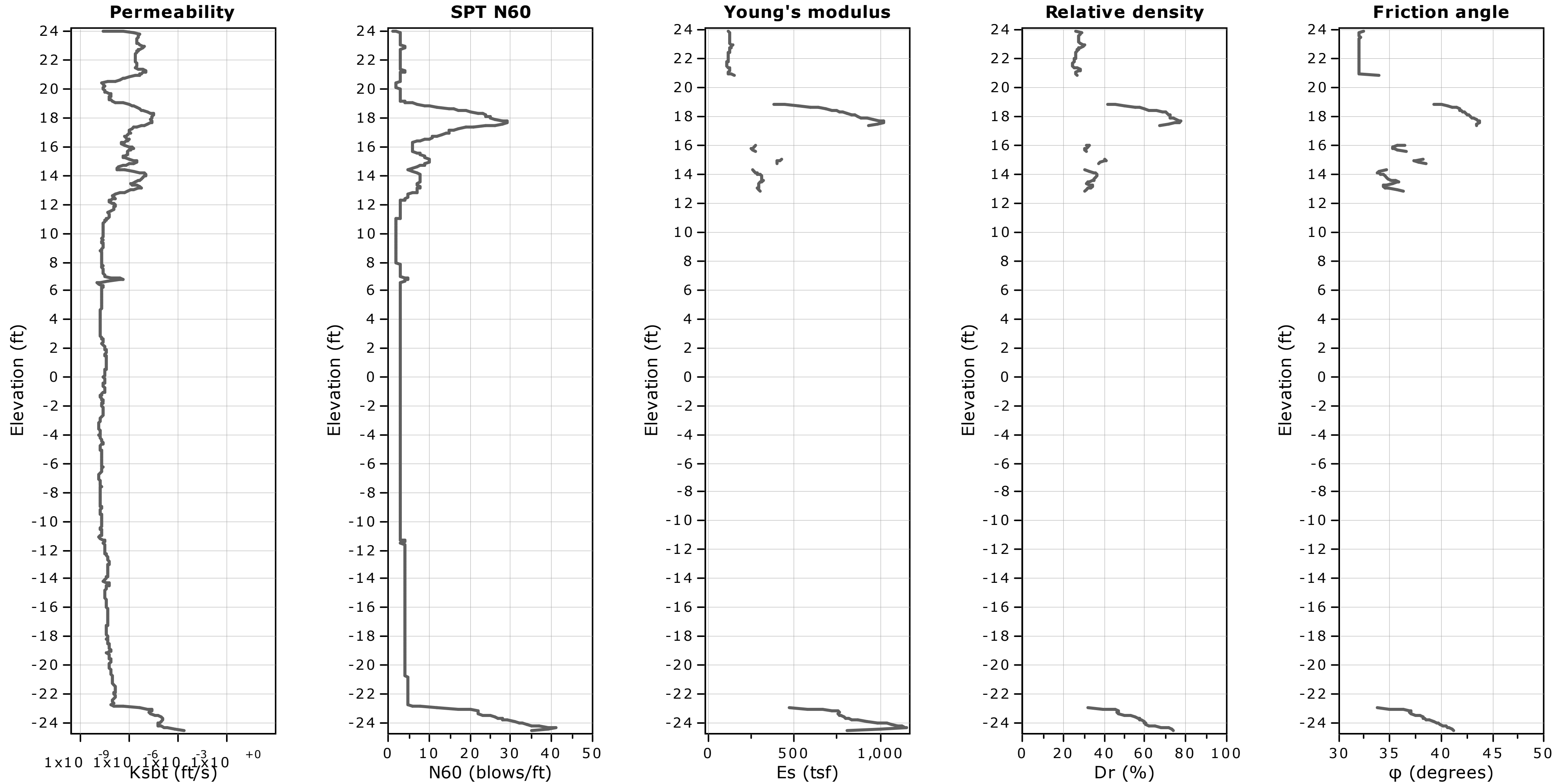
Location: **Falmouth, Maine**



Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

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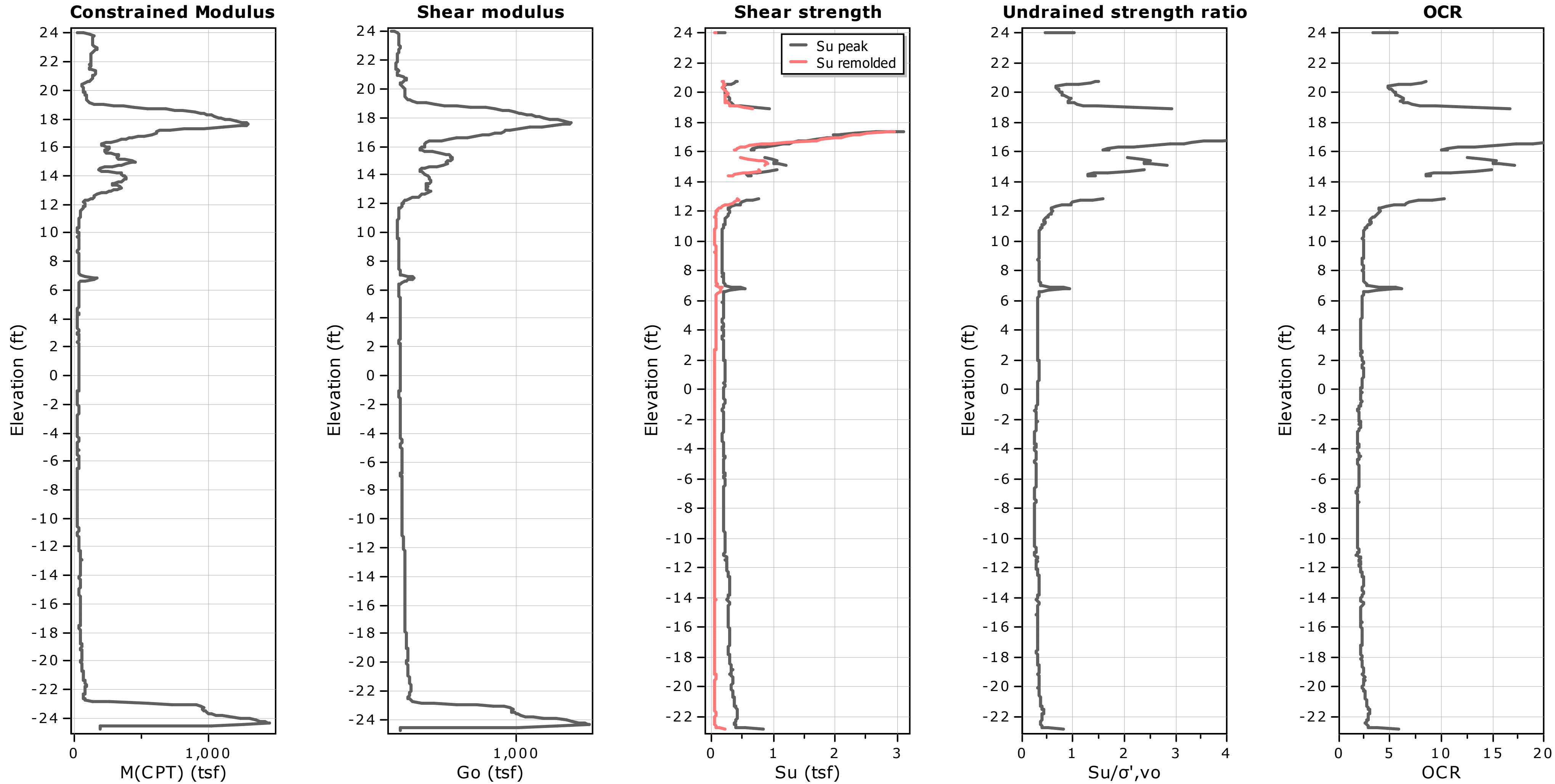

Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 22
OCR factor for clays, N_{kt}: 0.33
Flat Dilatometer Test data

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

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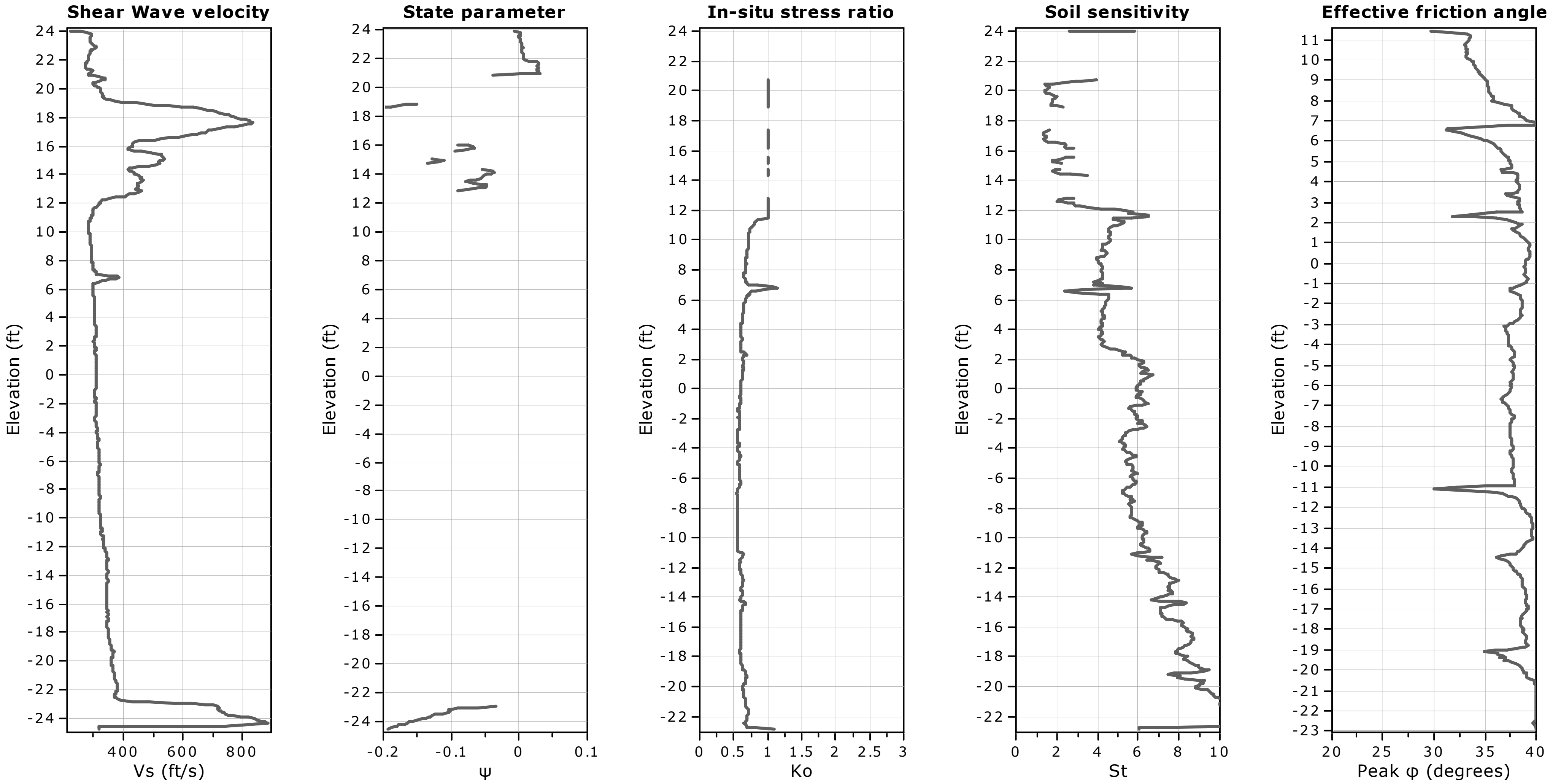

Calculation parameters

Soil Sensitivity factor, N_S: 7.00

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

CPT-FJR-102
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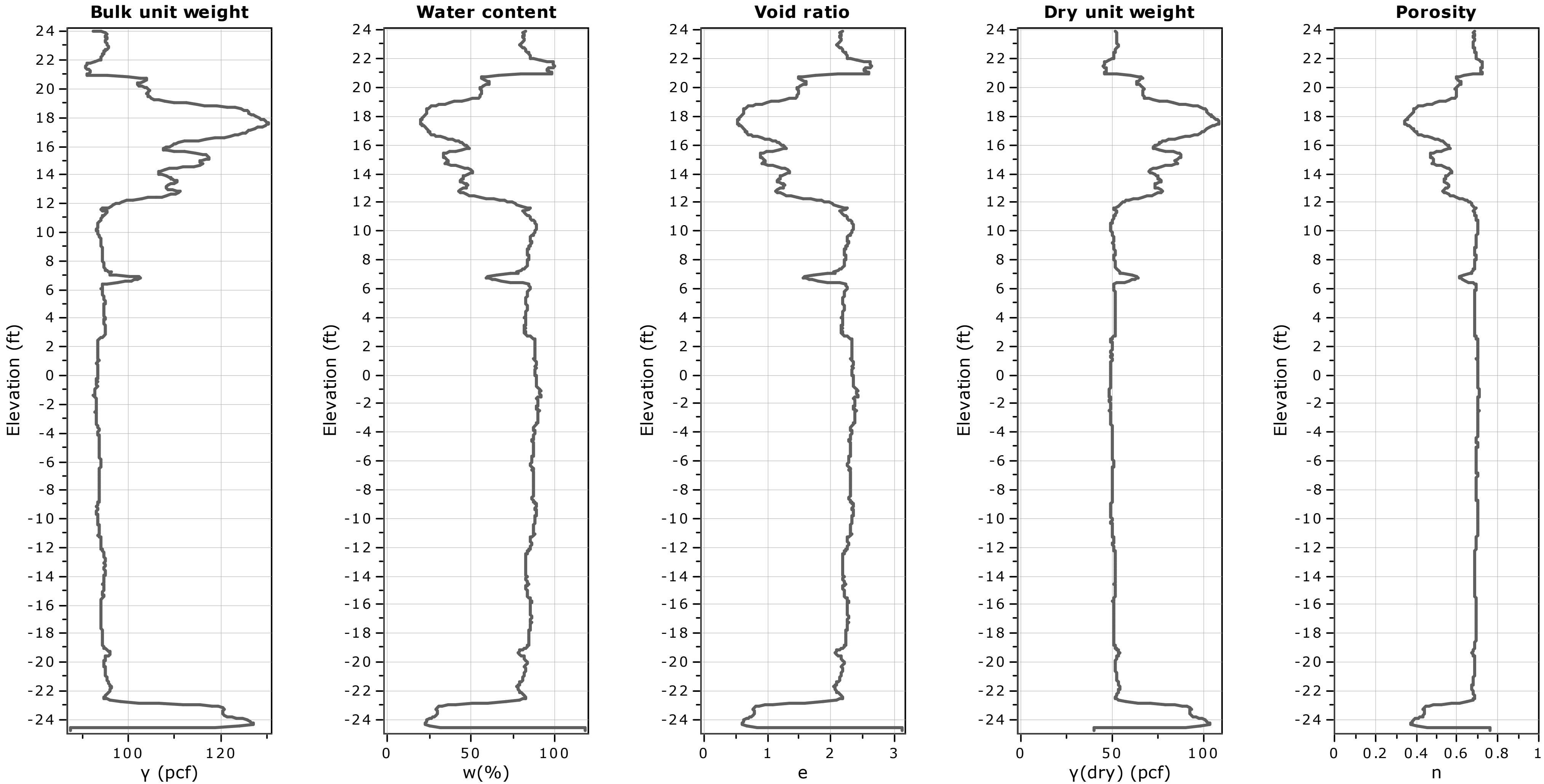

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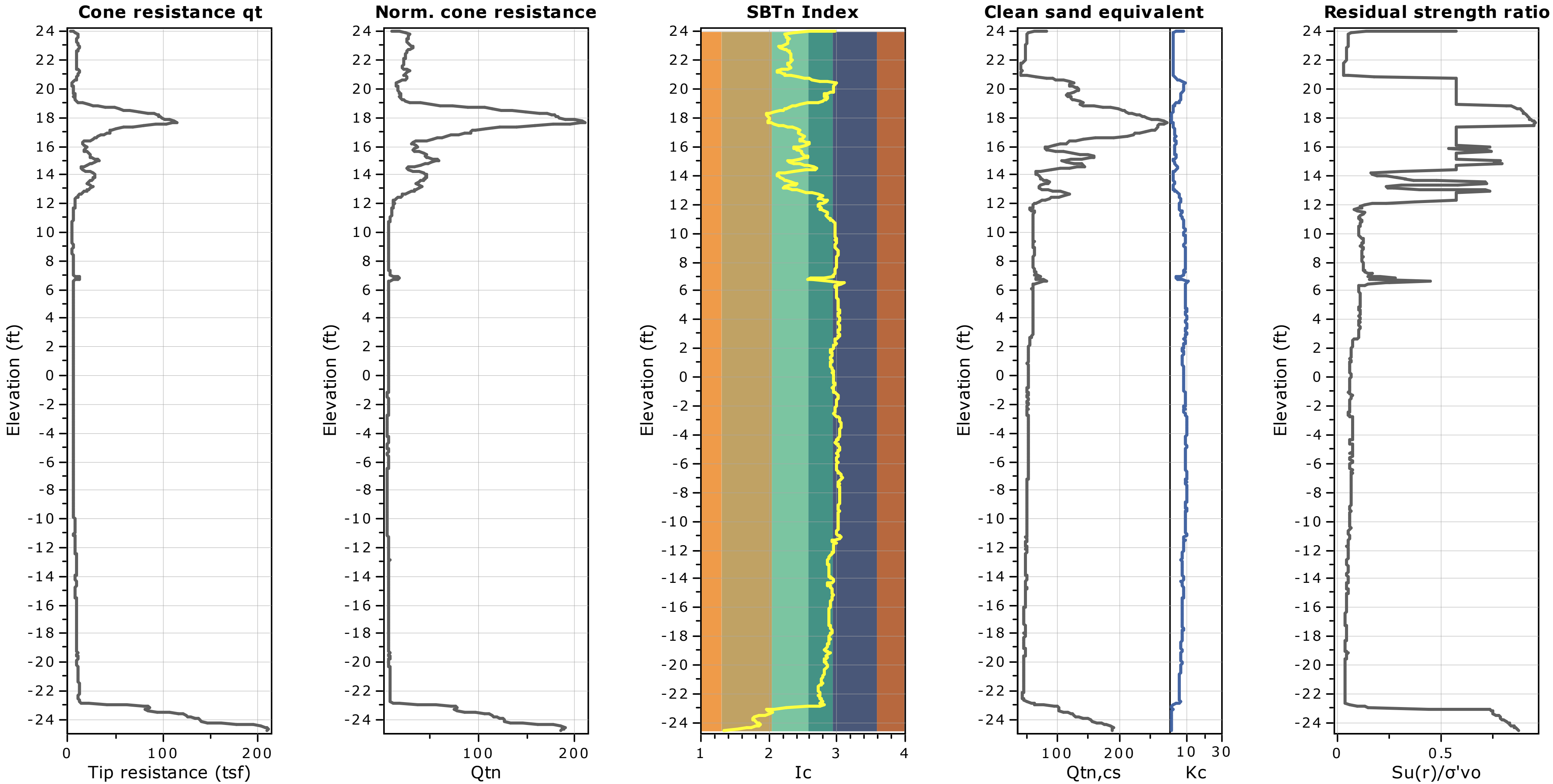
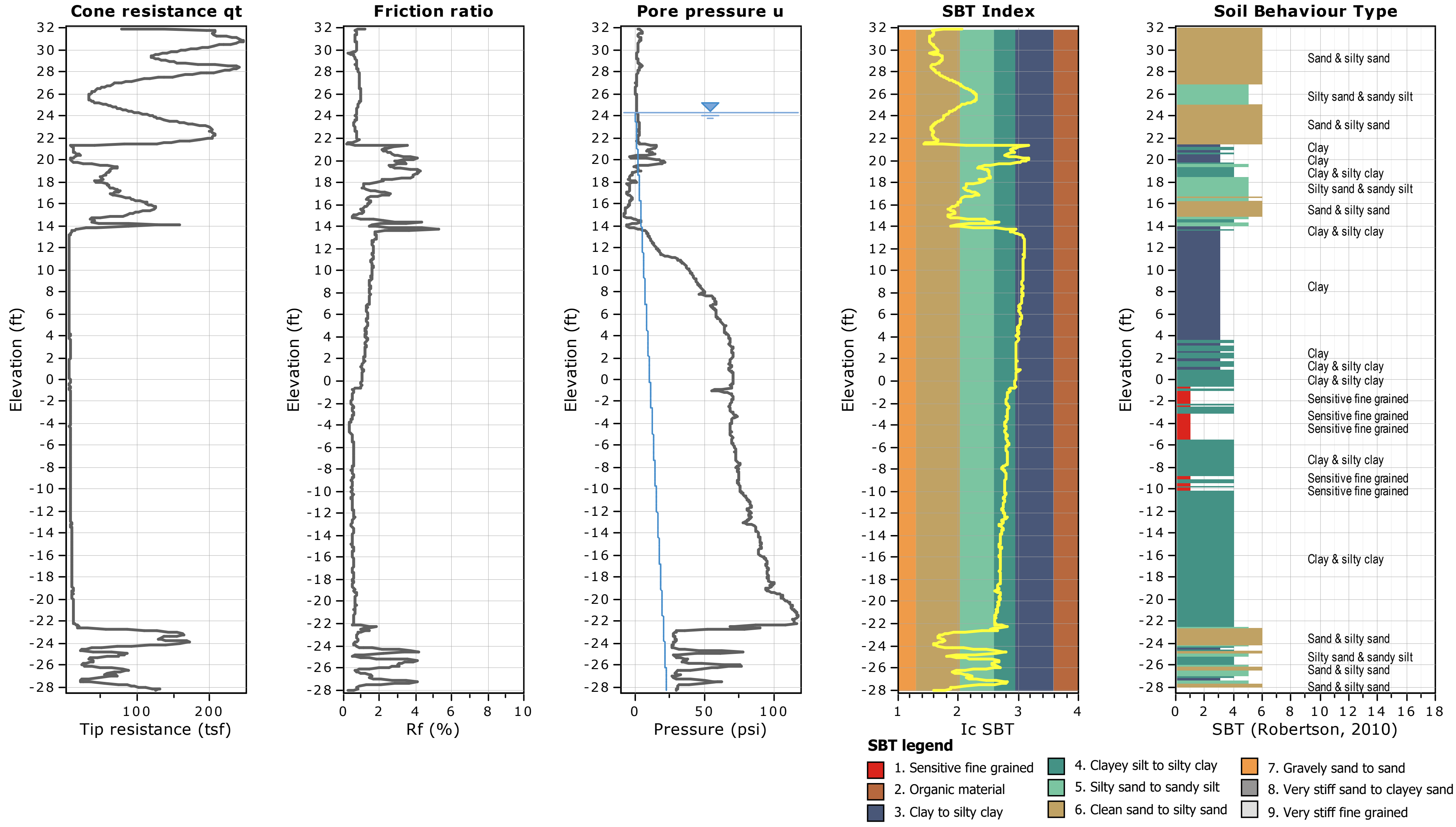


GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

CPT-FJR-103
Total depth: 65.10 ft, Date: 6/17/2019
Surface Elevation: 36.80 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



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Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

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Surface Elevation: 36.80 ft
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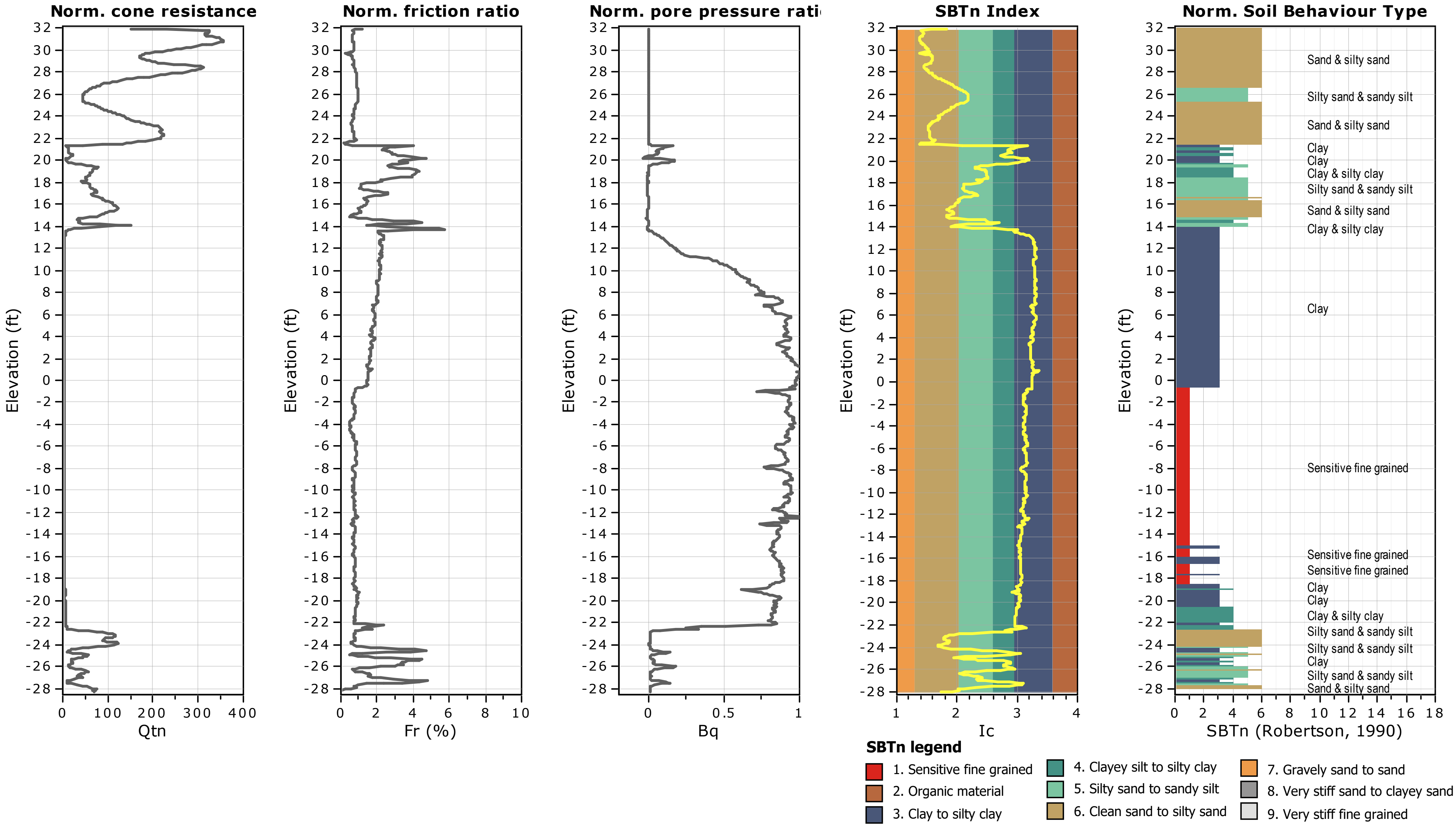
Cone Operator: Summit Geoengineering Services



GZA GeoEnvironmental Inc.
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South Portland, Maine

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Surface Elevation: 36.80 ft
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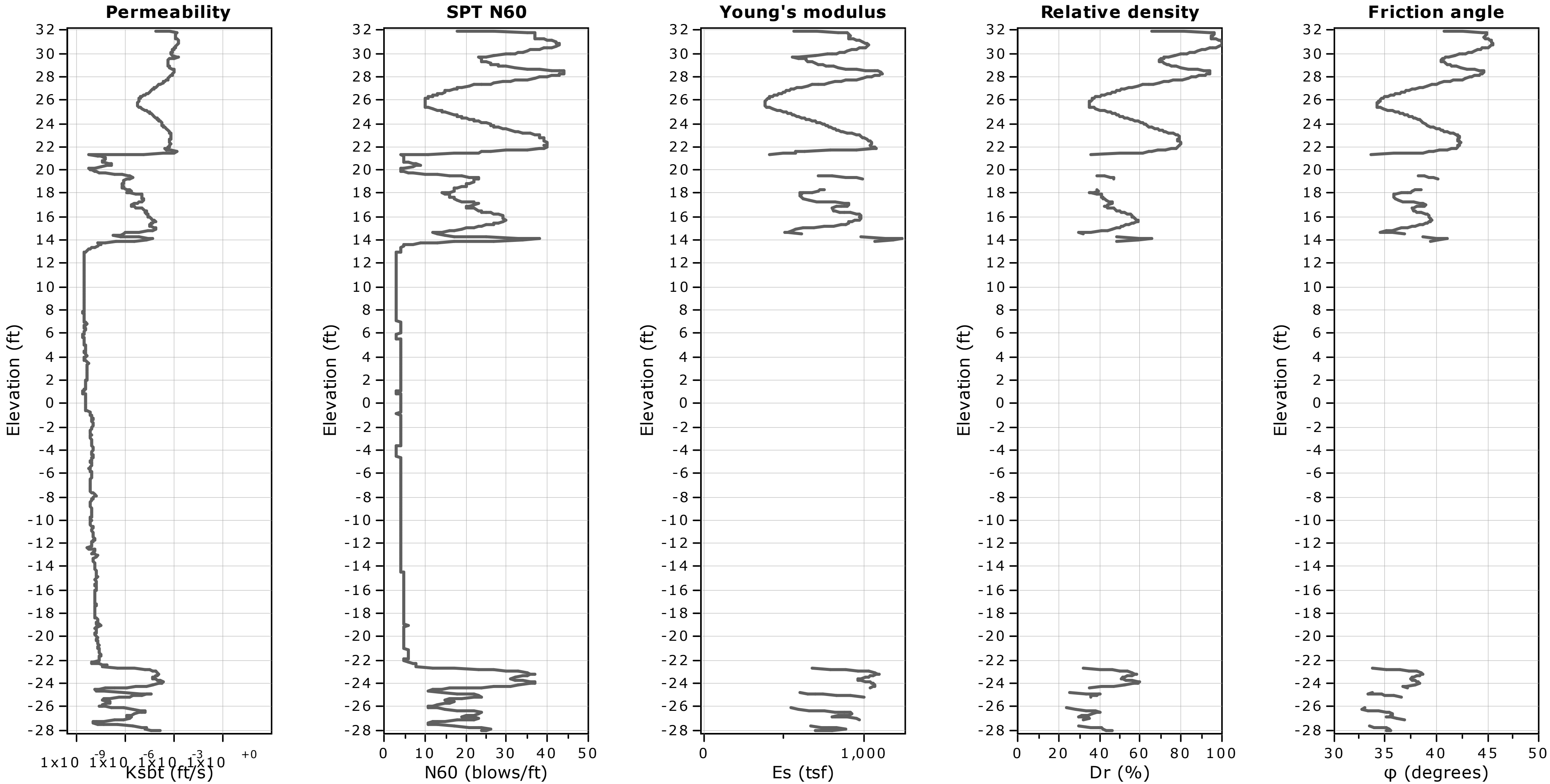
Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-103
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Surface Elevation: 36.80 ft
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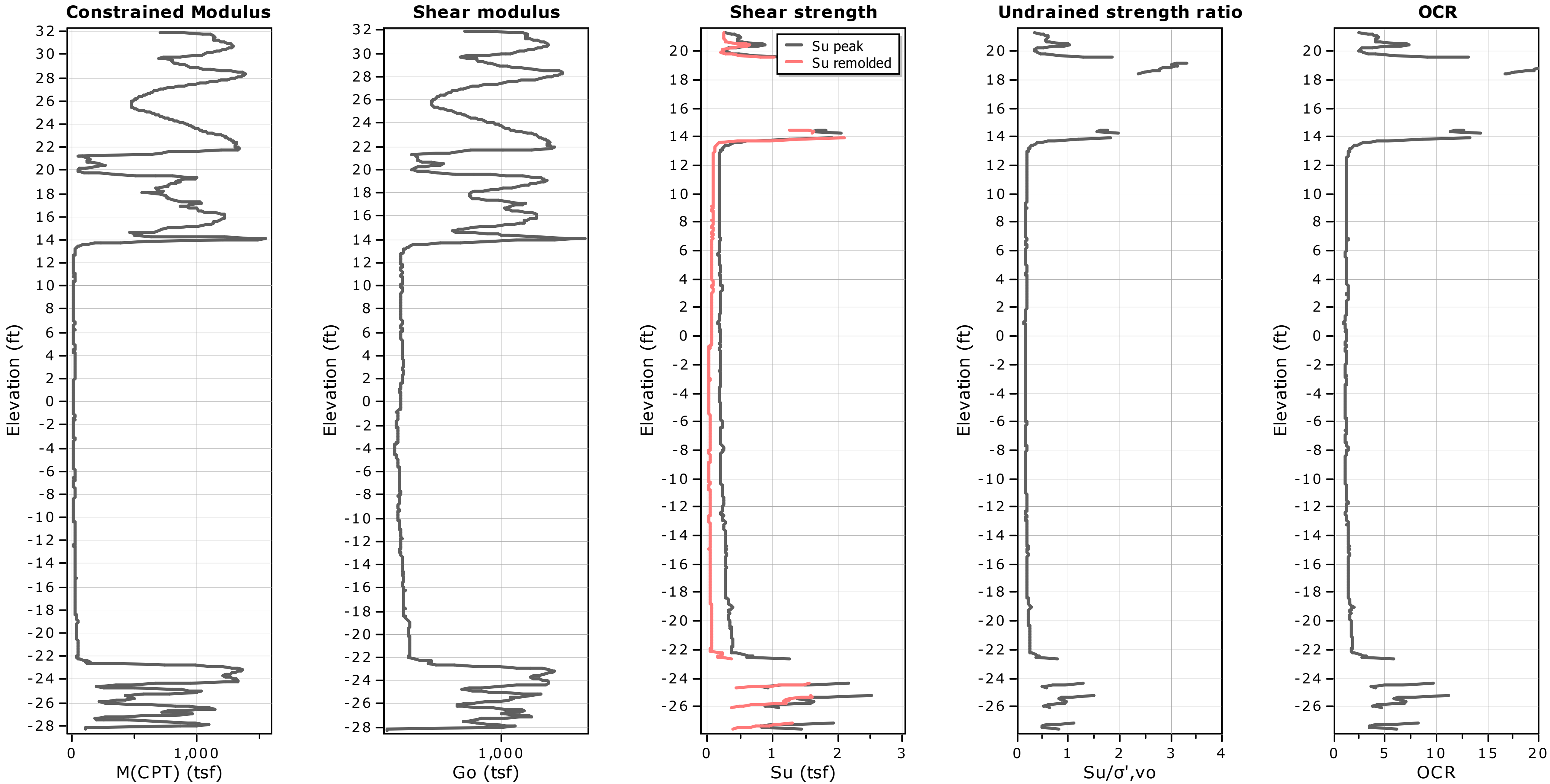
Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 22

OCR factor for clays, N_{kt}: 0.33
Flat Dilatometer Test data

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: Johnson Road Bridge
Location: Falmouth, Maine

CPT-FJR-103
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Surface Elevation: 36.80 ft
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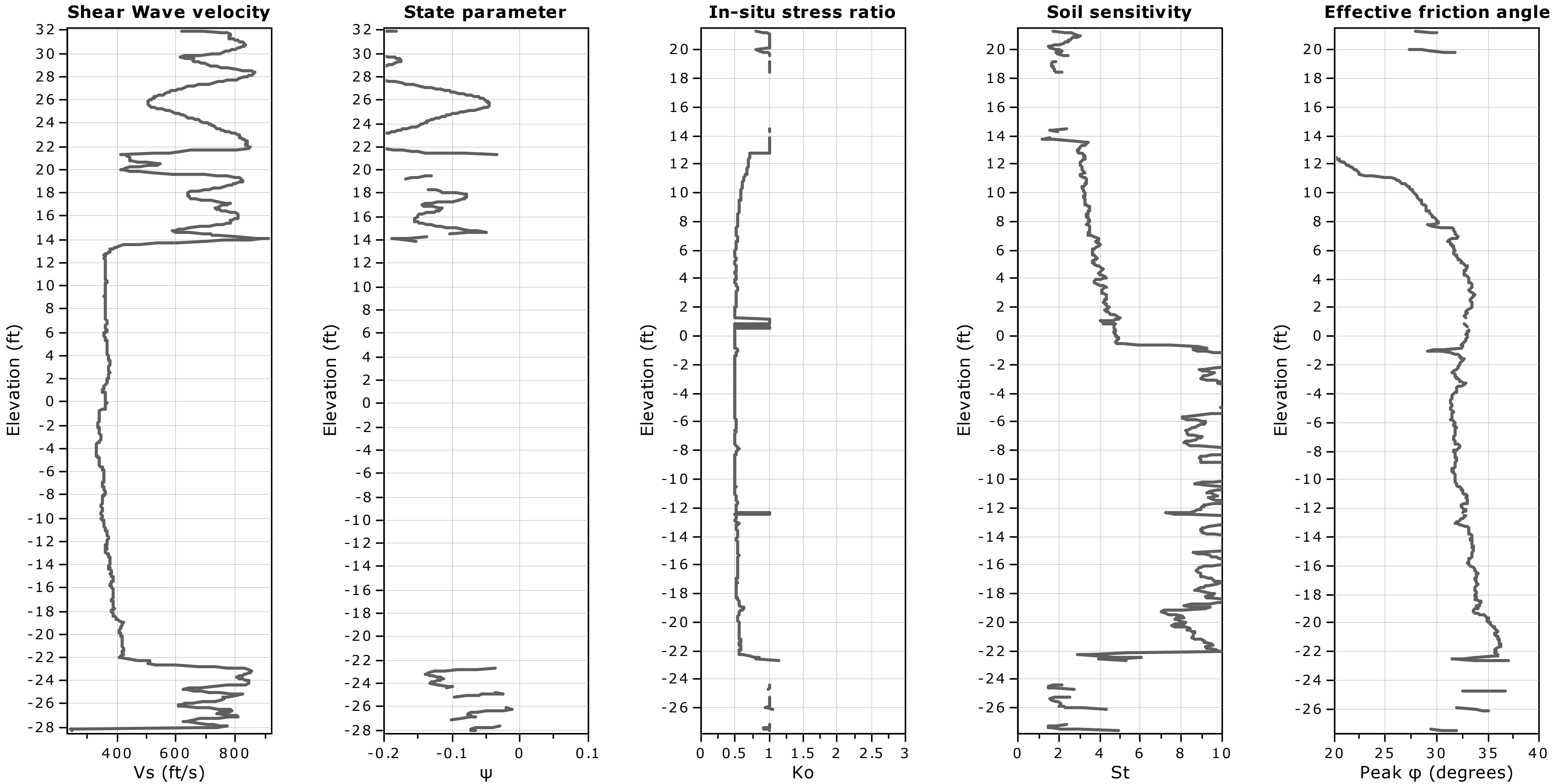

Calculation parameters

Soil Sensitivity factor, N_S: 7.00

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-103
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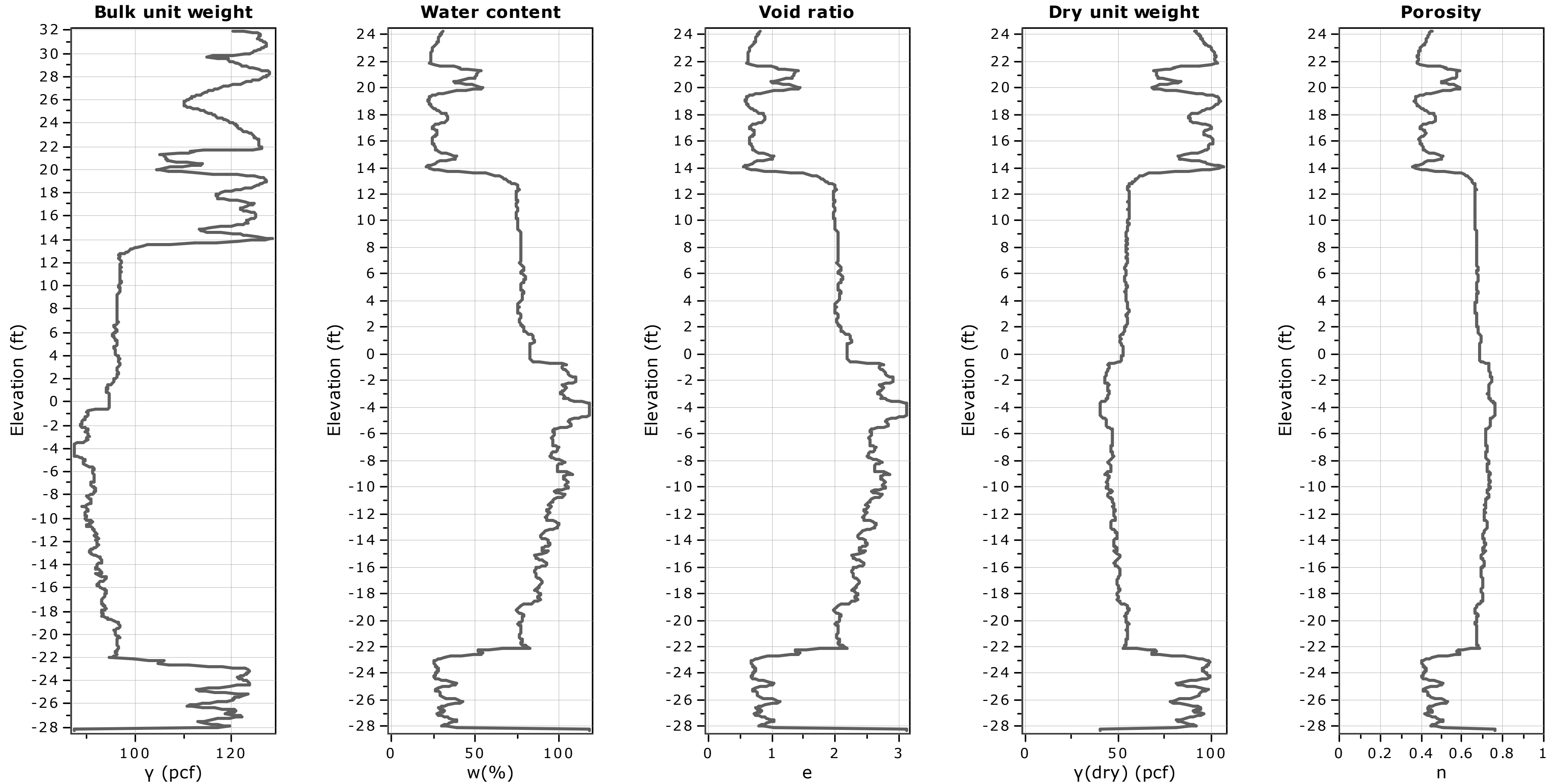

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South Portland, Maine

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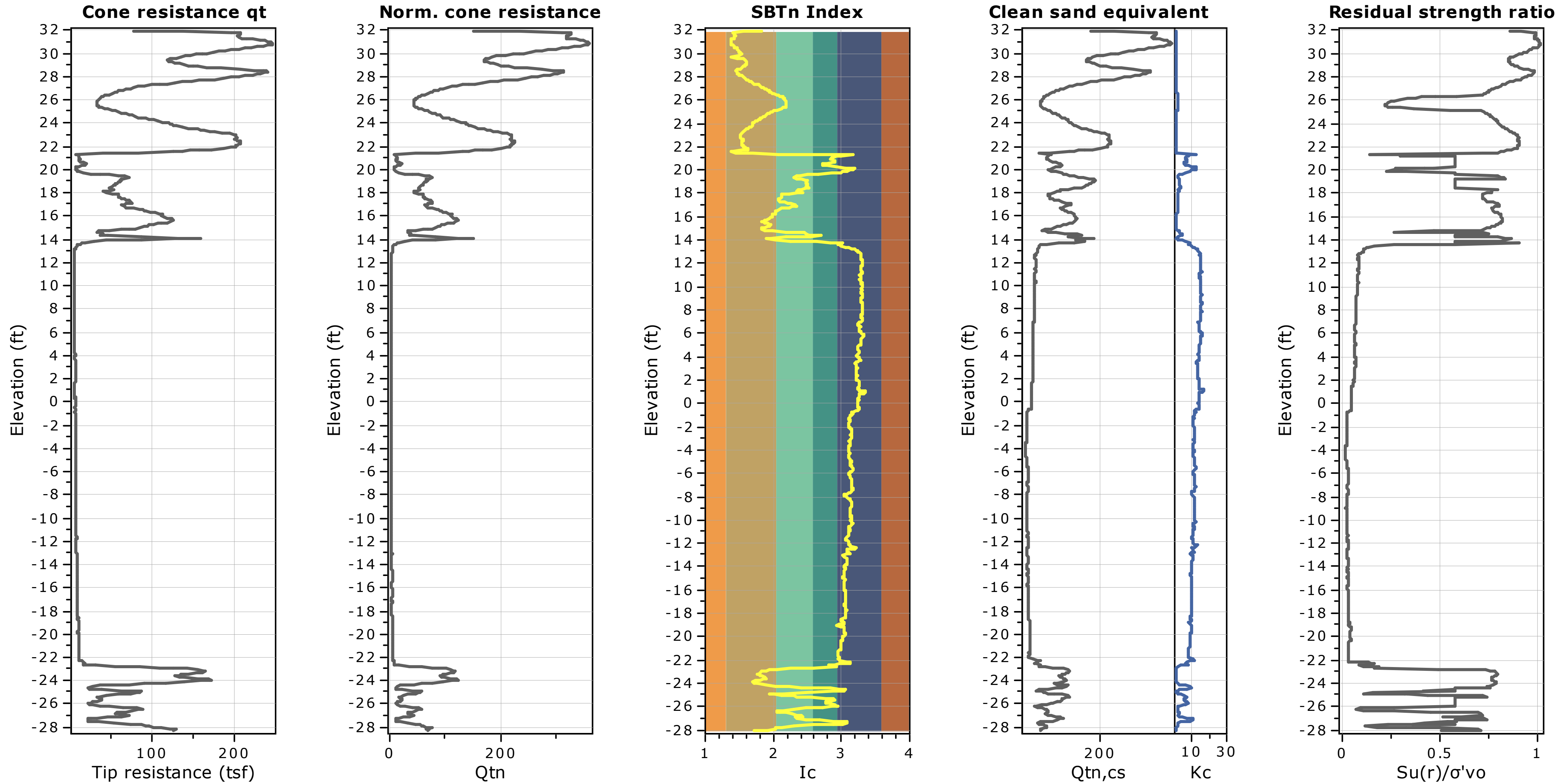
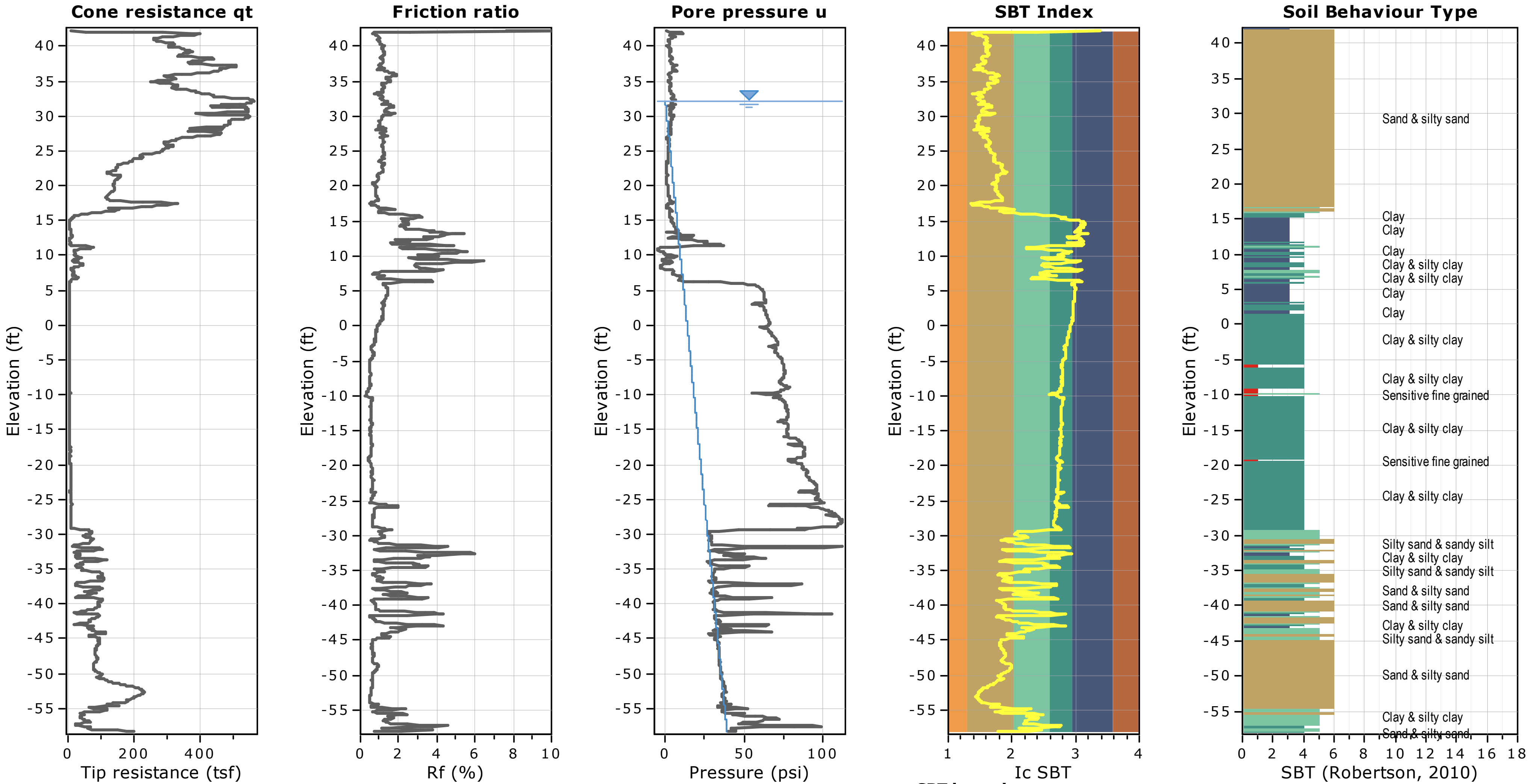


GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



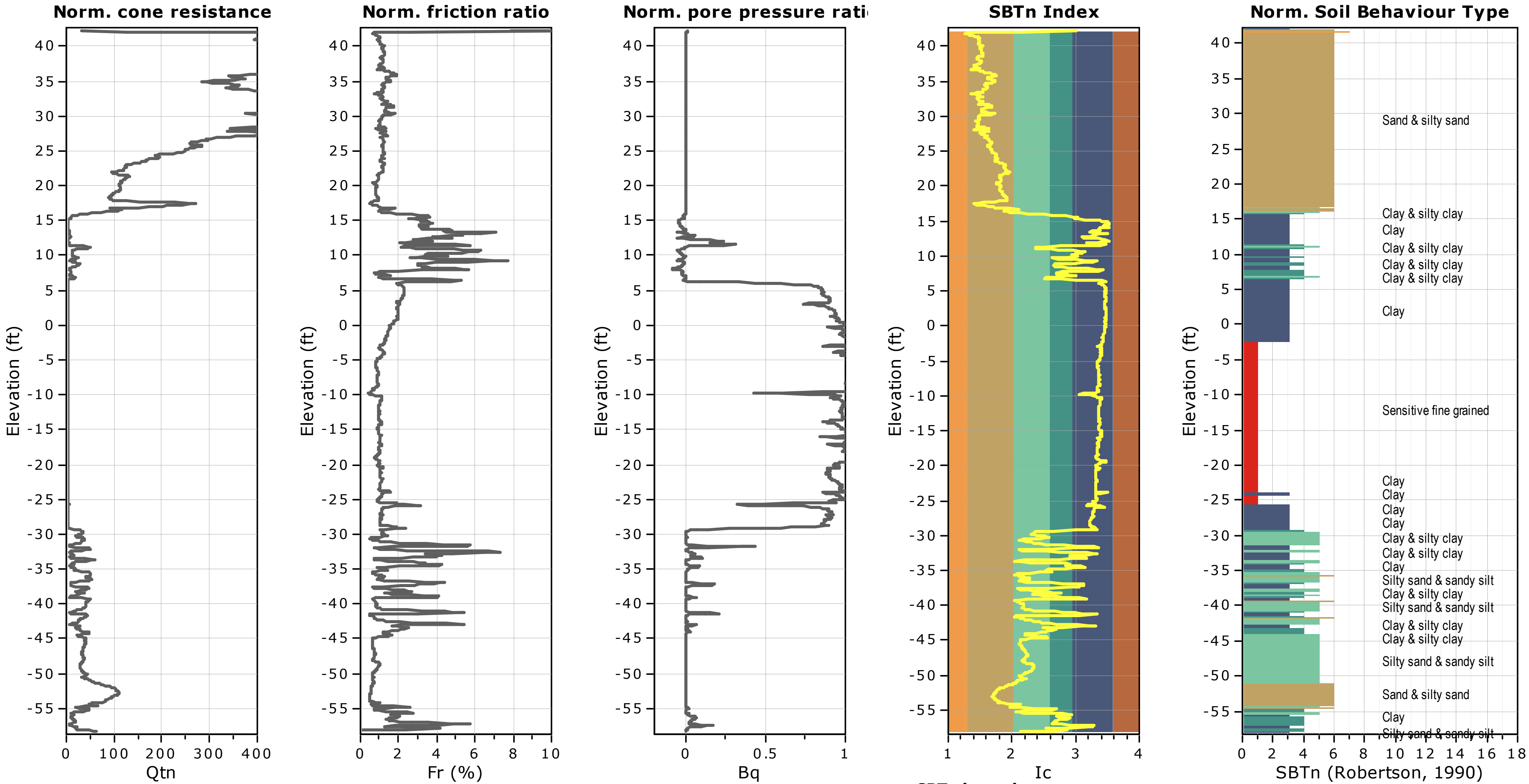
SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: Johnson Road Bridge
Location: Falmouth, Maine

CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
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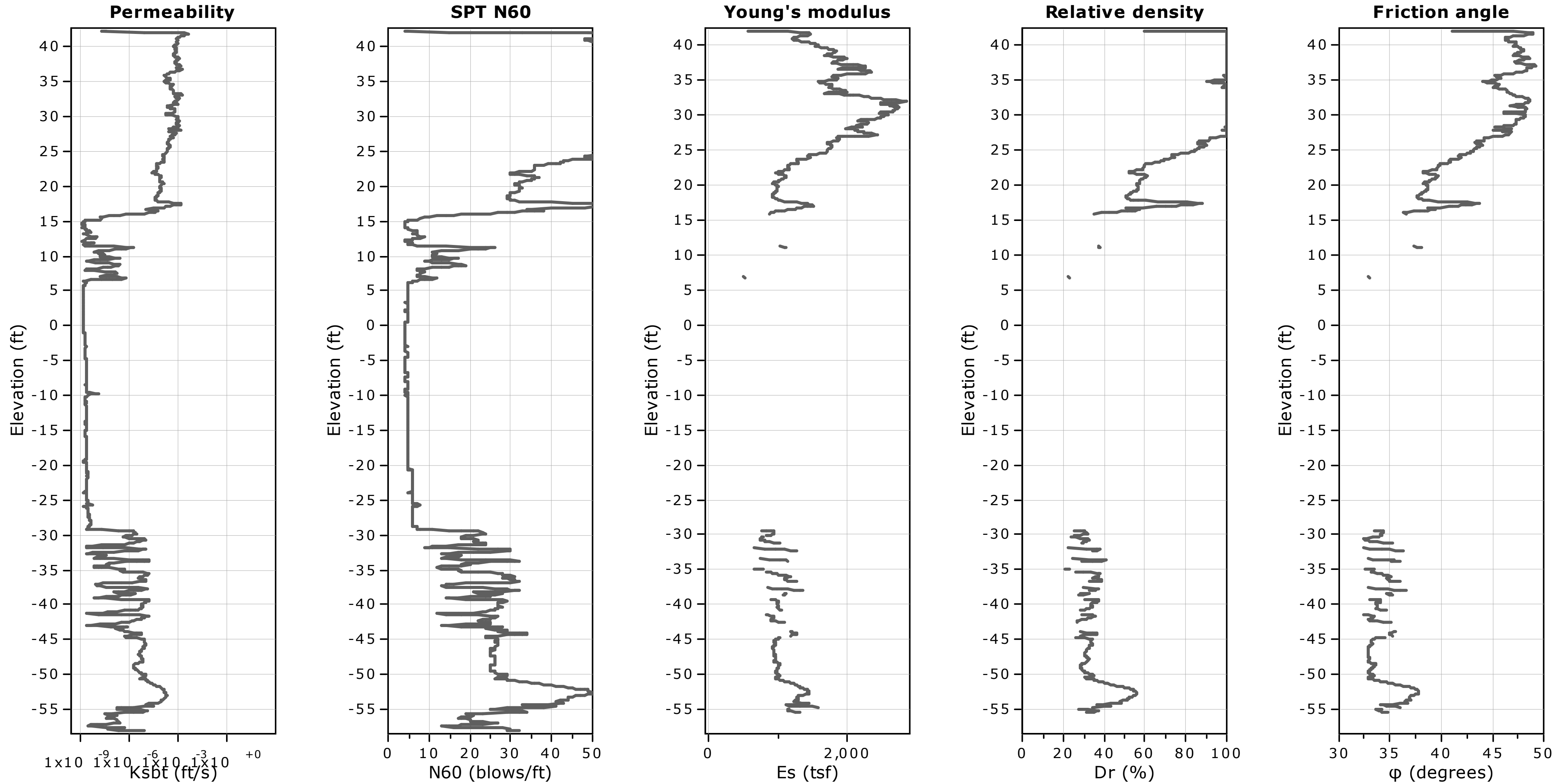
Cone Operator: Summit Geoengineering Services



Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

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Total depth: 105.39 ft, Date: 6/17/2019
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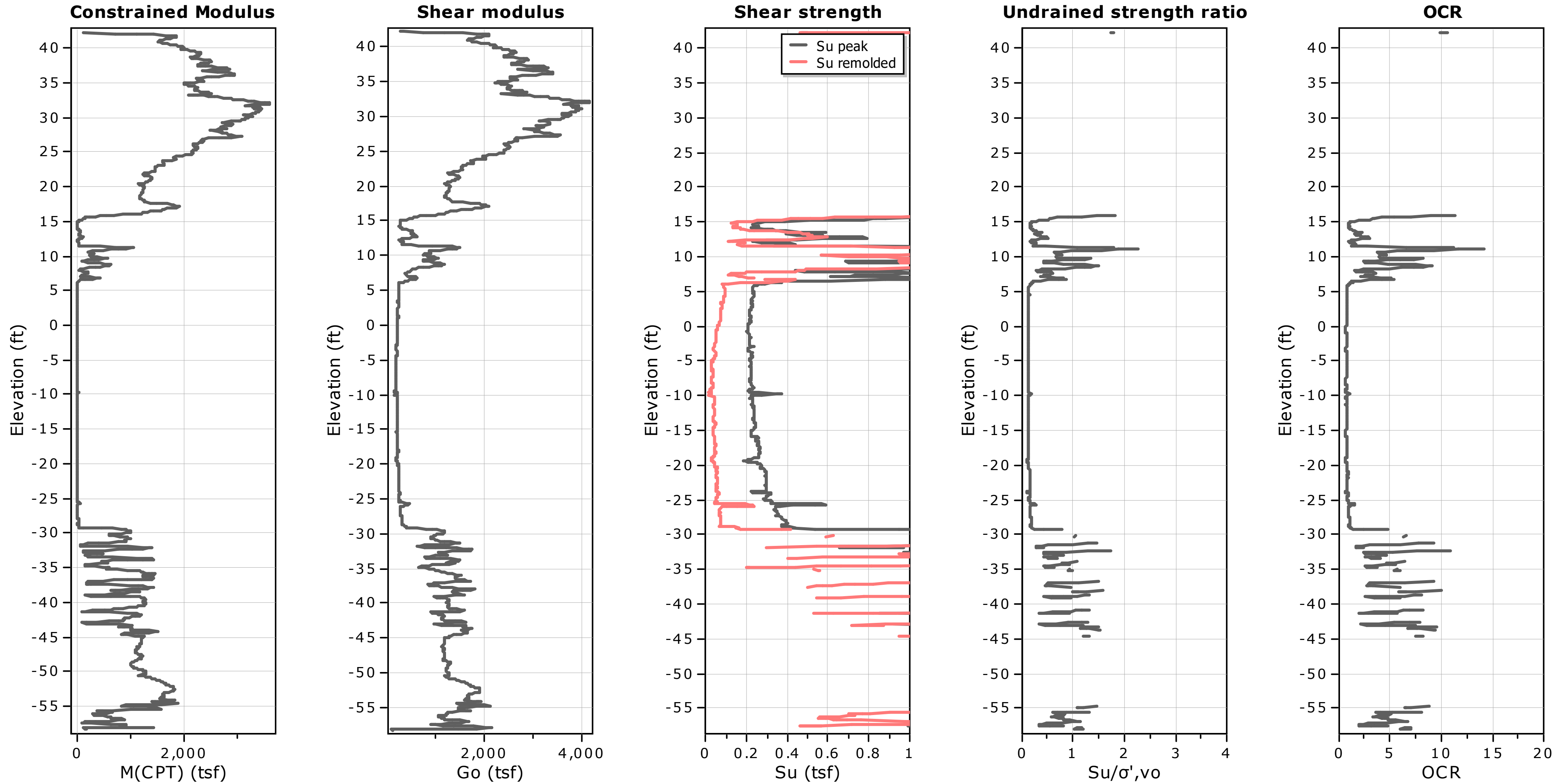
Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

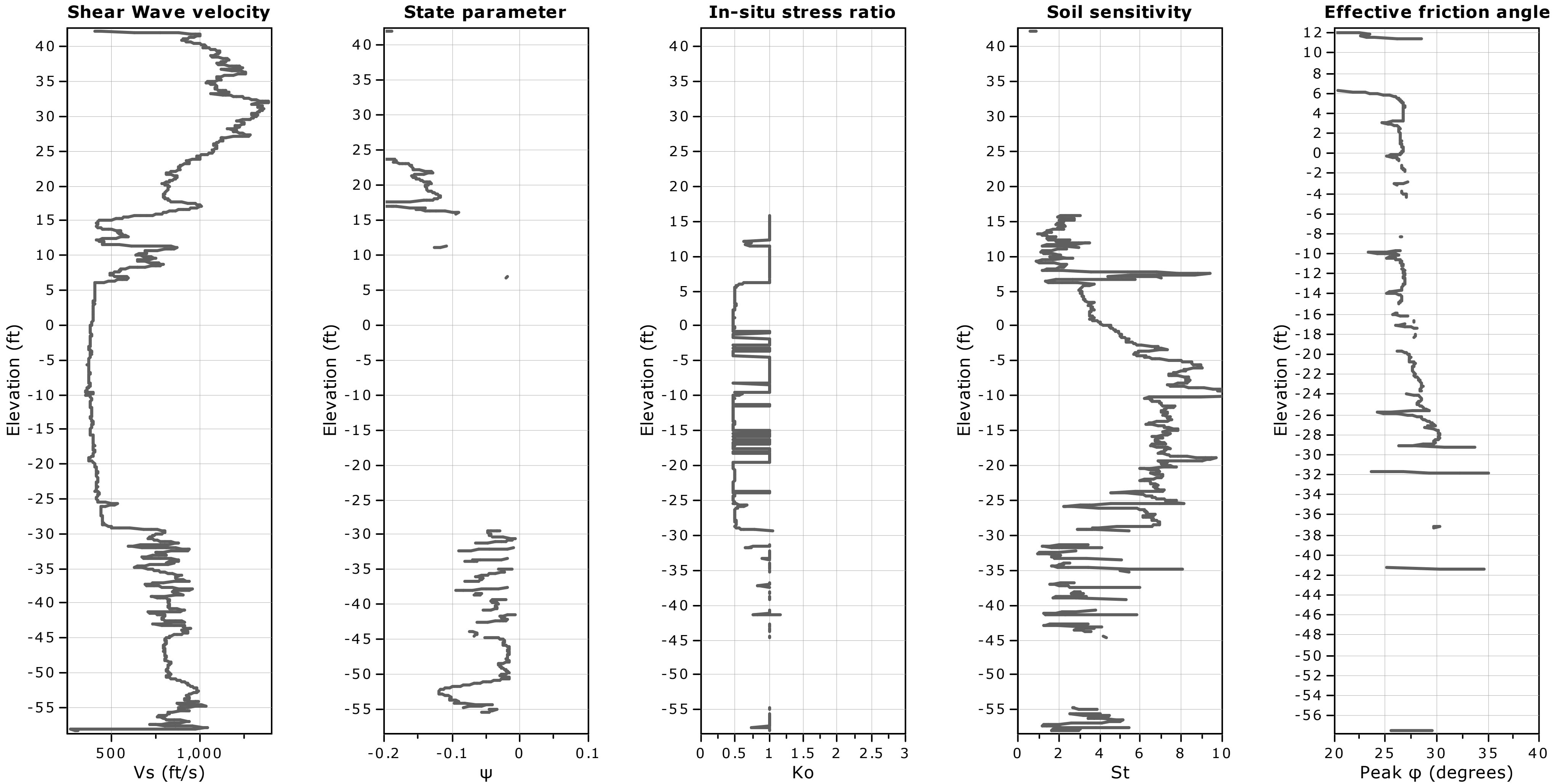


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 18
OCR factor for clays, N_{kt}: 0.33
Flat Dilatometer Test data

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



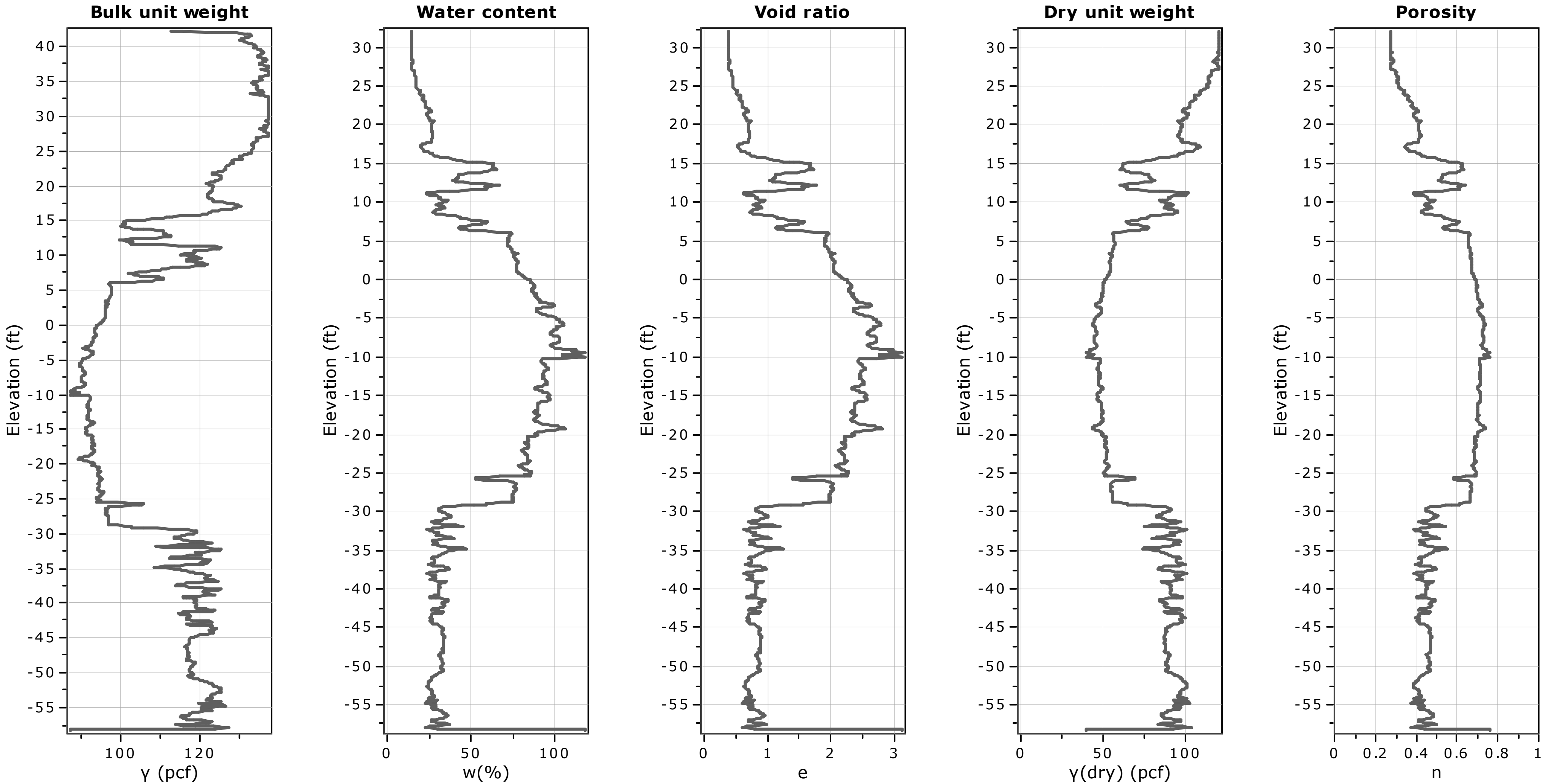
Calculation parameters

Soil Sensitivity factor, N_S: 7.00

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

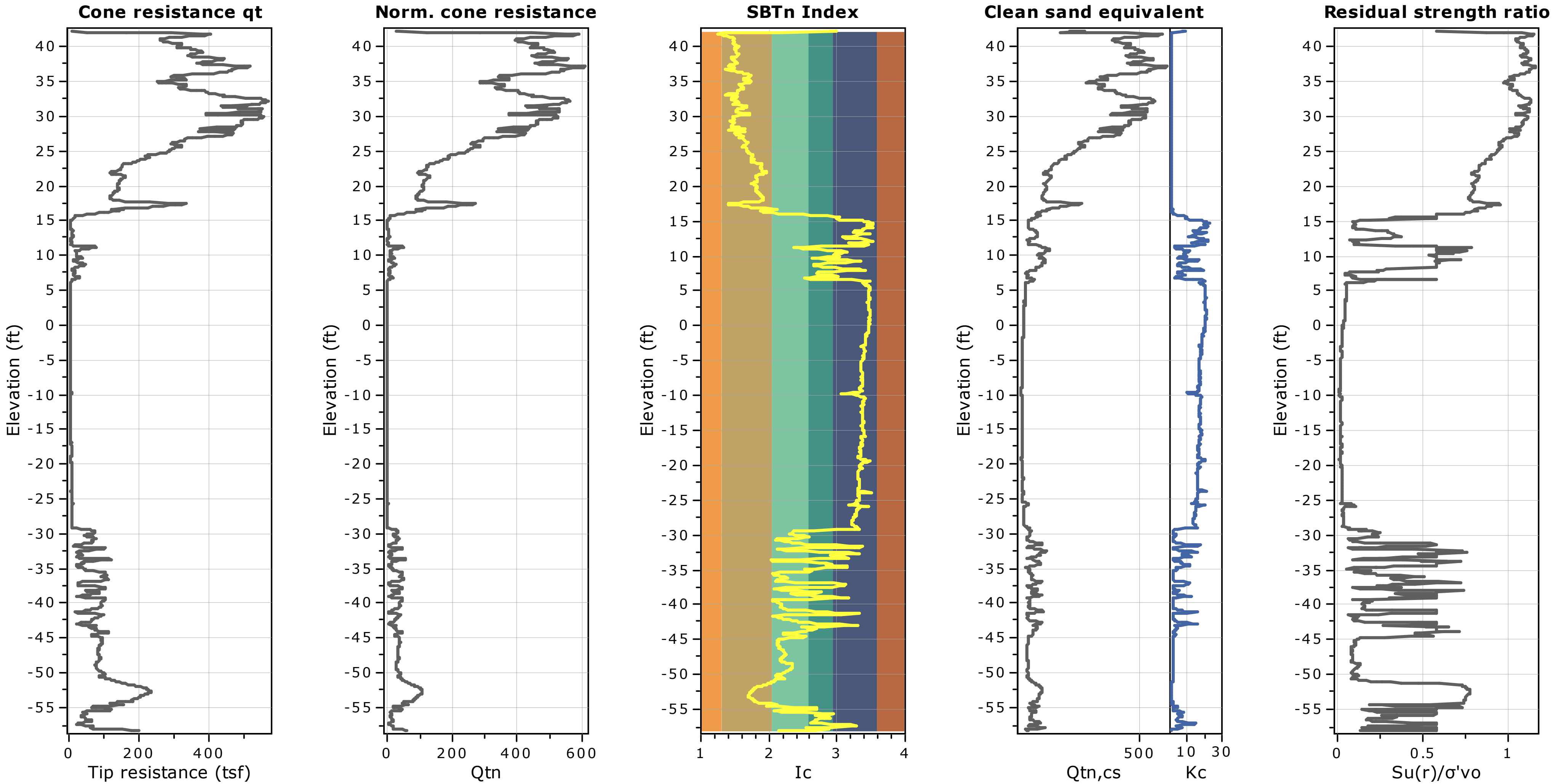
Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

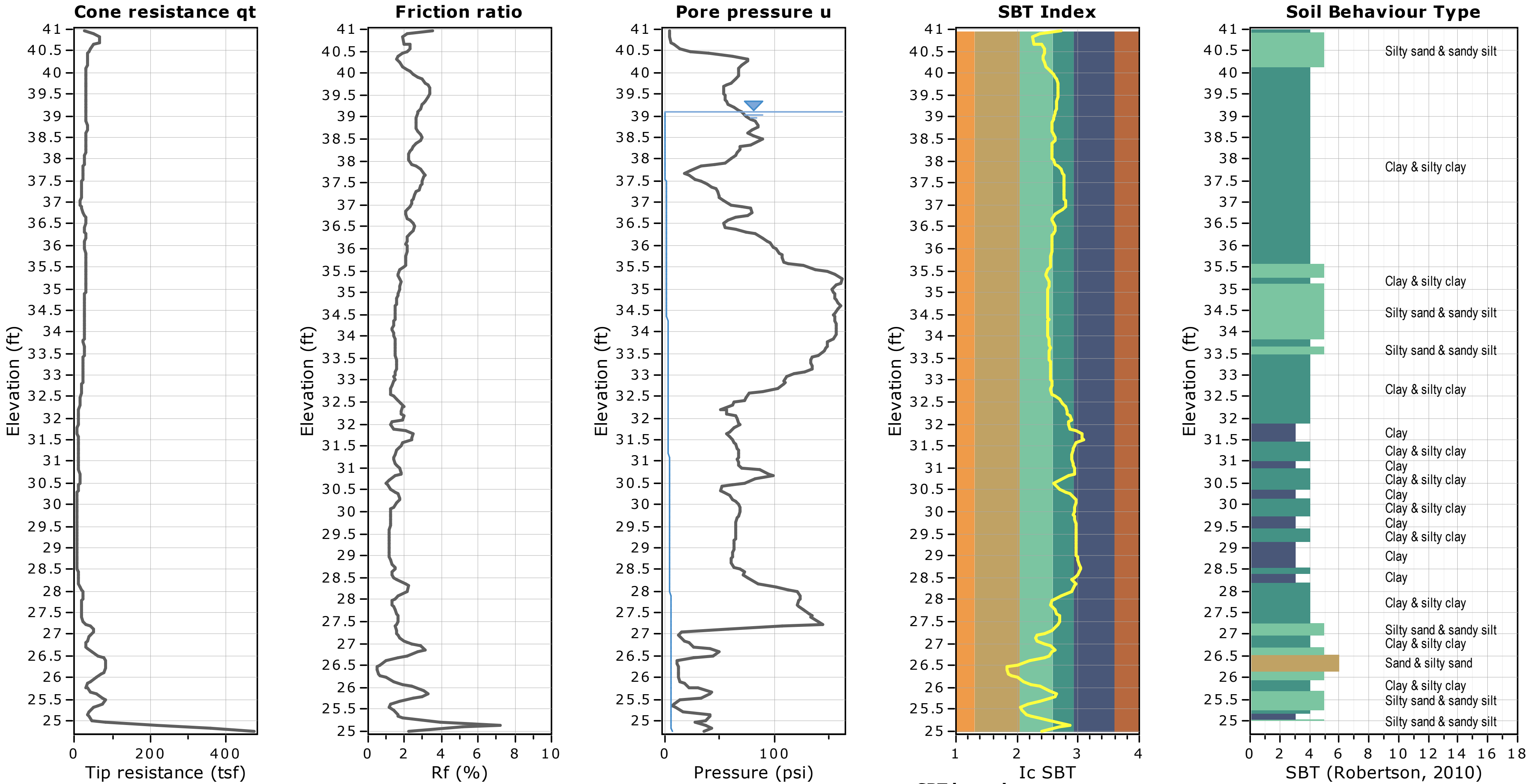
CPT-FJR-104
Total depth: 105.39 ft, Date: 6/17/2019
Surface Elevation: 47.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



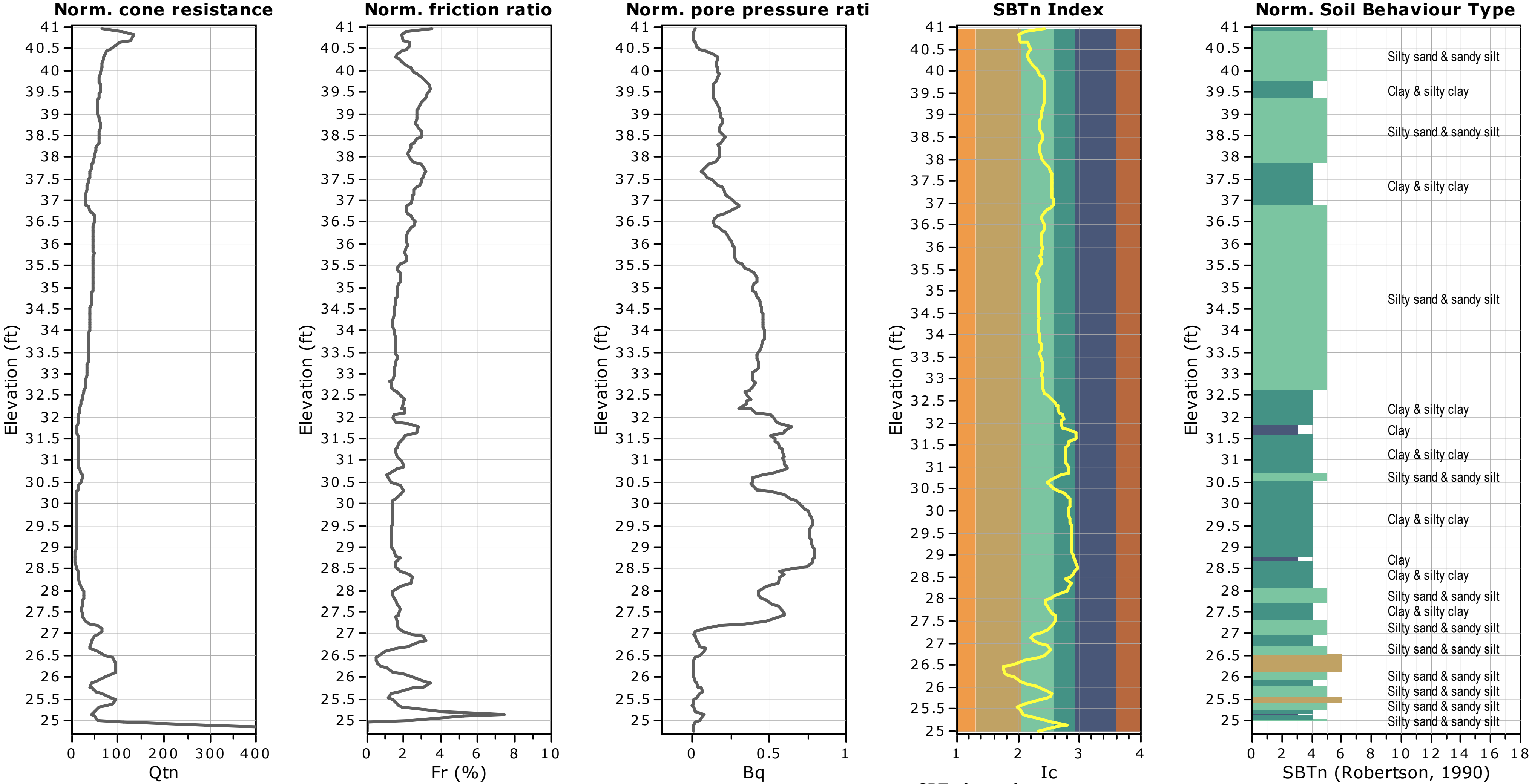
SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services



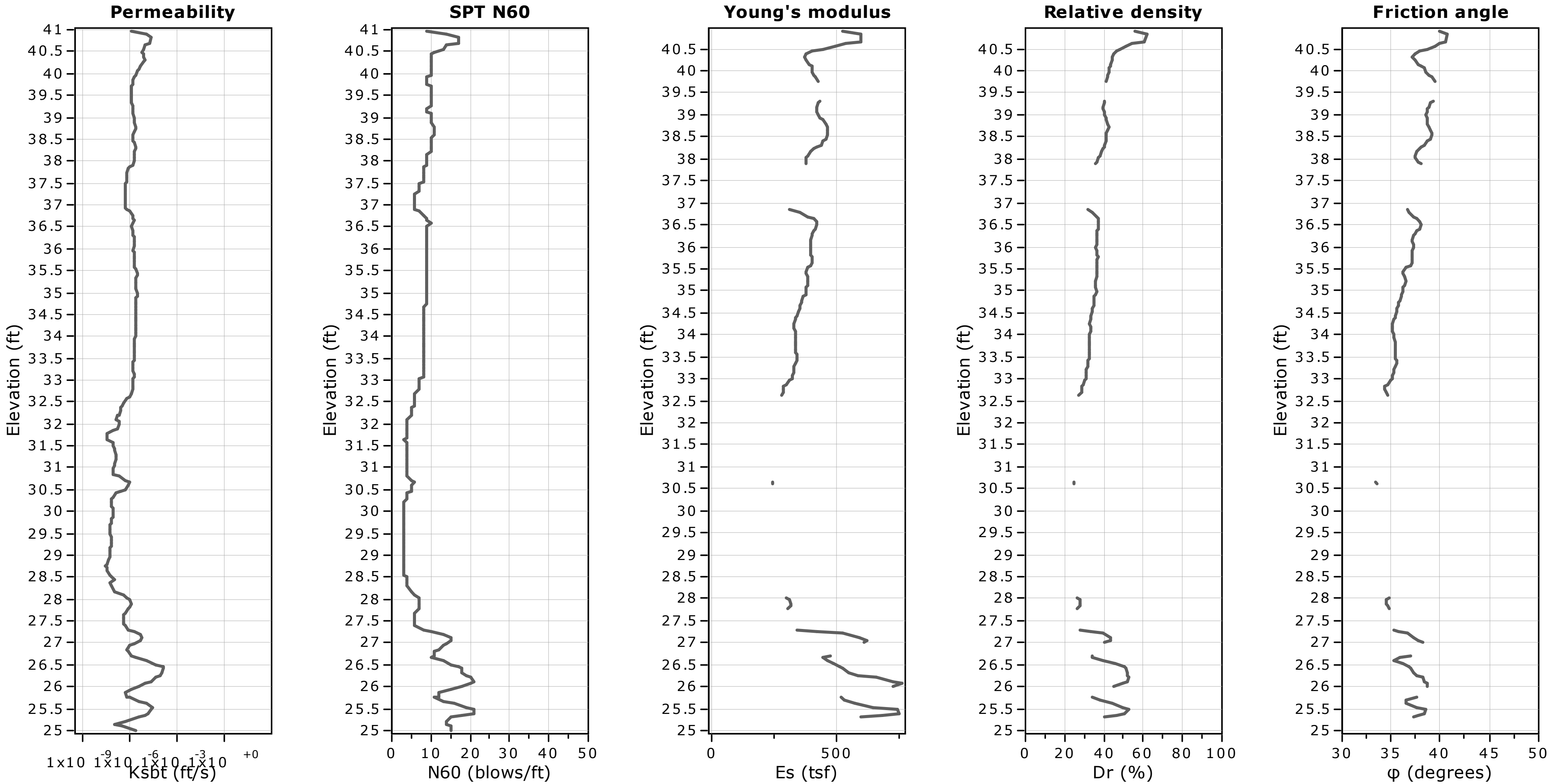
SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**

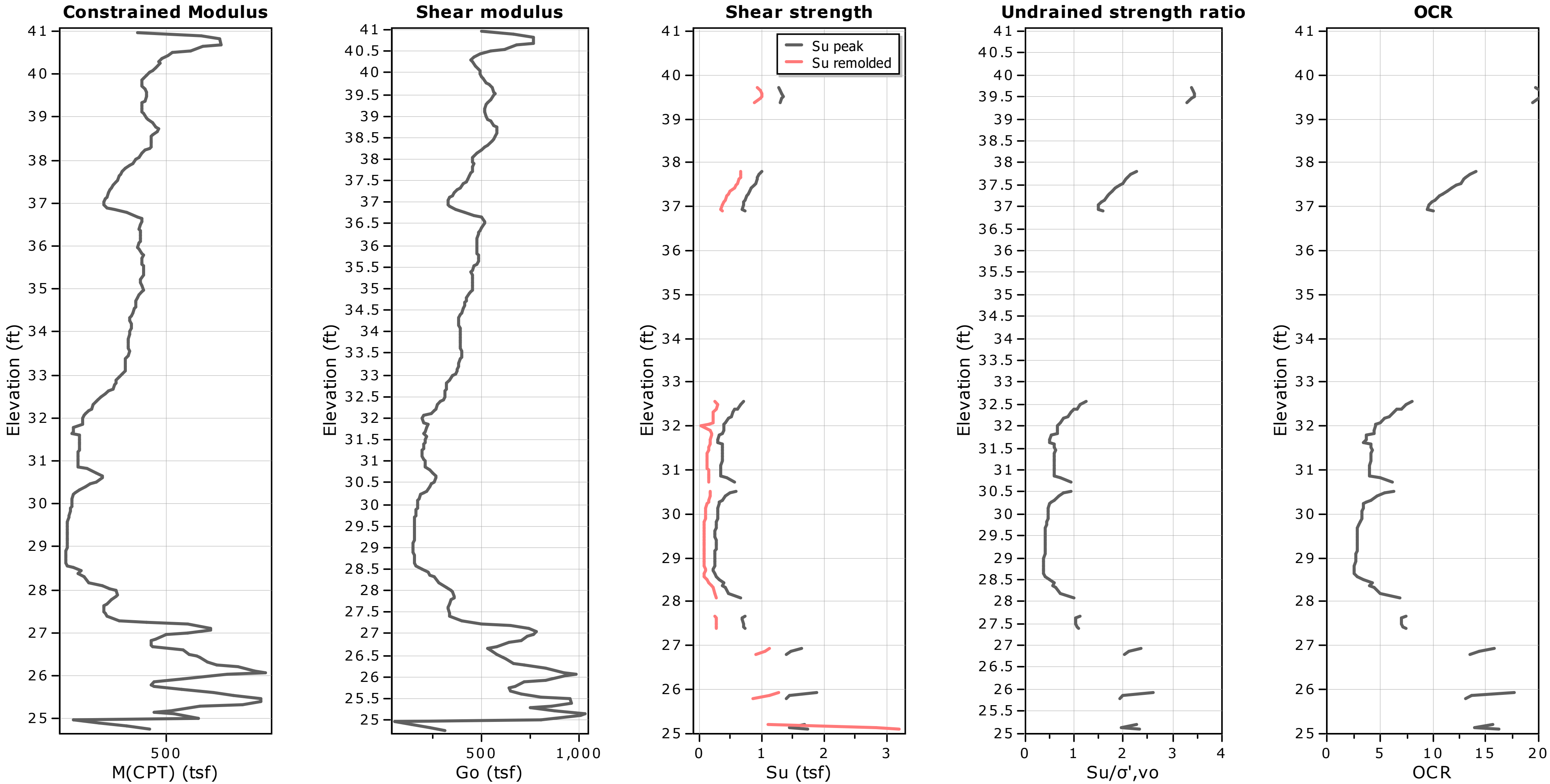


Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

Project: Johnson Road Bridge
Location: Falmouth, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

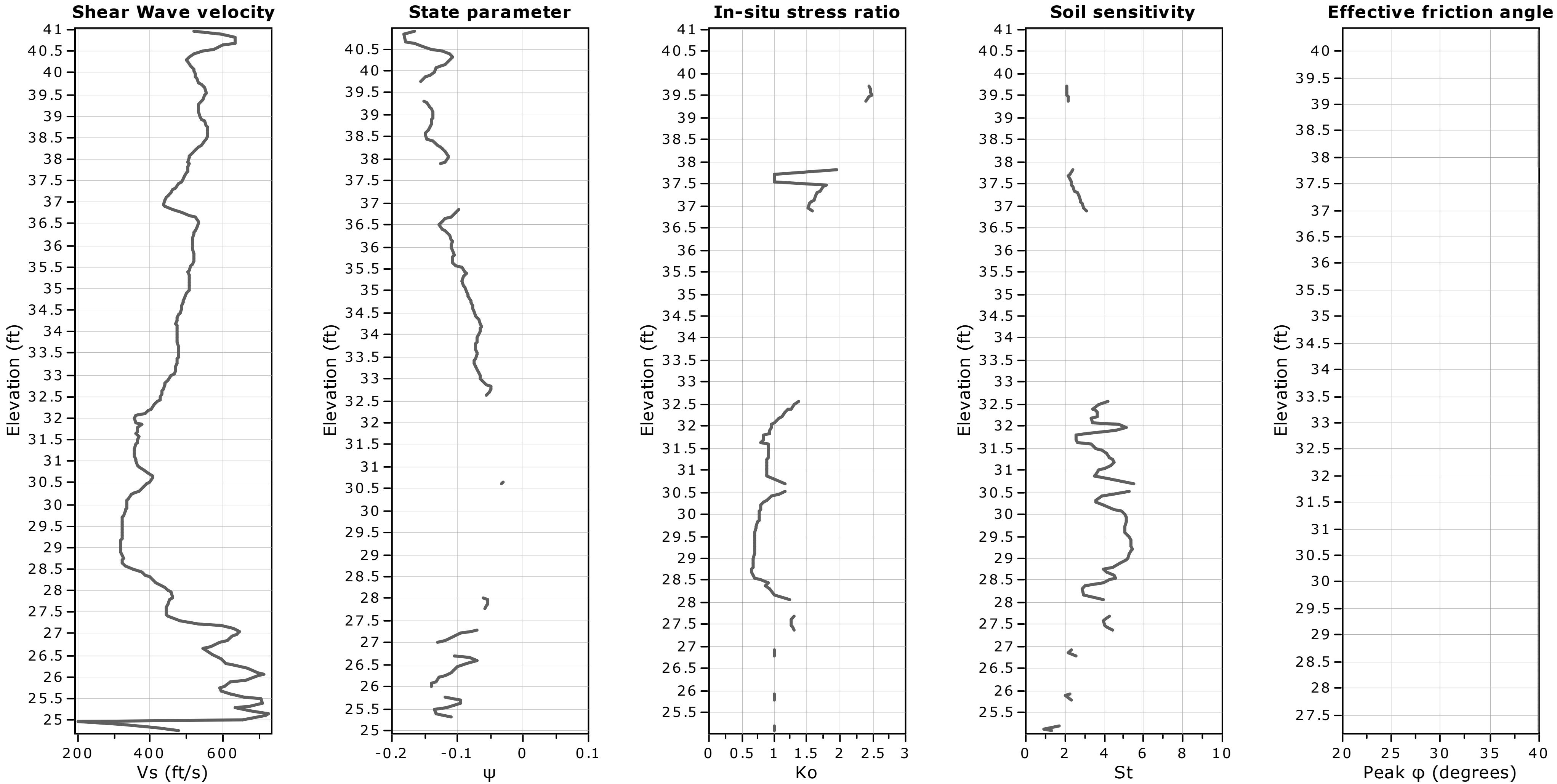


Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 22
OCR factor for clays, N_{kt}: 0.33
Flat Dilatometer Test data

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



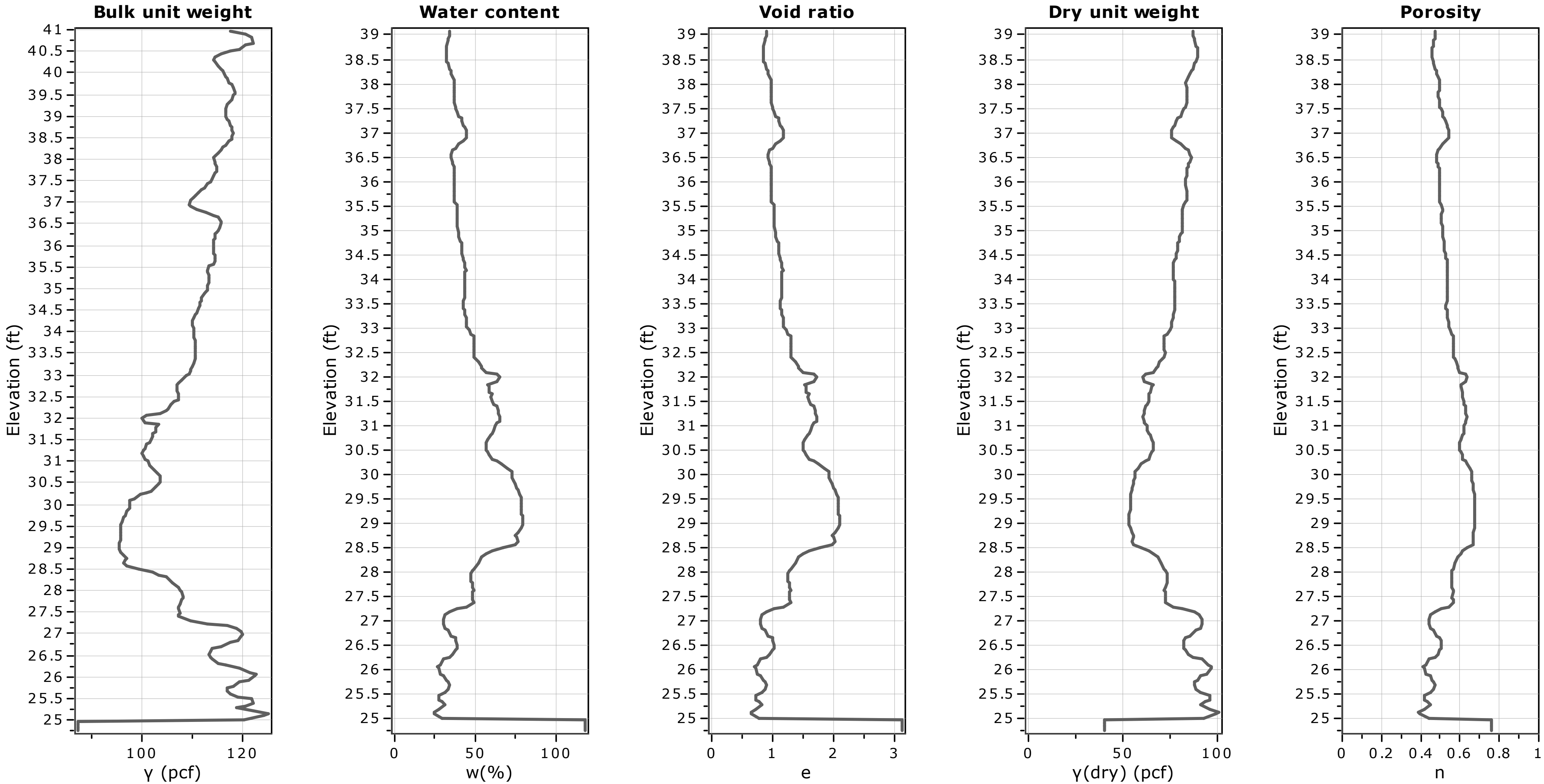
Calculation parameters

Soil Sensitivity factor, N_S: 7.00

GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

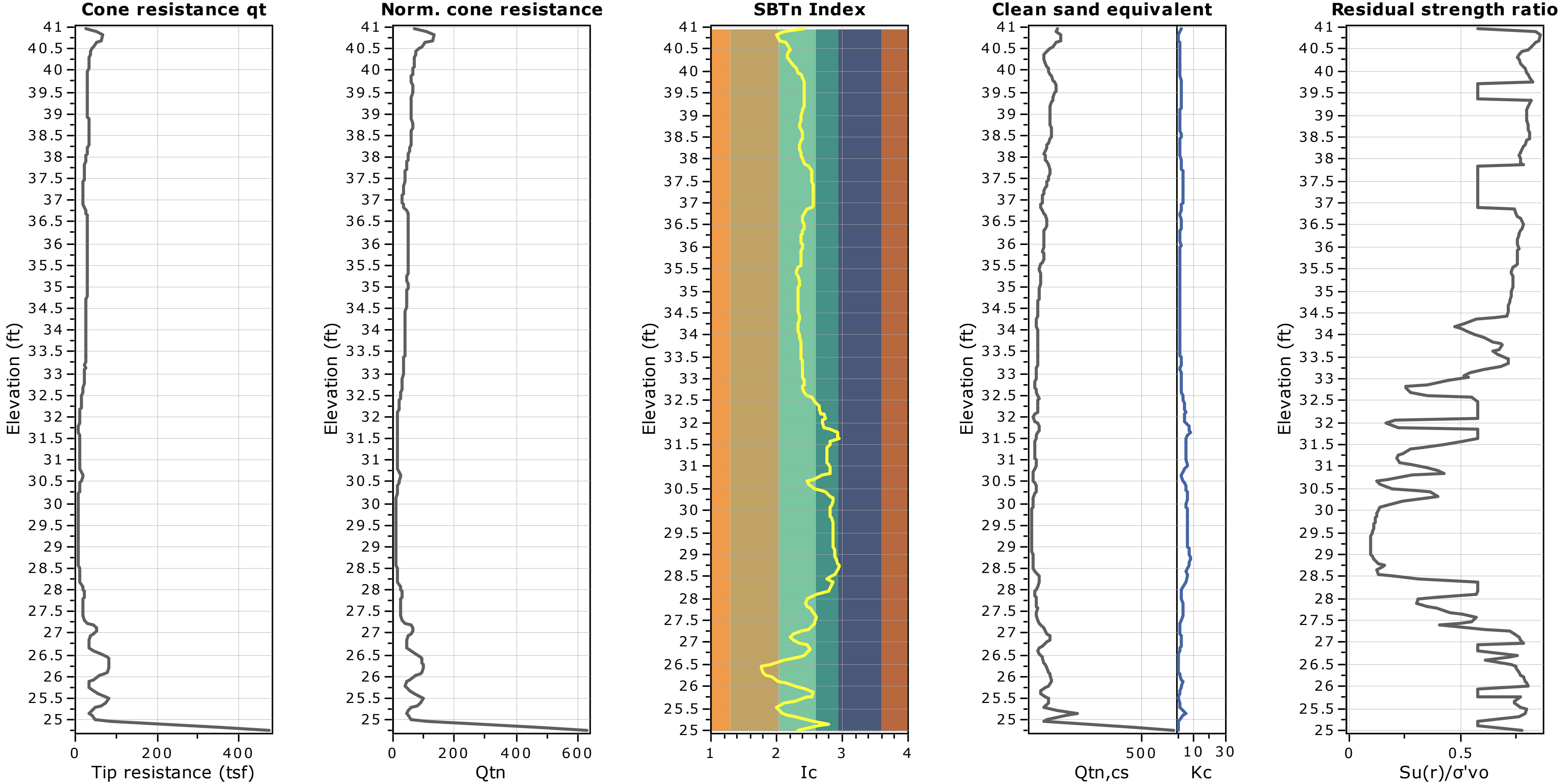
Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



GZA GeoEnvironmental Inc.
Geotechnical Engineers
South Portland, Maine

CPT-FJR-105
Total depth: 21.34 ft, Date: 6/17/2019
Surface Elevation: 46.10 ft
Cone Type: Vertek Digital Cone
Cone Operator: Summit Geoengineering Services

Project: **Johnson Road Bridge**
Location: **Falmouth, Maine**



Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$$S_{u(rem)} = f_s$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

• Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012

• Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

• N Barounis, J Philpot, Estimation of in-situ water content, void ratio, dry unit weight and porosity using CPT for saturated sands, Proc. 20th NZGS Geotechnical Symposium

APPENDIX D – LABORATORY TEST RESULTS

THIELSCH ENGINEERING

195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

Client Information:
GZA GeoEnvironmental
Portland, ME
PM: B. Cardali
Assigned By: B. Cardali
Collected By: M. Walsh

Project Information:
Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
GZA Project Number: 09.0026024.00

Report Date: 09.04.19

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	
				D2216	D4318		D6913			D2874	D854			D1557						
BB-FJR-101	7D	19-21	S-1	45.8	42	23														Gray CLAY & SILT
BB-FJR-101	8D	24-26	S-2	40.1																Water Content Only
BB-FJR-101	11D	39-41	S-3	41.2	46	22														Gray Silty CLAY
BB-FJR-103	1D	1.5-3.5	S-4	2.6			30.2	59.8	10.0											Light Brown f-c SAND, some f-c Gravel, trace Silt
BB-FJR-103	2D	5-7	S-5	13.8			0.0	93.4	6.6											Light Brown f-m SAND, trace Silt
BB-FJR-103	6D	23-35	S-6	17.9			0.0	94.7	5.3											Light Brown f-m SAND, trace Silt
BB-FJR-103	8D	27-29	S-7	15.2			0.0	52.1	47.9											Dark Brown CLAYEY SILTY fine to medium SAND
BB-FJR-103	10D	35-37	S-8	30.6			0.0	41.7	58.3											Dark Brown SANDY Clayey SILT
BB-FJR-103	11D	40-42	S-9	41.9																Water Content Only
BB-FJR-103	12D	47-49	S-10	36.7																Water Content Only
BB-FJR-103	13D	50-52	S-11	30.0																Water Content Only
BB-FJR-103	15D	60-62	S-12	38.9	47	22														Gray Silty CLAY

Date Received: *08.26.19*

Reviewed By:

Date Reviewed: *09.05.19*

THIELSCH
ENGINEERING

195 Frances Avenue
Cranston RI, 02910
Phone: (401)-467-6454
Fax: (401)-467-2398
thielsch.com
Let's Build a Solid Foundation

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Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
GZA Project Number: 09.0026024.00

Report Date: 09.04.19

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	
				D2216	D4318		D6913			D2874	D854			D1557						
BB-FJR-103	18D	80-82	S-13				0.0	78.1	21.9											Light Brown fine SAND, some Silt
BB-FJR-103	23D	106-108	S-14				27.3	54.5	18.2											Light Gray f-c SAND, some f-c Gravel, little Silt
BB-FJR-105	1D	2-4	S-15	17.7			3.9	87.0	9.1											Brown f-m SAND, trace Silt, trace coarse Gravel
BB-FJR-105	3D	12-14	S-16	29.6																Water Content Only
BB-FJR-105	5D	22-24	S-17	41.3	45	22														Gray Silty CLAY
BB-FJR-105	7D	33-35	S-18	38.9																Water Content Only
BB-FJR-105	9D	43-45	S-19	37.8																Water Content Only
BB-FJR-106	1D	1-3	S-20	2.3			28.1	65.1	6.8											Light Brown f-c SAND, some f-c Gravel, trace Silt
BB-FJR-106	2D	5-7	S-21	6.3			7.3	84.2	8.5											Brown f-m SAND, trace Silt, trace f-c Gravel
BB-FJR-106	8D	28-30	S-22	30.7																Water Content Only
BB-FJR-106	9D	30-32	S-23	33.3	39	20														Gray Silty CLAY
BB-FJR-102A	4D	25-27	S-24	36.6																Water Content Only

Date Received: *08.26.19*

Reviewed By:

Date Reviewed: *09.05.19*

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: GZA GeoEnvironmental Portland, ME PM: B. Cardali Assigned By: B. Cardali Collected By: M. Walsh	Project Information: **Johnson Road Bridge No. 5792, MEDOT WIN 21721.00** **Falmouth, ME** GZA Project Number: 09.0026024.00 Summary Page: 3 of 3 Report Date: 09.04.19

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests									Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G_s		Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W_{opt} (%)	γ_d MAX (pcf) W_{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Perme-ability cm/sec	
				D2216	D4318		D6913			D2874	D854				D1557						
BB-FJR-102A	5D	35-37	S-25	39.8																	Water Content Only
BB-FJR-102A	6D	45-47	S-26	25.3																	Water Content Only
BB-FJR-104	2D	4-6	S-27	6.6			7.5	85.0	7.5												Light Brown f-m SAND, trace fine Gravel, trace Silt
BB-FJR-104	4D	15-17	S-28	29.2																	Water Content Only
BB-FJR-104	5D	20-22	S-29	33.1	42	22															Gray Silty CLAY
BB-FJR-104	6D	25-27	S-30	43.2																	Water Content Only
BB-FJR-104	7D	35-37	S-31	28.8																	Water Content Only
BB-FJR-104	8D	50-52	S-32	40.4																	Water Content Only
BB-FJR-104	10D	60-62	S-33	28.6			0.0	54.9	45.1												Gray Silty fine to medium SAND

Date Received: *08.26.19* Reviewed By: [signature] Date Reviewed: *09.05.19*

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Johnson Road
Bridge #5792
Town(s): Falmouth, ME

MDOT Project Number:

GZA Project Number: 09.0026024.00

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Lab Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-FJR-103		1D	1.5-3.5	S-4		2.6			SP-SM	A-1-b	II
BB-FJR-103		2D	5-7	S-5		13.8			SP-SM	A-3	II
BB-FJR-103		6D	23-25	S-6					SP-SM	A-3	0
BB-FJR-103		8D	27-29	S-7		15.2			SM	A-4	III
BB-FJR-103		10D	35-37	S-8		30.6			MH	A-5(1)	IV
BB-FJR-103		18D	80-82	S-13					SM	A-2-4(0)	II
BB-FJR-103		23D	106-108	S-14					SM	A-1-b	II
BB-FJR-105		1D	2-4	S-15		17.7			SP-SM	A-3	II
BB-FJR-106		1D	1-3	S-20					SP-SM	A-1-b	II
BB-FJR-106		2D	5-7	S-21					SP-SM	A-3	II
BB-FJR-104		2D	4-6	S-27		6.6			SP-SM	A-3	II
BB-FJR-104		10D	60-62	S-33		28.6			SM	A-4	III

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MDOT and Corps of Engineers Classification Systems.

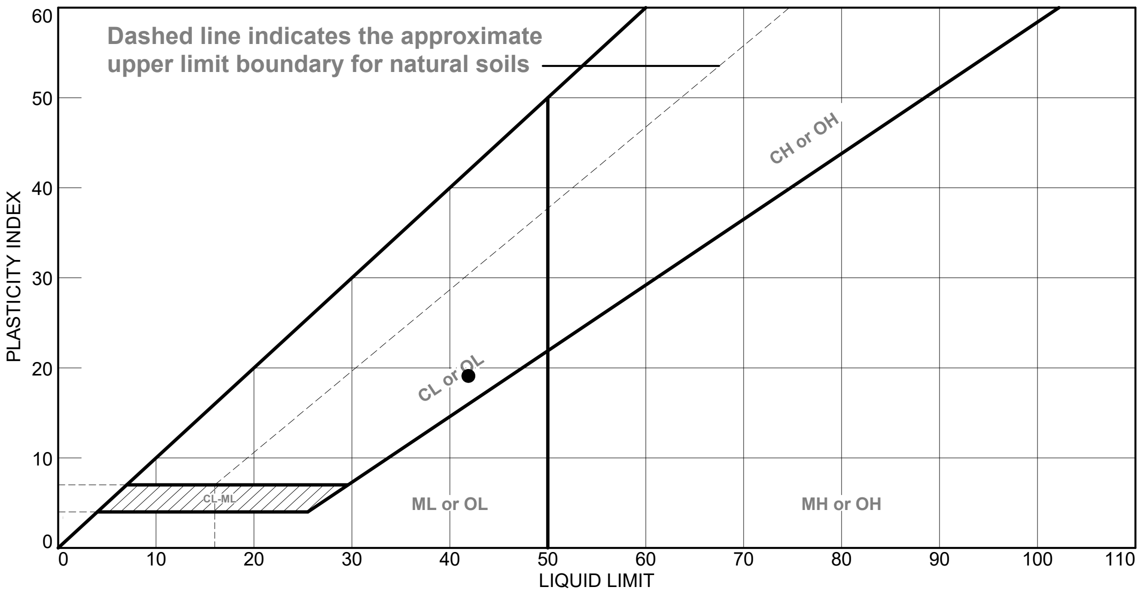
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

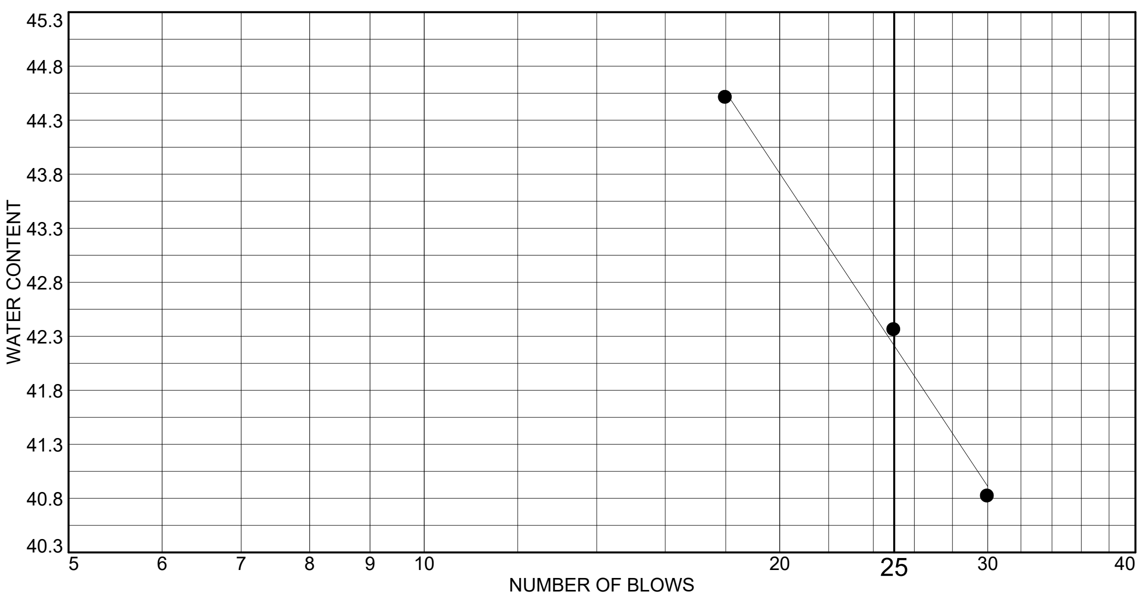
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray CLAY & SILT	42	23	19			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-101 **Depth:** 19-21'

Sample Number: 7D

Thielsch Engineering Inc.

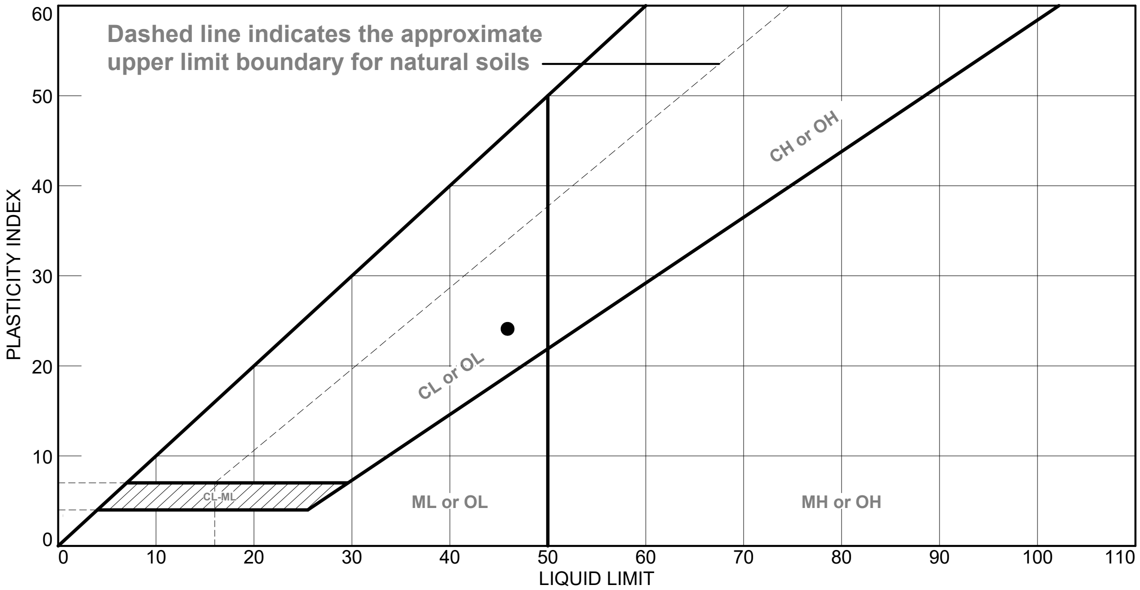
Cranston, RI

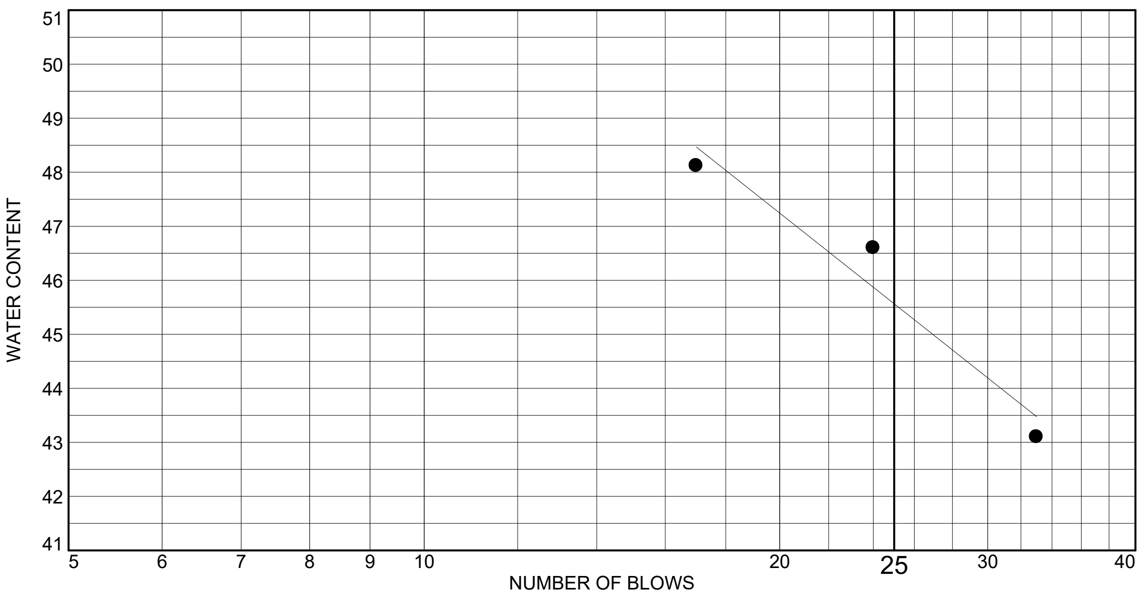
Remarks:

Figure L-1

Tested By: JM **Checked By:** SA

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray Silty CLAY	46	22	24			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-101 **Depth:** 39-41'

Sample Number: 11D

Thielsch Engineering Inc.

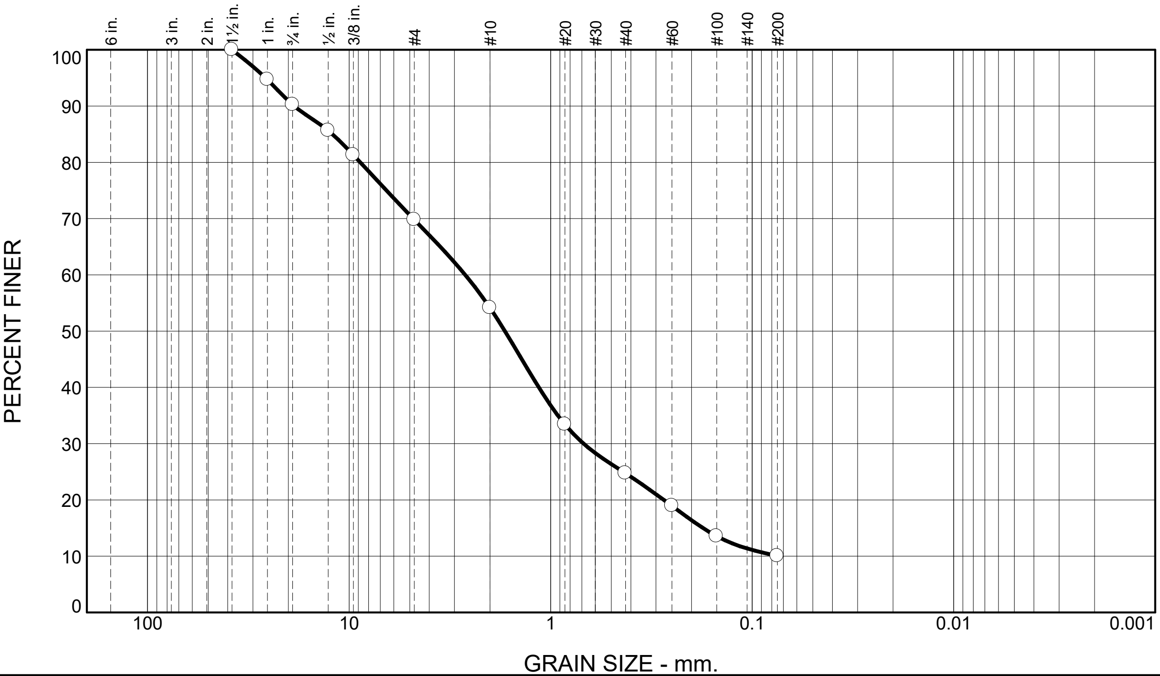
Cranston, RI

Remarks:

Figure L-3

Tested By: JM **Checked By:** SA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.8	20.4	15.7	29.4	14.7	10.0	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	94.7		
3/4"	90.2		
1/2"	85.7		
3/8"	81.3		
#4	69.8		
#10	54.1		
#20	33.4		
#40	24.7		
#60	18.9		
#100	13.6		
#200	10.0		

* (no specification provided)

Material Description

Light Brown f-c SAND, some f-c Gravel, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 18.6952 D_{85}= 12.0924 D_{60}= 2.6513
D_{50}= 1.6867 D_{30}= 0.6856 D_{15}= 0.1754
D_{10}= C_u= C_c=

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / JM / IA

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-FJR-103 **Depth:** 1.5-3.5' **Date Sampled:**
Sample Number: 1D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-4

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.9	25.6	66.9	6.6	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.1		
#20	92.9		
#40	73.5		
#60	43.9		
#100	15.9		
#200	6.6		

* (no specification provided)

Material Description

Light Brown f-m SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.7198 D_{85}= 0.5844 D_{60}= 0.3264
D_{50}= 0.2757 D_{30}= 0.1993 D_{15}= 0.1464
D_{10}= 0.1213 C_u= 2.69 C_c= 1.00

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-FJR-103 **Depth:** 5-7' **Date Sampled:**
Sample Number: 2D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-5

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.4	28.3	65.0	5.3	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	98.6		
#20	91.9		
#40	70.3		
#60	34.6		
#100	11.0		
#200	5.3		

* (no specification provided)

Material Description

Light Brown f-m SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.7581 D_{85}= 0.6080 D_{60}= 0.3603
D_{50}= 0.3125 D_{30}= 0.2321 D_{15}= 0.1702
D_{10}= 0.1333 C_u= 2.70 C_c= 1.12

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

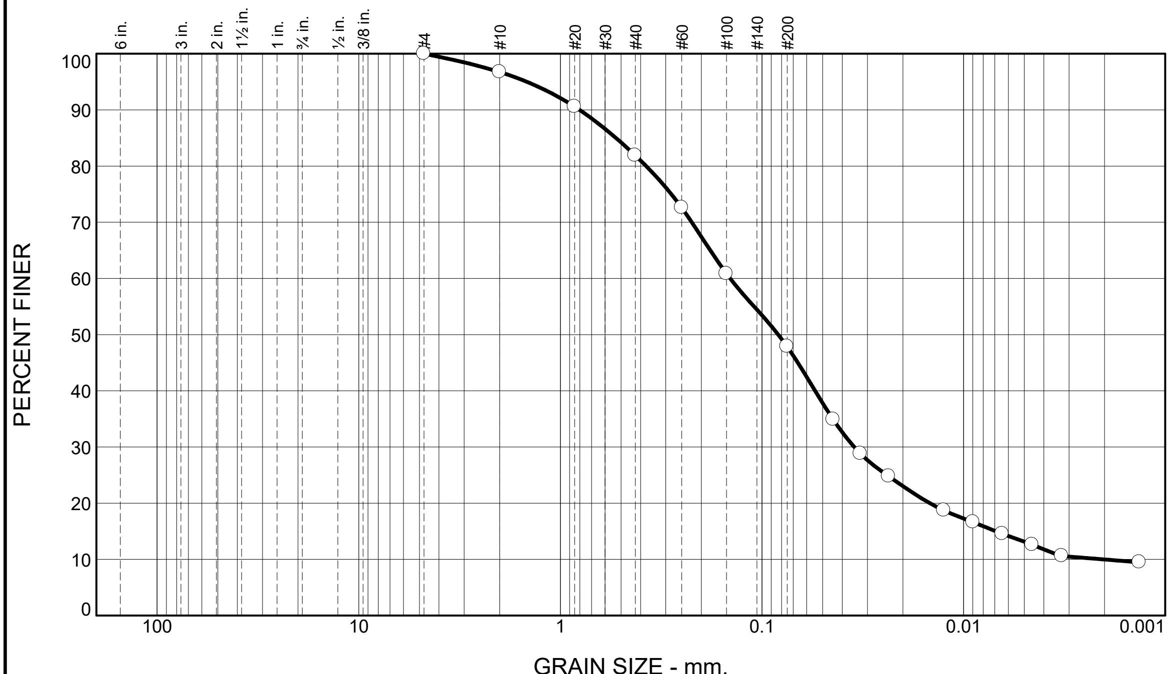
Source of Sample: BB-FJR-103 **Depth:** 23-25' **Date Sampled:**
Sample Number: 6D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	3.3	14.8	34.0	37.9	10.0

Test Results (D7928 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	96.7		
#20	90.6		
#40	81.9		
#60	72.6		
#100	60.8		
#200	47.9		
0.0443 mm.	34.9		
0.0325 mm.	28.8		
0.0235 mm.	24.8		
0.0125 mm.	18.7		
0.0090 mm.	16.6		
0.0064 mm.	14.5		
0.0046 mm.	12.6		
0.0033 mm.	10.6		
0.0013 mm.	9.5		

* (no specification provided)

Material Description

Dark Brown CLAYEY SILTY fine to medium SAND

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4

Coefficients

D_{90}= 0.8028 D_{85}= 0.5299 D_{60}= 0.1439
D_{50}= 0.0830 D_{30}= 0.0349 D_{15}= 0.0069
D_{10}= 0.0020 C_u= 70.97 C_c= 4.17

Remarks

Sample visually classified as plastic. Sample rolled to 1/4".

Date Received: 8.26.19 **Date Tested:** 08.29.19

Tested By: JM/LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

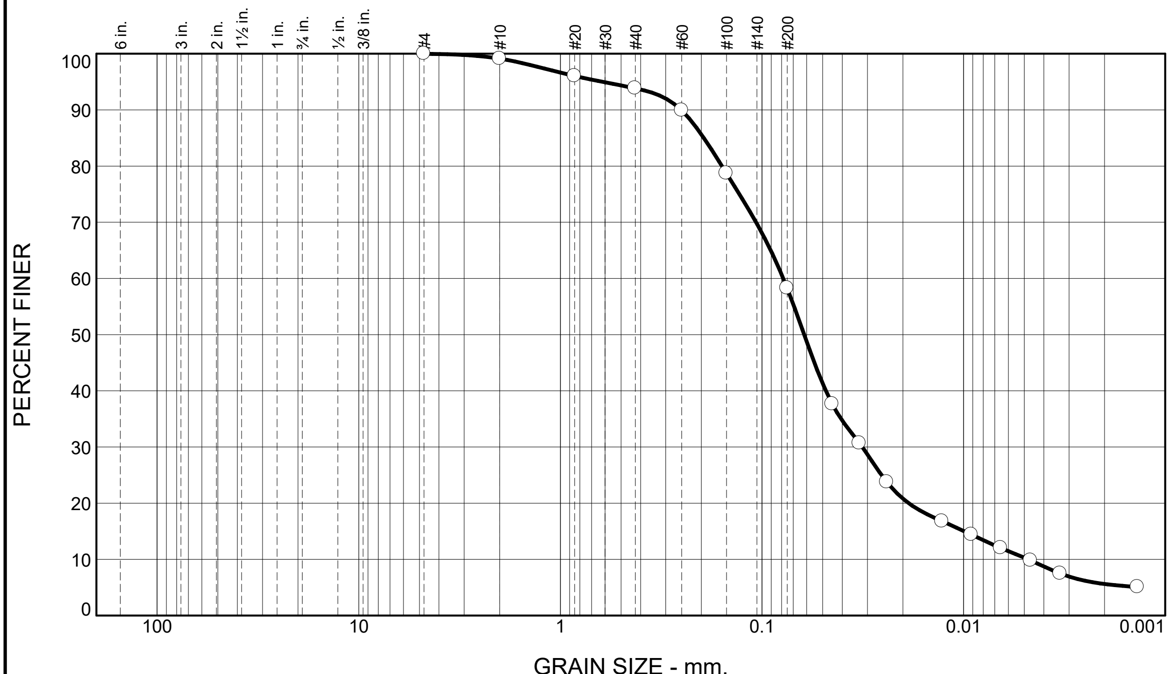
Source of Sample: BB-FJR-103 **Depth:** 27-29' **Date Sampled:**
Sample Number: 8D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.9	5.3	35.5	52.7	5.6

Test Results (D7928 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.1		
#20	96.0		
#40	93.8		
#60	89.9		
#100	78.7		
#200	58.3		
0.0448 mm.	37.6		
0.0328 mm.	30.7		
0.0240 mm.	23.7		
0.0128 mm.	16.8		
0.0091 mm.	14.4		
0.0065 mm.	12.0		
0.0047 mm.	9.8		
0.0033 mm.	7.5		
0.0014 mm.	5.1		

* (no specification provided)

Material Description

Dark Brown SANDY Clayey SILT

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= MH **AASHTO (M 145)=** A-5(1)

Coefficients

D_{90}= 0.2515 D_{85}= 0.1942 D_{60}= 0.0784
D_{50}= 0.0617 D_{30}= 0.0318 D_{15}= 0.0099
D_{10}= 0.0048 C_u= 16.39 C_c= 2.70

Remarks

Sample visually classified as plastic. Sample rolled to 1/4".

Date Received: 08.26.19 **Date Tested:** 08.29.19

Tested By: JM/LR

Checked By: Steven Accetta

Title: Laboratory Coordinator

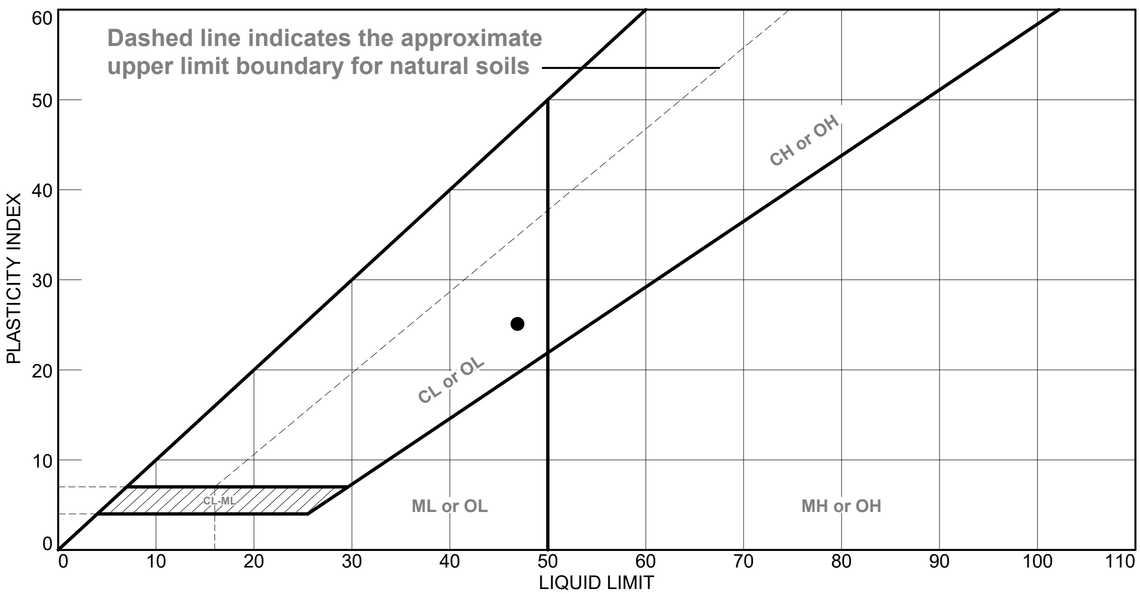
Source of Sample: BB-FJR-103 **Depth:** 35-37' **Date Sampled:**
Sample Number: 10D

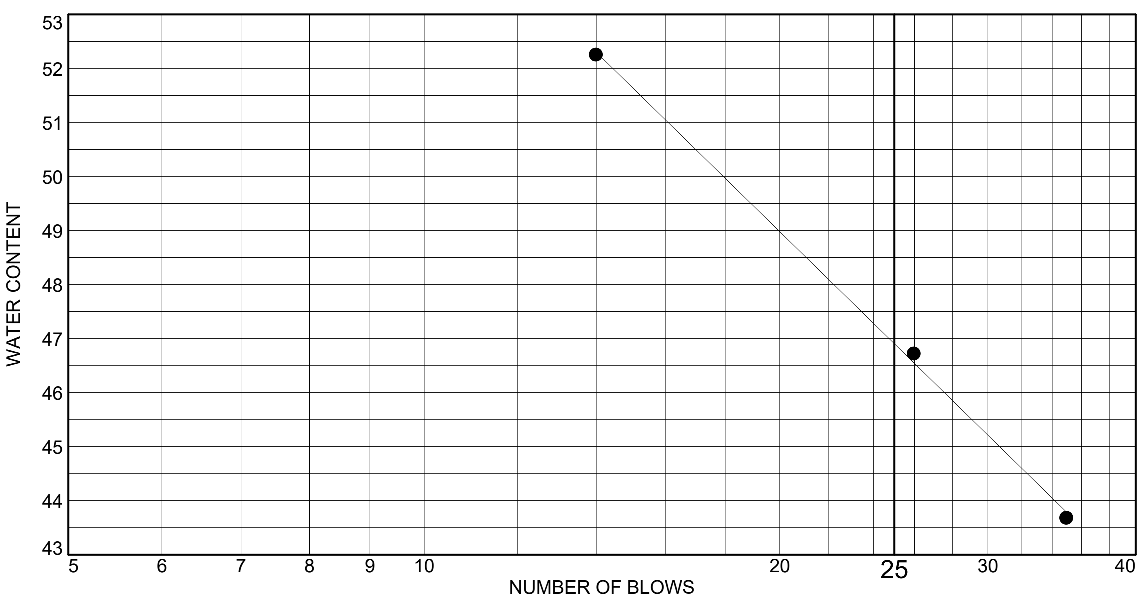
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-8

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray Silty CLAY	47	22	25			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-103 **Depth:** 60-62'

Sample Number: 15D

Remarks:

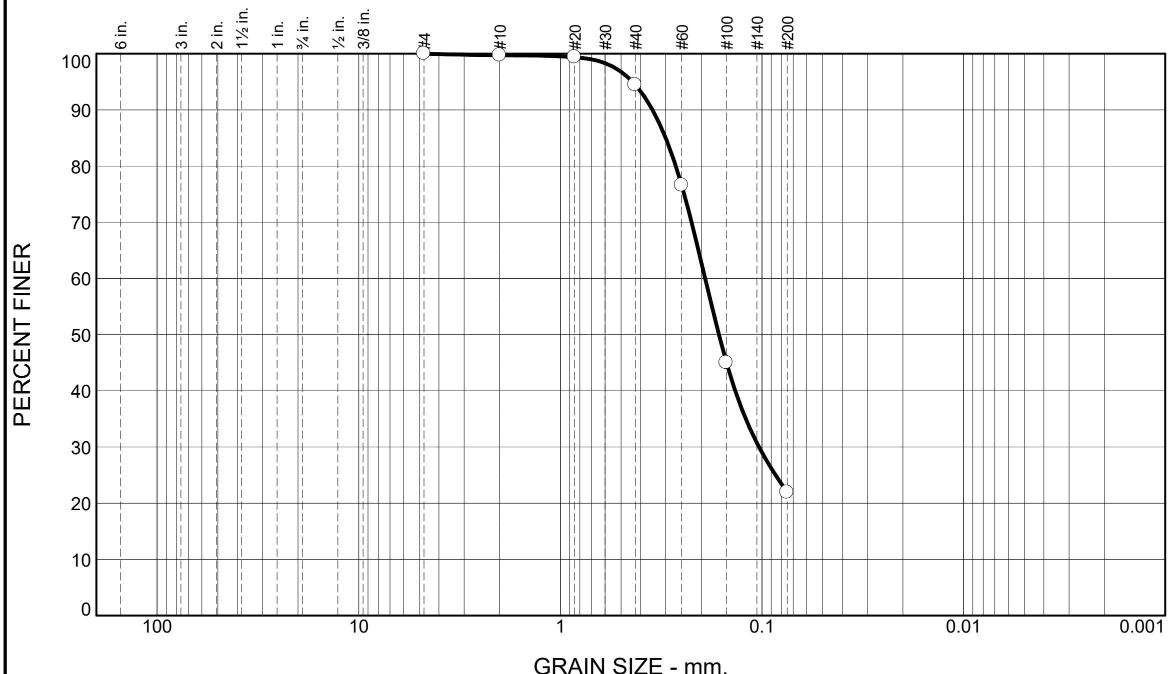
Thielsch Engineering Inc.

Cranston, RI

Figure L-12

Tested By: JM **Checked By:** SA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.3	5.2	72.6	21.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	99.7		
#20	99.4		
#40	94.5		
#60	76.6		
#100	45.0		
#200	21.9		

* (no specification provided)

Material Description

Light Brown fine SAND, some Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-2-4(0)

Coefficients

D_{90}= 0.3491 D_{85}= 0.3003 D_{60}= 0.1913
D_{50}= 0.1636 D_{30}= 0.1035 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

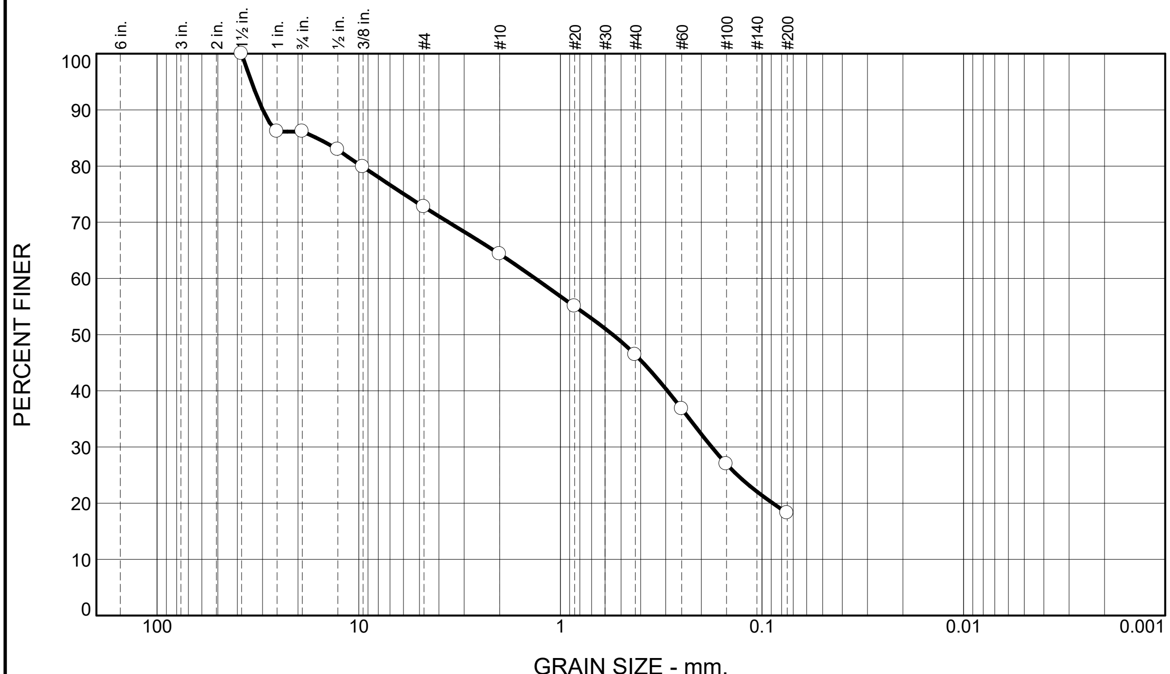
Source of Sample: BB-FJR-103 **Depth:** 80-82' **Date Sampled:**
Sample Number: 18D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.9	13.4	8.4	17.9	28.2	18.2	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1-1/2"	100.0		
1"	86.1		
3/4"	86.1		
1/2"	82.9		
3/8"	79.8		
#4	72.7		
#10	64.3		
#20	55.0		
#40	46.4		
#60	36.8		
#100	27.0		
#200	18.2		

* (no specification provided)

Material Description

Light Gray f-c SAND, some f-c Gravel, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 29.8965 D_{85}= 16.1658 D_{60}= 1.3314
D_{50}= 0.5497 D_{30}= 0.1777 D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

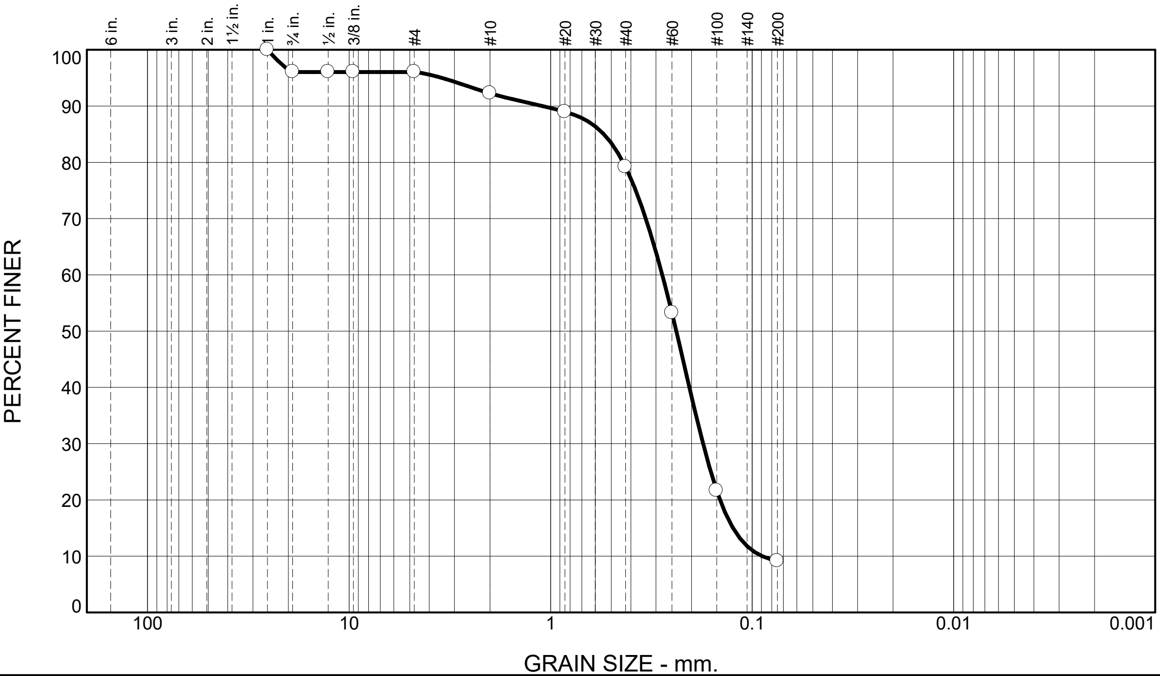
Source of Sample: BB-FJR-103 **Depth:** 106-108' **Date Sampled:**
Sample Number: 23D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-14

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.9	0.0	3.8	13.1	70.1	9.1	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	96.1		
0.5"	96.1		
0.375"	96.1		
#4	96.1		
#10	92.3		
#20	89.0		
#40	79.2		
#60	53.3		
#100	21.6		
#200	9.1		

* (no specification provided)

Material Description

Brown f-m SAND, trace Silt, trace coarse Gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 1.0942 D_{85}= 0.5441 D_{60}= 0.2792
D_{50}= 0.2377 D_{30}= 0.1753 D_{15}= 0.1244
D_{10}= 0.0890 C_u= 3.14 C_c= 1.24

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

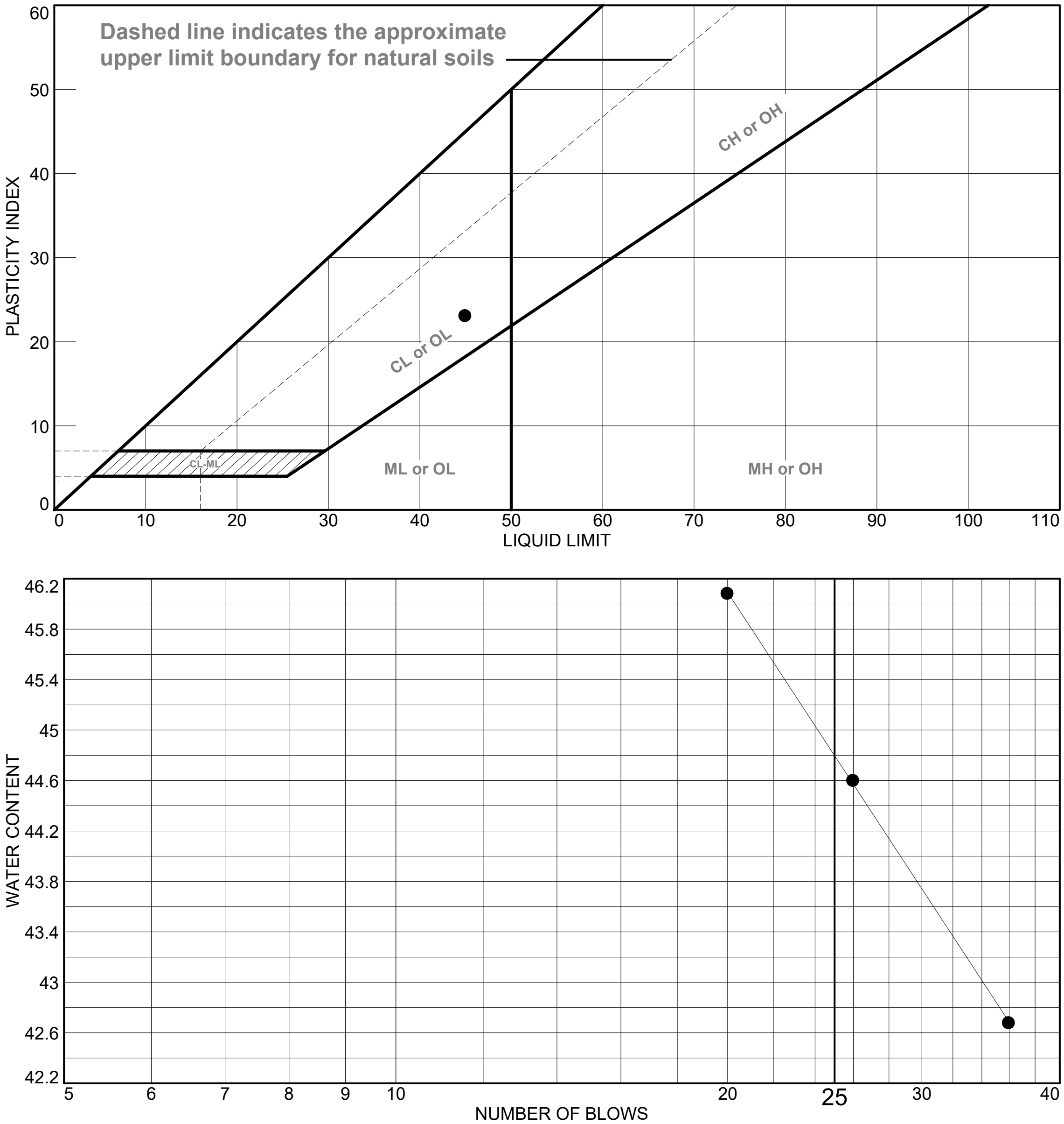
Source of Sample: BB-FJR-105 **Depth:** 2-4' **Date Sampled:**
Sample Number: 1D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-15

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray Silty CLAY	45	22	23			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-105 **Depth:** 22-24'

Sample Number: 5D

Thielsch Engineering Inc.

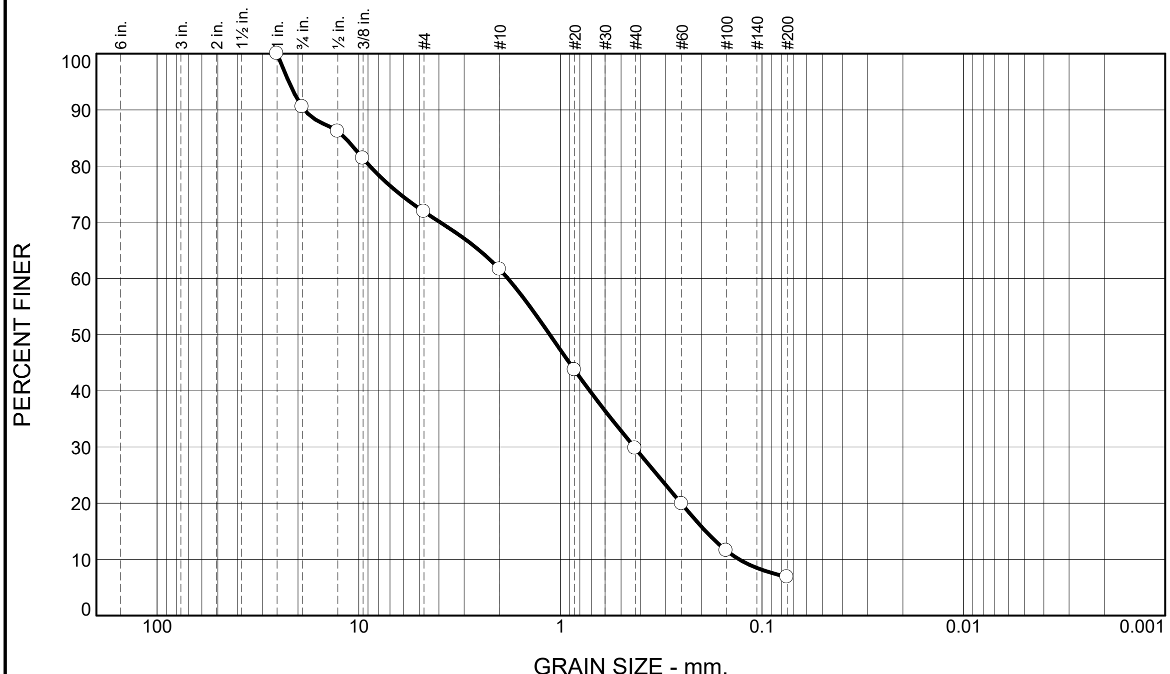
Cranston, RI

Remarks:

Figure L-17

Tested By: IA **Checked By:** RR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.5	18.6	10.3	31.8	23.0	6.8	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	90.5		
0.5"	86.2		
0.375"	81.4		
#4	71.9		
#10	61.6		
#20	43.7		
#40	29.8		
#60	19.9		
#100	11.5		
#200	6.8		

* (no specification provided)

Material Description

Light Brown f-c SAND, some f-c Gravel, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 18.5582 D_{85}= 11.6669 D_{60}= 1.8224
D_{50}= 1.1280 D_{30}= 0.4303 D_{15}= 0.1900
D_{10}= 0.1297 C_u= 14.05 C_c= 0.78

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

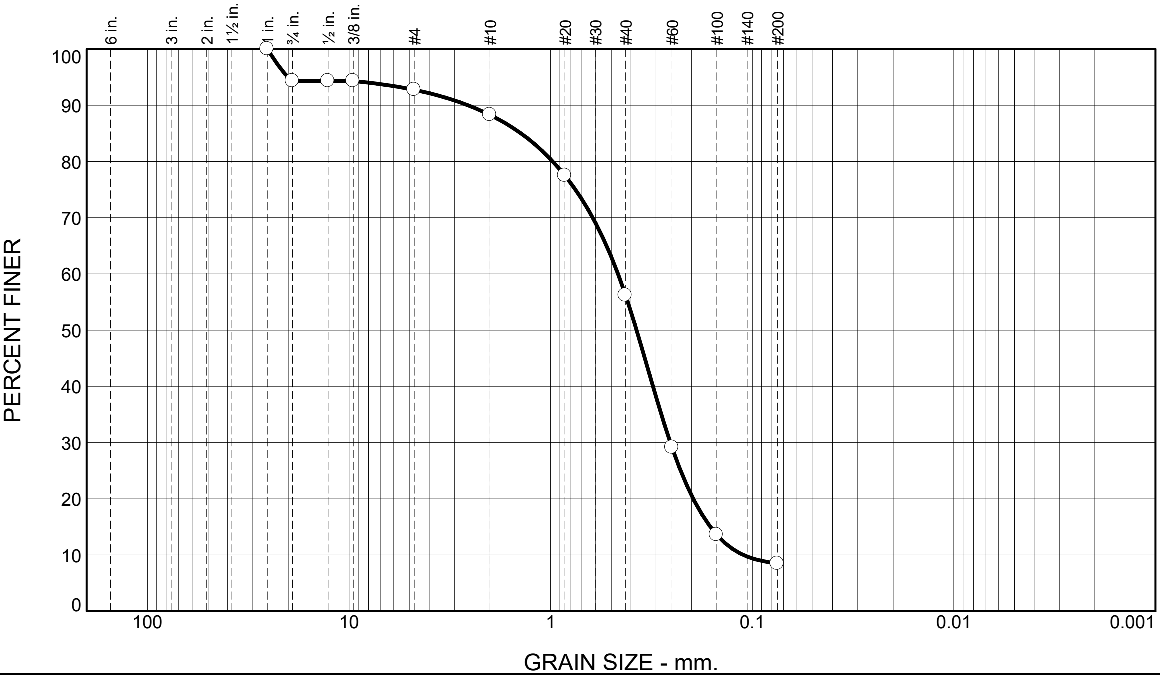
Source of Sample: BB-FJR-106 **Depth:** 1-3' **Date Sampled:**
Sample Number: 1D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-20

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.7	1.6	4.4	32.1	47.7	8.5	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	94.3		
0.5"	94.3		
0.375"	94.3		
#4	92.7		
#10	88.3		
#20	77.5		
#40	56.2		
#60	29.1		
#100	13.6		
#200	8.5		

* (no specification provided)

Material Description

Brown f-m SAND, trace Silt, trace f-c Gravel

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 2.5695 D_{85}= 1.4028 D_{60}= 0.4633
D_{50}= 0.3749 D_{30}= 0.2548 D_{15}= 0.1613
D_{10}= 0.1105 C_u= 4.19 C_c= 1.27

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

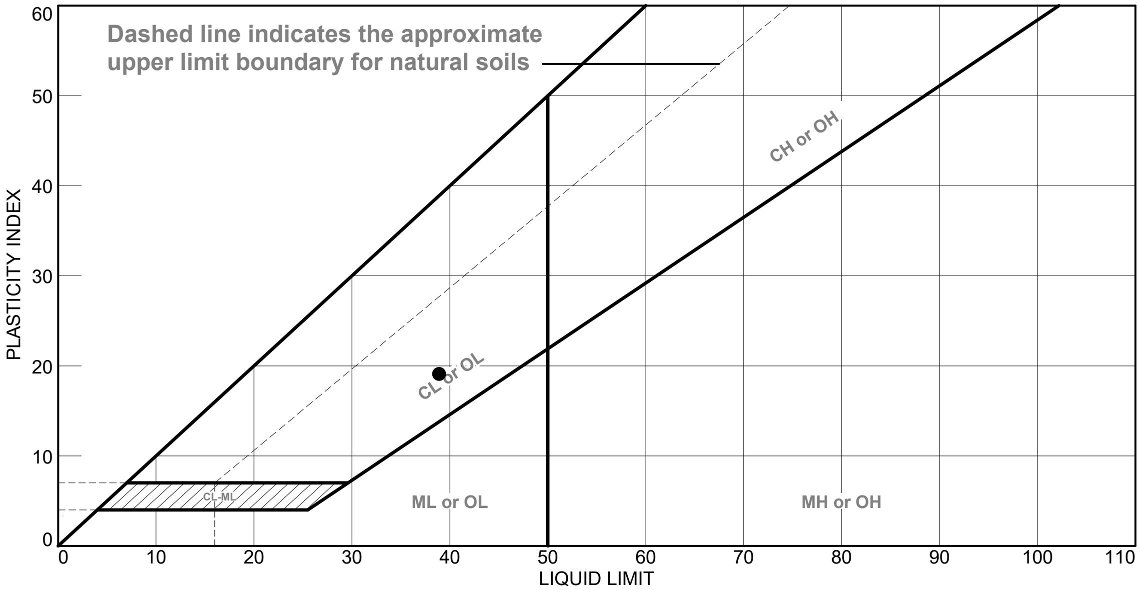
Source of Sample: BB-FJR-106 **Depth:** 5-7' **Date Sampled:**
Sample Number: 2D

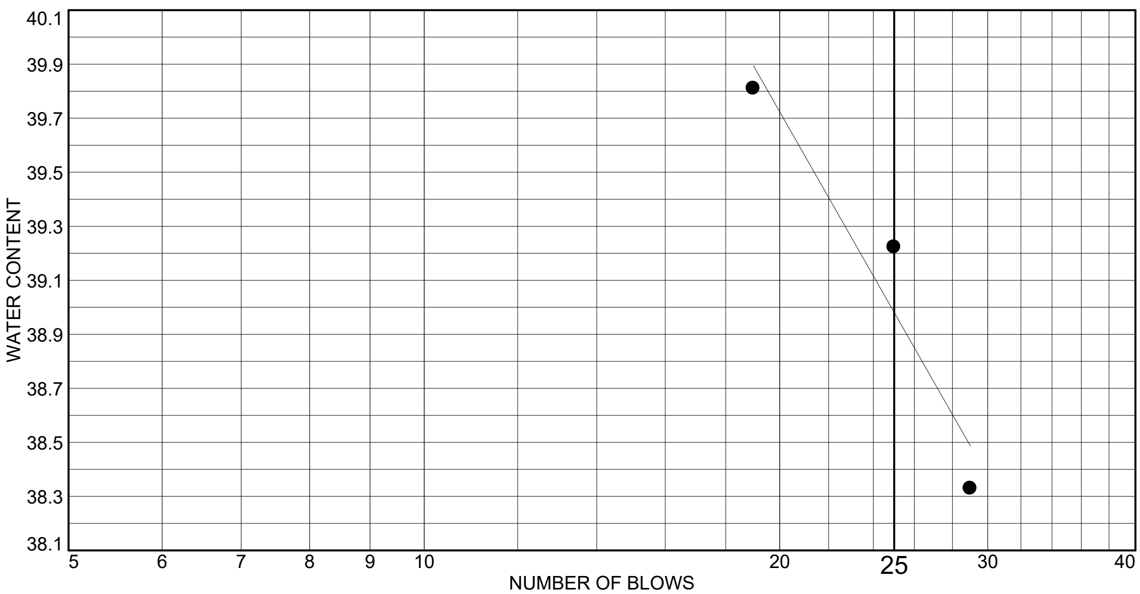
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-21

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray CLAY & SILT	39	20	19			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-106 **Depth:** 30-32'

Sample Number: 9D

Thielsch Engineering Inc.

Cranston, RI

Remarks:

Figure L-23

Tested By: IA **Checked By:** SA

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.5	3.4	15.7	65.9	7.5	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	98.1		
0.375"	94.2		
#4	92.5		
#10	89.1		
#20	82.8		
#40	73.4		
#60	49.4		
#100	19.0		
#200	7.5		

* (no specification provided)

Material Description

Light Brown f-m SAND, trace fine Gravel, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 2.3864 D_{85}= 1.1514 D_{60}= 0.3026
D_{50}= 0.2523 D_{30}= 0.1841 D_{15}= 0.1350
D_{10}= 0.1086 C_u= 2.79 C_c= 1.03

Remarks

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

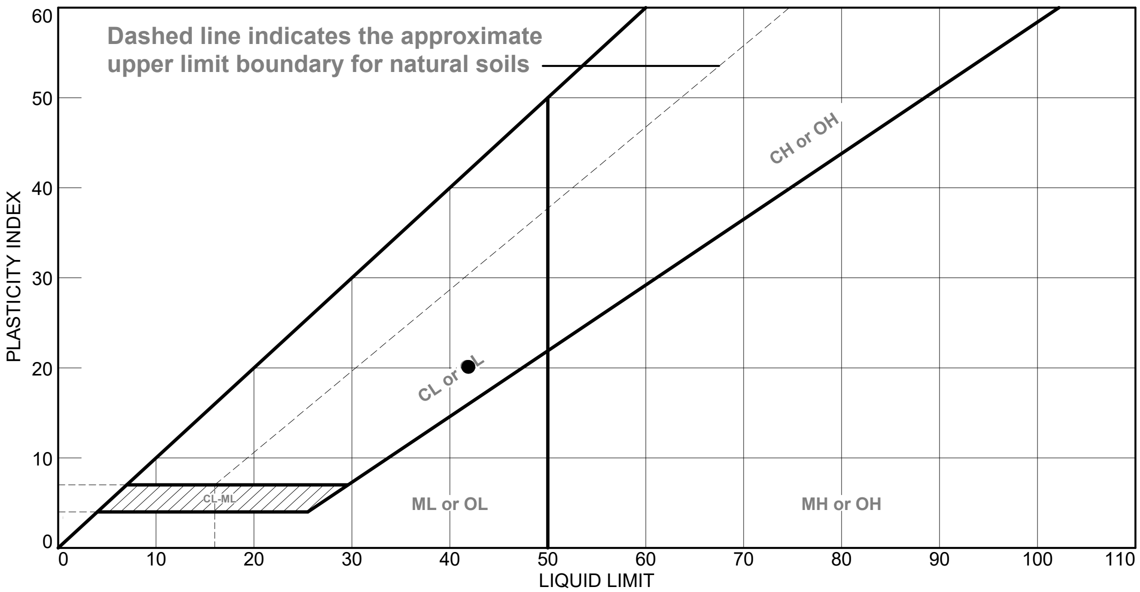
Source of Sample: BB-FJR-104 **Depth:** 4-6' **Date Sampled:**
Sample Number: 2D

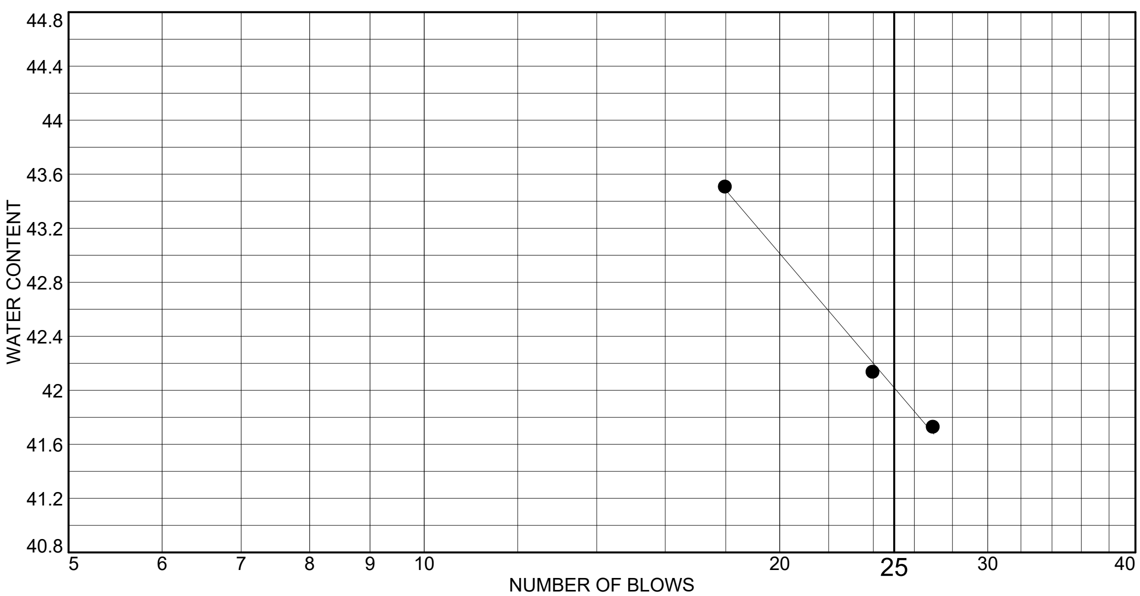
Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-27

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray CLAY & SILT	42	22	20			

Project No. 09.0026024.00 **Client:** GZA GeoEnvironmental

Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00

Falmouth, ME

Source of Sample: BB-FJR-104 **Depth:** 20-22'

Sample Number: 5D

Remarks:

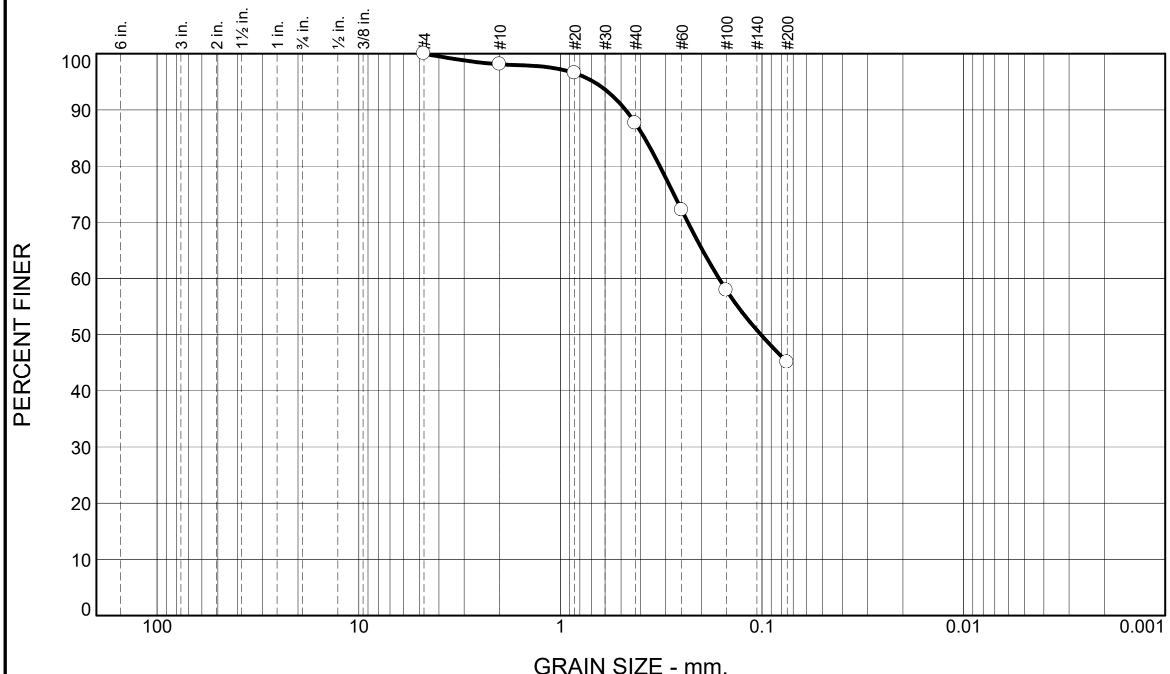
Thielsch Engineering Inc.

Cranston, RI

Figure L-29

Tested By: IA **Checked By:** SA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.9	10.5	42.5	45.1	

Test Results (D6913 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100.0		
#10	98.1		
#20	96.6		
#40	87.6		
#60	72.2		
#100	57.9		
#200	45.1		

* (no specification provided)

Material Description

Gray f-m SAND and Clayey Silt

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4

Coefficients

D_{90}= 0.4750 D_{85}= 0.3823 D_{60}= 0.1636
D_{50}= 0.1013 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/4".

Date Received: 08.26.19 **Date Tested:** 08.28.19

Tested By: LR / IA / JM

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-FJR-104 **Depth:** 60-62' **Date Sampled:**
Sample Number: 10D

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental
Project: Johnson Road Bridge No. 5792, MEDOT WIN 21721.00
Falmouth, ME
Project No: 09.0026024.00 **Figure** S-33

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Transmittal

TO:
Blaine Cardali
GZA GeoEnvironmental, Inc.
477 Congress Street, Suite 700
Portland, ME 04101

DATE: 7/25/2019	GTX NO: 310146
RE: MEDOT Replace Johnson Rd Bridge No. 5792	

COPIES	DATE	DESCRIPTION
	7/25/2019	**July 2019 Laboratory Test Report**

REMARKS:

SIGNED:
Jonathan Campbell, Laboratory Manager

CC:

APPROVED BY :
Joe Tomei, Director of Testing Services

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July 25, 2019

Blaine Cardali
GZA GeoEnvironmental, Inc.
477 Congress Street, Suite 700
Portland, ME 04101

RE: MEDOT Replace Johnson Rd Bridge No. 5792, Falmouth, ME (GTX-310146)

Dear Blaine:

Enclosed are the test results you requested for the above referenced project. GeoTesting Express, Inc. (GTX) received three samples from you on 6/14/2019. These samples were labeled as follows:

Boring Number	Sample Number	Depth
BB-FJR-103	1U	45-47
BB-FJR-103	2U	55-57
BB-FJR-105	2U	28-30

GTX performed the following tests on these samples:

3 ASTM D2216 - Moisture Content
2 ASTM D2435 - Incremental Consolidation
1 ASTM D4186 - Constant-Rate-of-Strain (CRS) Consolidation
3 ASTM D4318 - Atterberg Limits
1 ASTM D6528 - Direct Simple Shear

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,

Jonathan Campbell
Laboratory Manager

GeoTesting EXPRESS
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Geotechnical Test Report **7/25/2019**

GTX-310146
MEDOT Replace Johnson Rd Bridge No. 5792

Falmouth, ME

Client Project No.: 09.0026024.00

Prepared for:

GZA GeoEnvironmental, Inc.

GeoTesting EXPRESS

Client:	GZA GeoEnvironmental, Inc.		
Project:	MEDOT Replace Johnson Rd Bridge No. 5792		
Location:	Falmouth, ME	Project No:	GTX-310146
Boring ID: ---	Sample Type: ---	Tested By:	md
Sample ID: ---	Test Date: 07/09/19	Checked By:	bfs
Depth : ---	Test Id: 510327		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FJR-103	1U	45-47	Moist, dark gray clay	36.2
BB-FJR-103	2U	55-57	Moist, dark gray clay	31.0
BB-FJR-105	2U	28-30	Moist, dark gray clay	44.5

Notes: Temperature of Drying : 110º Celsius

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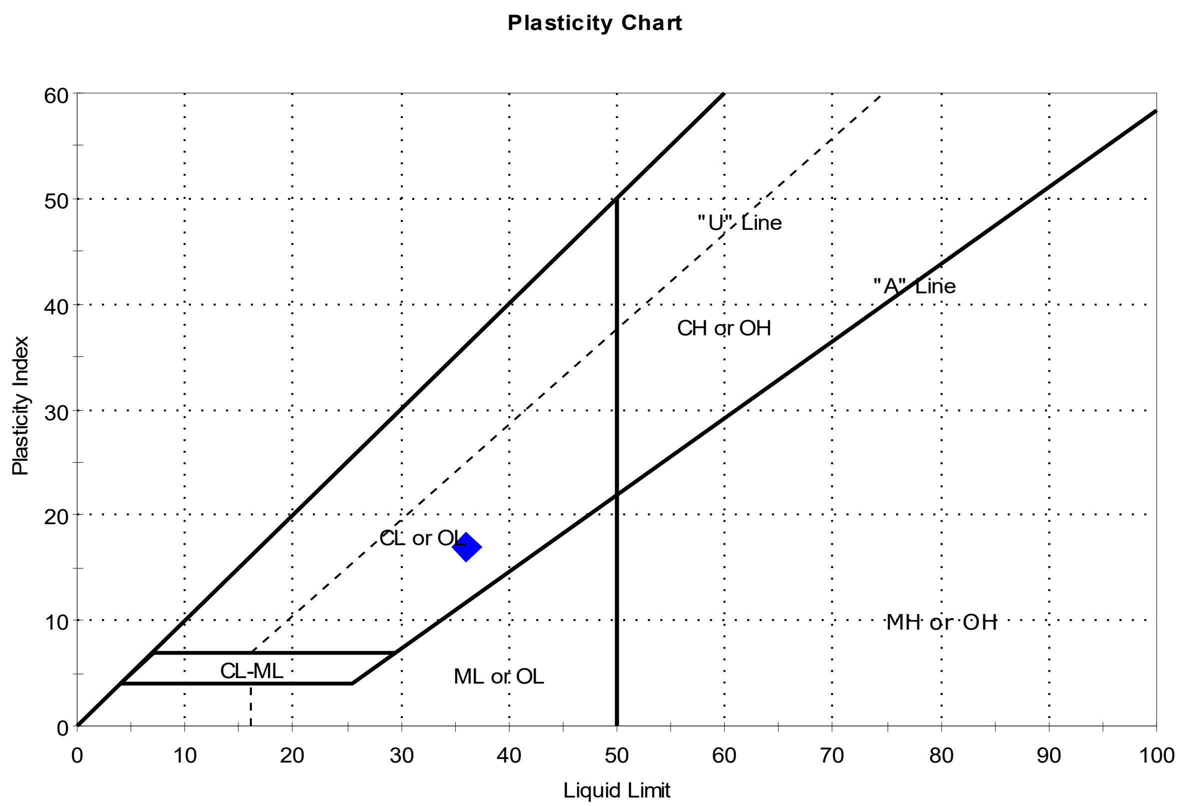
Client:	GZA GeoEnvironmental, Inc.				
Project:	MEDOT Replace Johnson Rd Bridge No. 5792				
Location:	Falmouth, ME			Project No:	GTX-310146
Boring ID:	BB-FJR-103	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	07/09/19	Checked By:	bfs
Depth :	45-47	Test Id:	510325		
Test Comment:	---				
Visual Description:	Moist, dark 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
◆	1U	B-FJR-10	45-47	36	36	19	17	1	

Sample Prepared using the WET method

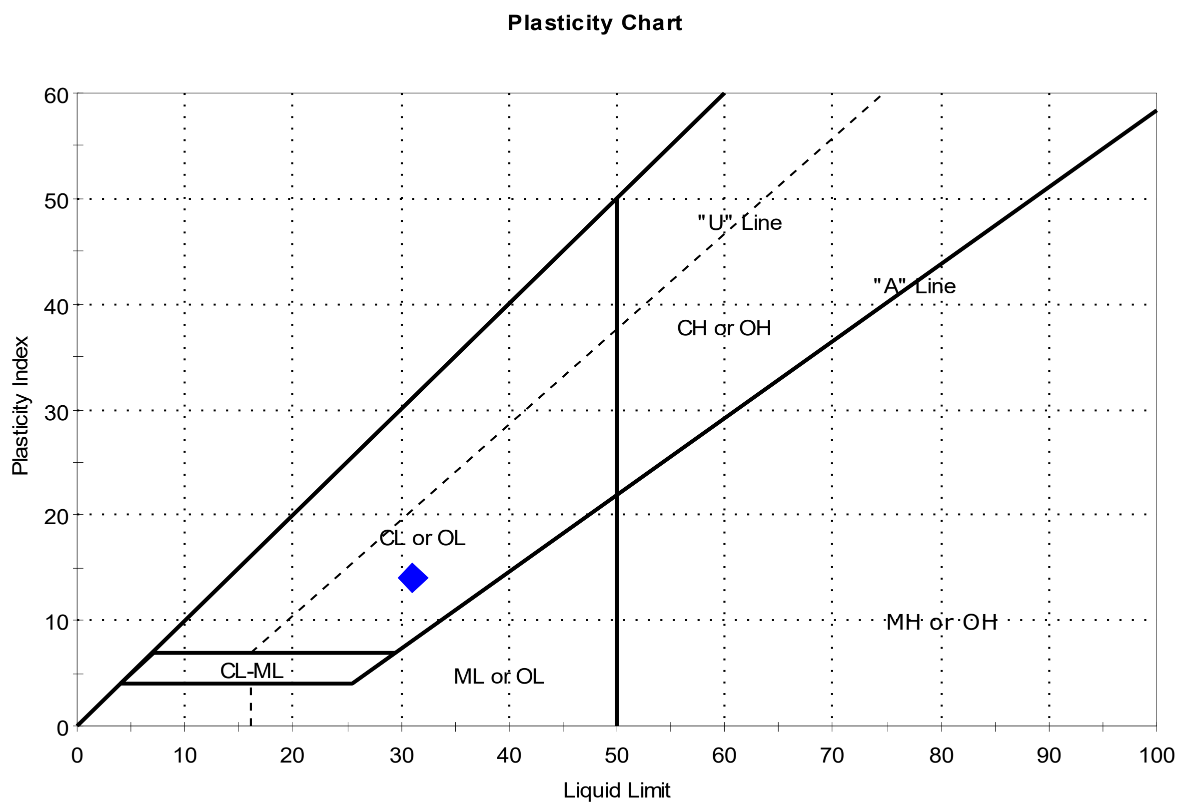
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	GZA GeoEnvironmental, Inc.		
Project:	MEDOT Replace Johnson Rd Bridge No. 5792		
Location:	Falmouth, ME	Project No:	GTX-310146
Boring ID: BB-FJR-103	Sample Type: tube	Tested By:	cam
Sample ID: 2U	Test Date: 07/09/19	Checked By:	bfs
Depth : 55-57	Test Id: 510326		
Test Comment:	---		
Visual Description:	Moist, dark 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
◆	2U	B-FJR-10	55-57	31	31	17	14	1	

Sample Prepared using the WET method

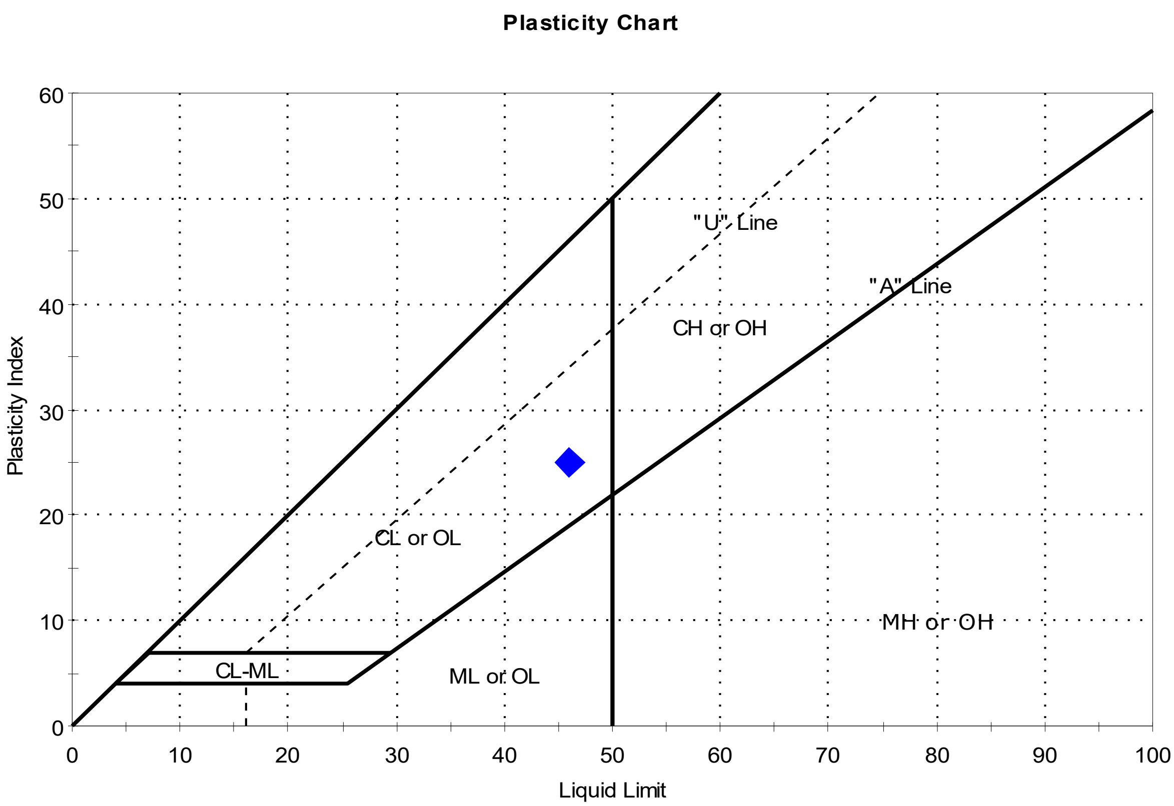
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	GZA GeoEnvironmental, Inc.		
Project:	MEDOT Replace Johnson Rd Bridge No. 5792		
Location:	Falmouth, ME	Project No:	GTX-310146
Boring ID: BB-FJR-105	Sample Type: tube	Tested By:	cam
Sample ID: 2U	Test Date: 07/09/19	Checked By:	bfs
Depth : 28-30	Test Id: 510324		
Test Comment:	---		
Visual Description:	Moist, dark 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
◆	2U	B-FJR-10	28-30	45	46	21	25	0.9	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

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

Client:	GZA GeoEnvironmental, Inc.	GTX#:	310146
Project Name:	Medot Replace Johnson Rd Bridge No. 5792	Test Date:	7/16/19
Project Location:	Falmouth, ME		

Boring ID: BBF-FJR-103
Sample ID: 1U
Depth, ft: 45-47

Visual Description: Moist, dark gray clay

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

Test Condition: Inundated prior to consolidation

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

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	38.4				
Initial Dry Density, pcf	85.1				
Nominal Rate of Shear Strain, %/hr	5.0				
Vertical Consolidation Stress, psf	4,500				
Final Moisture Content, %	31.2				
Measured Peak Shear Stress, psf	970				
Shear Strain at Peak Shear Stress, %	4.6				
Membrane Correction, psf	36				
Corrected Peak Shear Stress, psf	934				
S_u / σ'_{vc}	0.21				

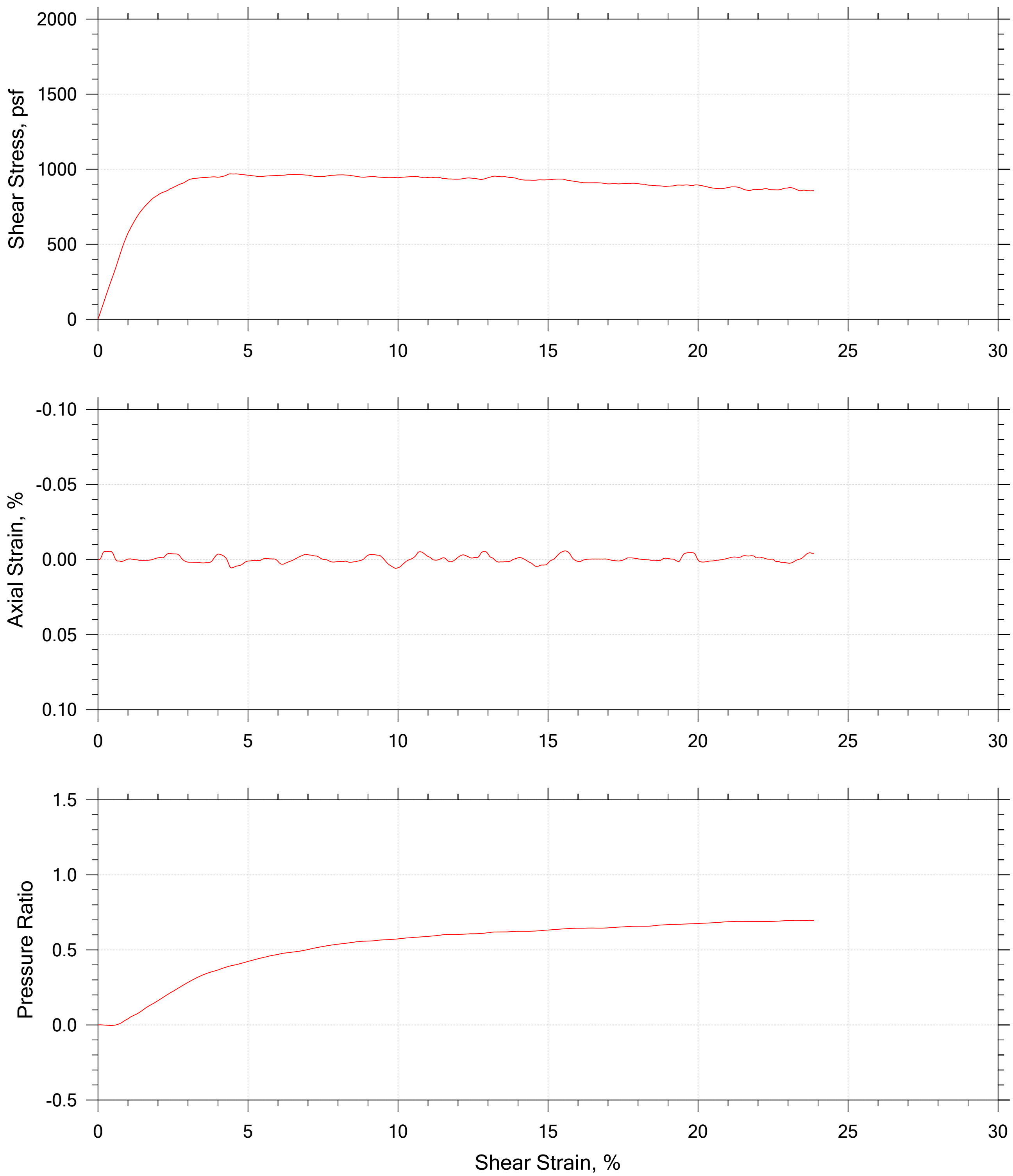
Comments:

Tested By: md

Checked By: njh

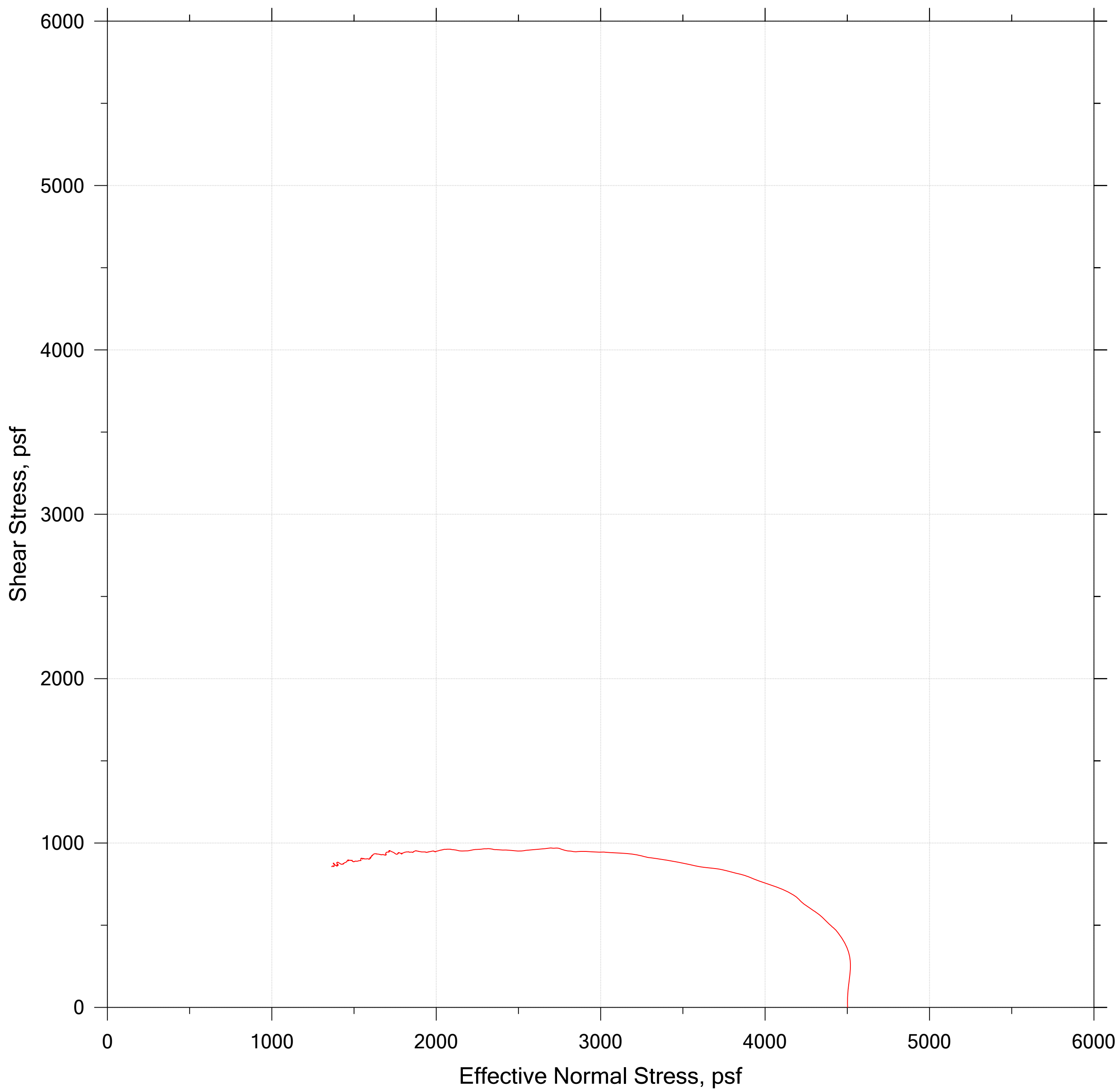
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

Direct Simple Shear Test



Project Name: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project Number: GTX-310146
Boring Number: BB-FJR-103	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 07/16/19	Depth: 45-47 ft
Test Number: DSS-1	Preparation: intact	Elevation: ---
Description: Moist , dark gray clay		
Remarks: System HH		

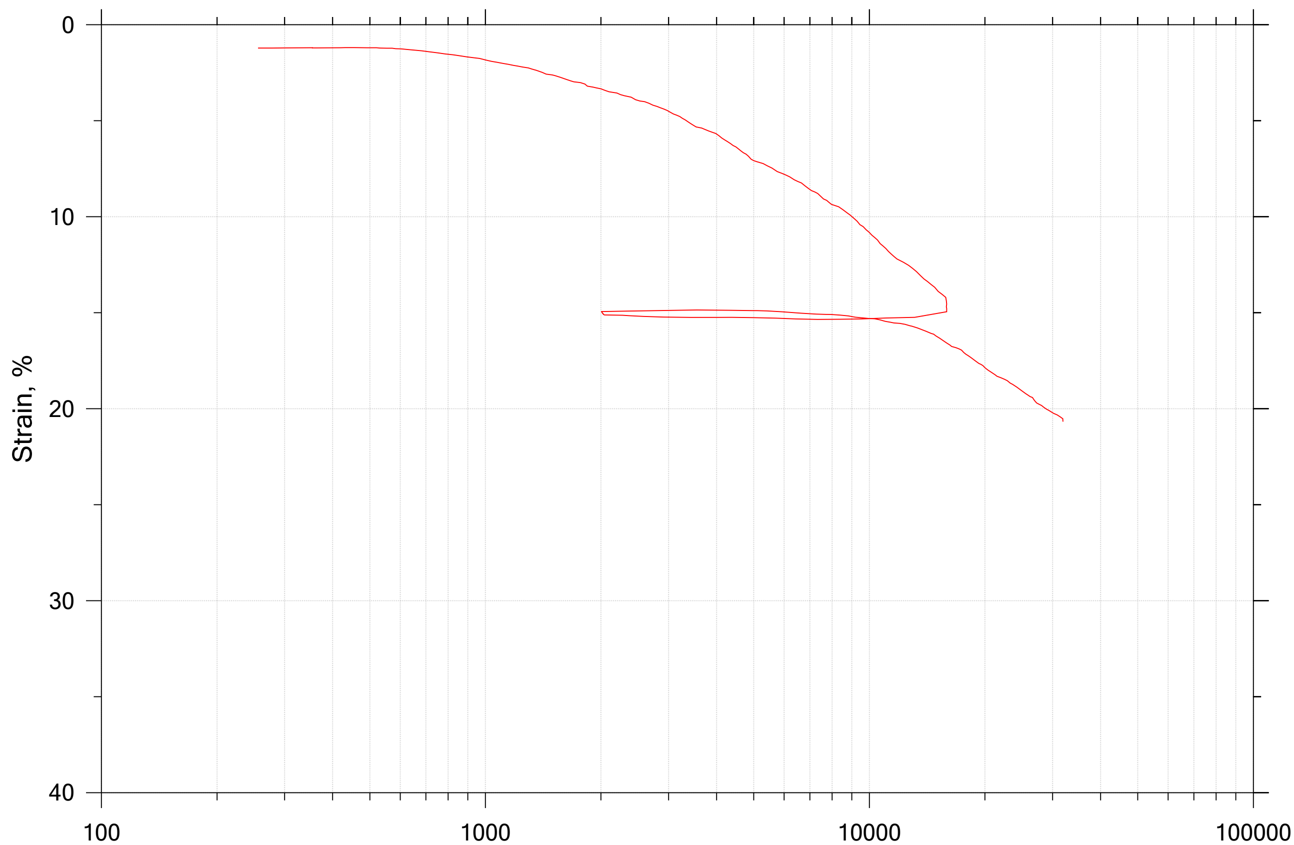
Direct Simple Shear Test

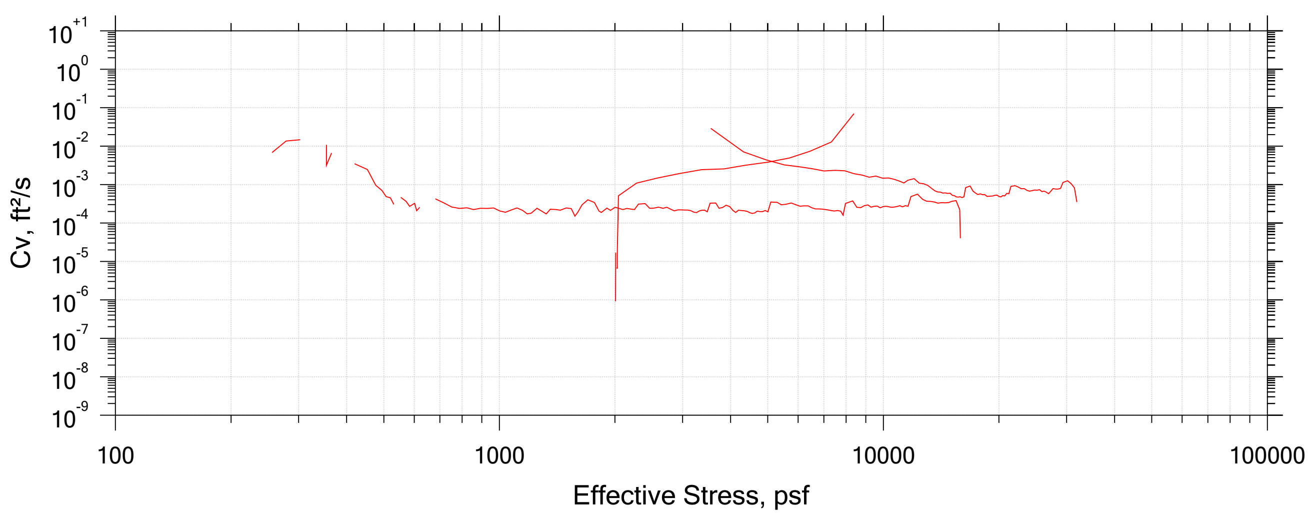


GeoTesting EXPRESS	Project Name: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project Number: GTX-310146
	Boring Number: BB-FJR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 07/16/19	Depth: 45-47 ft
	Test Number: DSS-1	Preparation: intact	Elevation: ---
	Description: Moist , dark gray clay		
	Remarks: System HH		

CRC Test

Summary

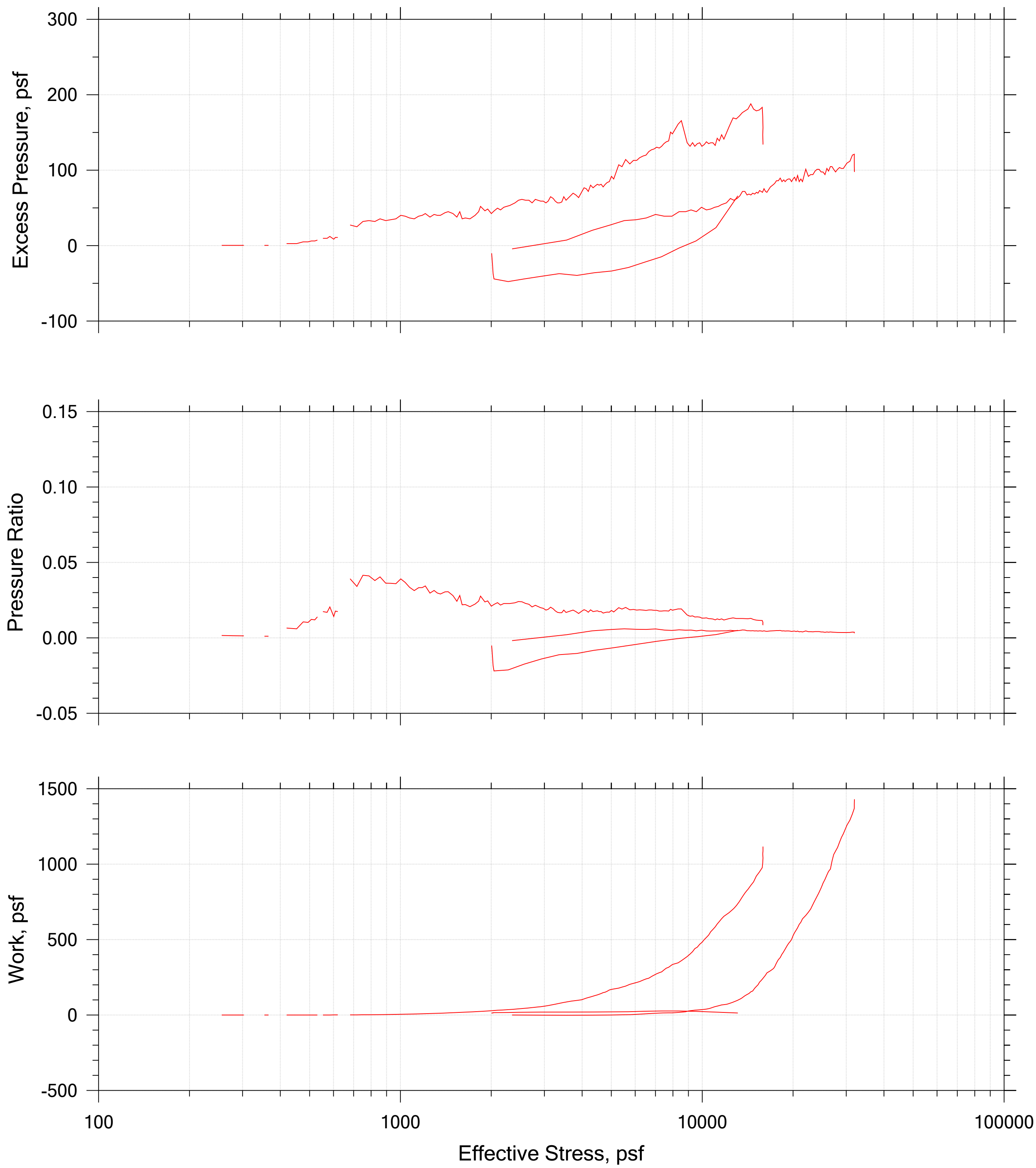




GeoTesting EXPRESS	Project Name: MEDOT Replace Johnson	Location: Falmouth, ME	Project Number: GTX-310146
	Boring Number: BB-FJR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 06/28/19	Depth: 45-47 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		
			Page 1 of 3

CRC Test

Pressure Curves



GeoTesting EXPRESS			
	Project Name: MEDOT Replace Johnson	Location: Falmouth, ME	Project Number: GTX-310146
	Boring Number: BB-FJR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 06/28/19	Depth: 45-47 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.73 (Estimated)	Liquid Limit: 36
Specimen Height, in: 1.00	Initial Void Ratio: 0.874	Plastic Limit: 19
Final Height, in: 0.81	Final Void Ratio: 0.518	Plasticity Index: 17

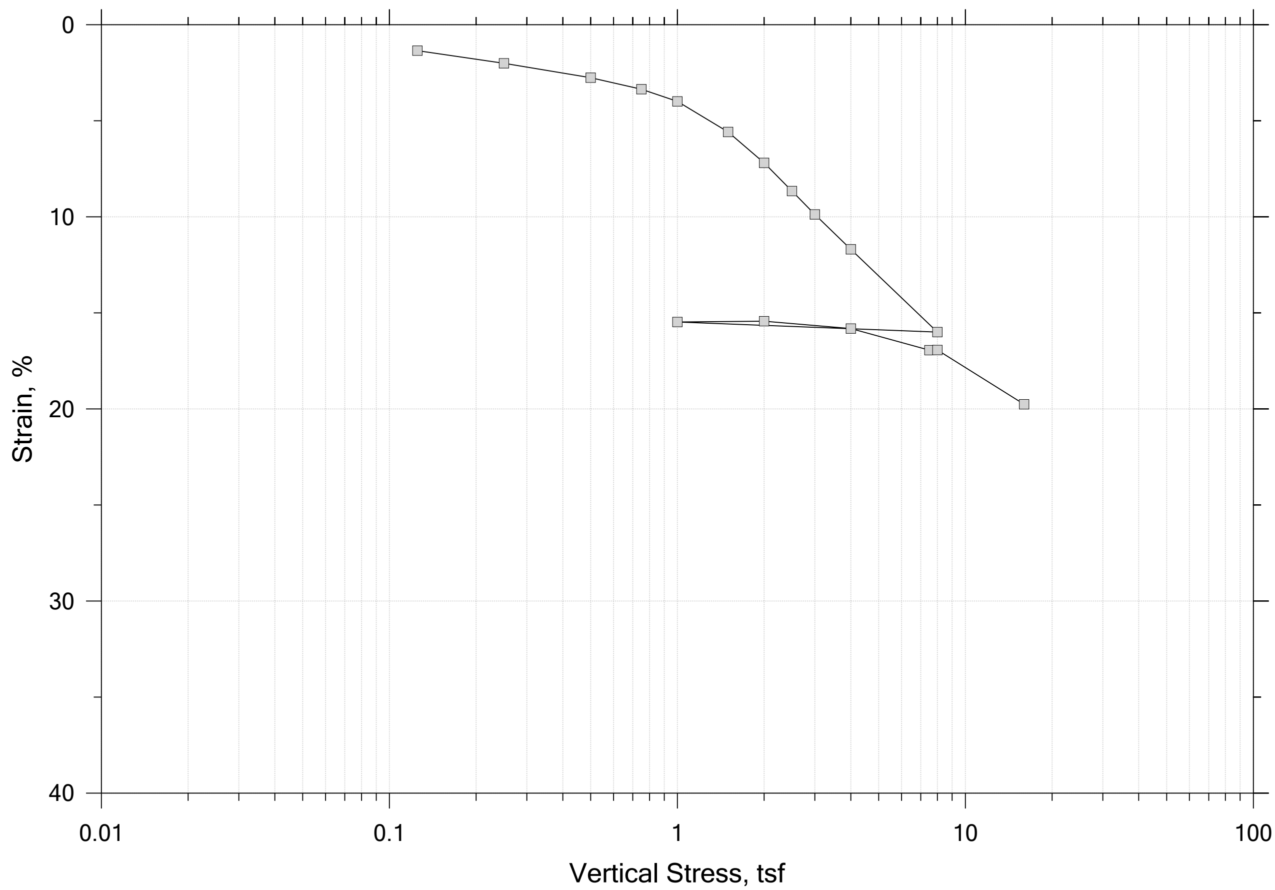
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	D-2206	---		A-2278
Mass Container, gm	8.44	110.86	110.86	8.17
Mass Container + Wet Soil, gm	131.04	264.85	250.18	149.39
Mass Container + Dry Soil, gm	99.2	227.94	227.94	126.85
Mass Dry Soil, gm	90.76	117.08	117.08	118.68
Water Content, %	35.08	31.52	18.99	18.99
Void Ratio	---	0.87	0.52	---
Degree of Saturation, %	---	98.36	100.00	---
Dry Unit Weight, pcf	---	90.866	112.18	---

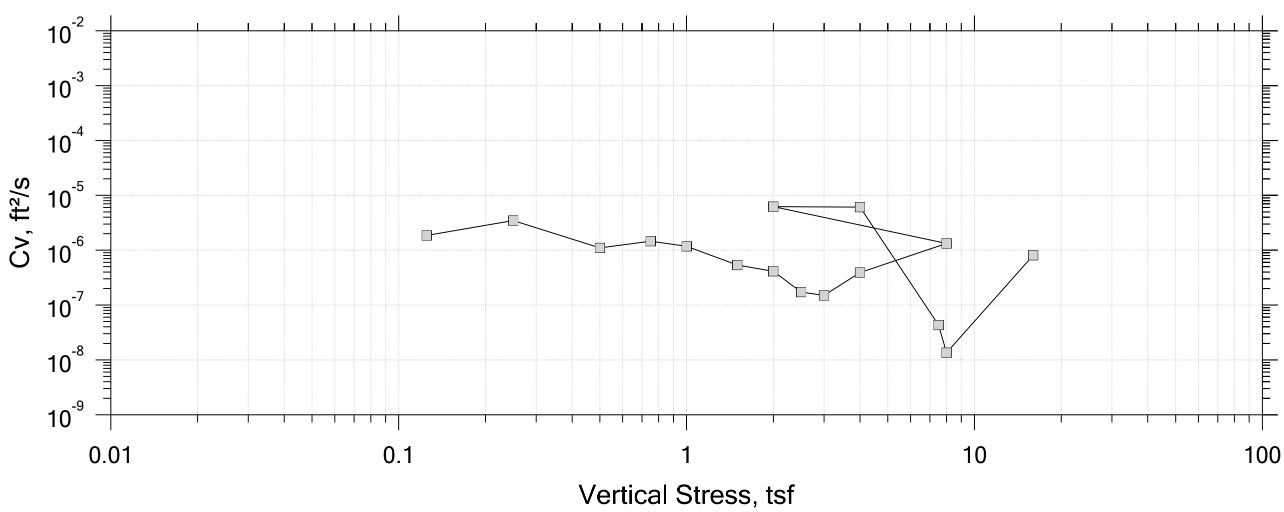
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test.
Therefore, values may not represent actual values for the specimen.

Project Name: MEDOT Replace Johnson	Location: Falmouth, ME	Project Number: GTX-310146
Boring Number: BB-FJR-103	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 06/28/19	Depth: 45-47 ft
Test Number: CRC-1	Preparation: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report





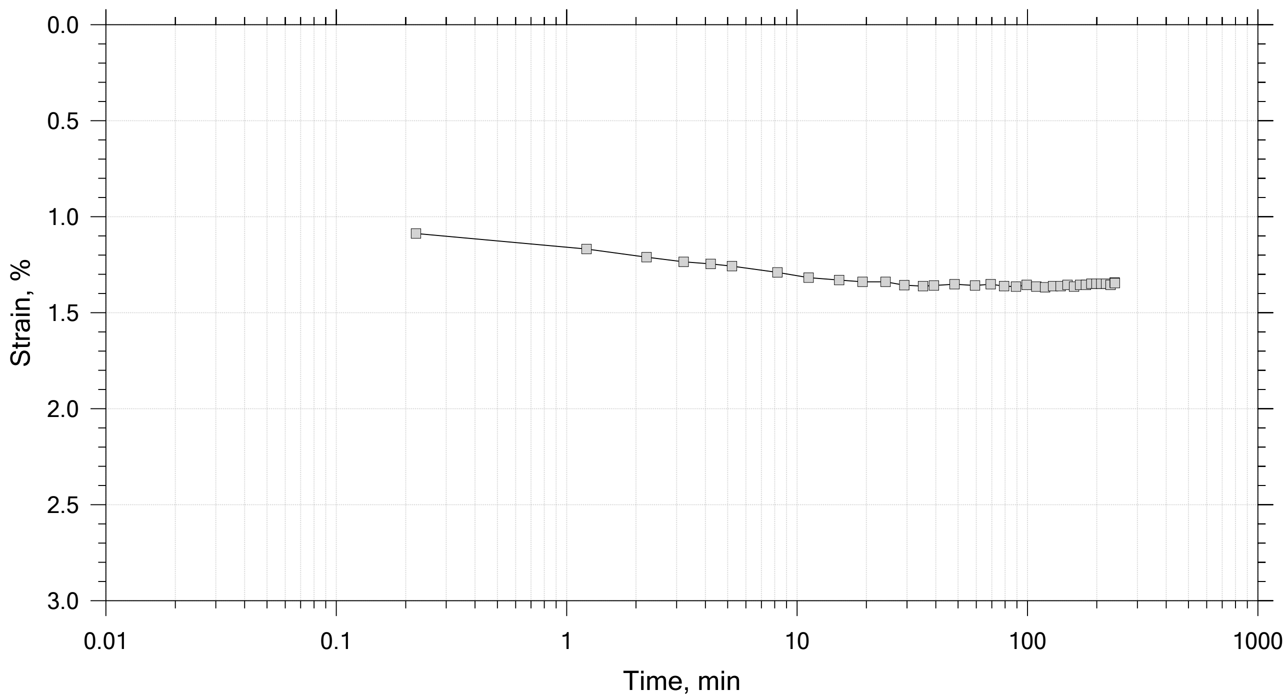
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		
	Displacement at End of Increment		

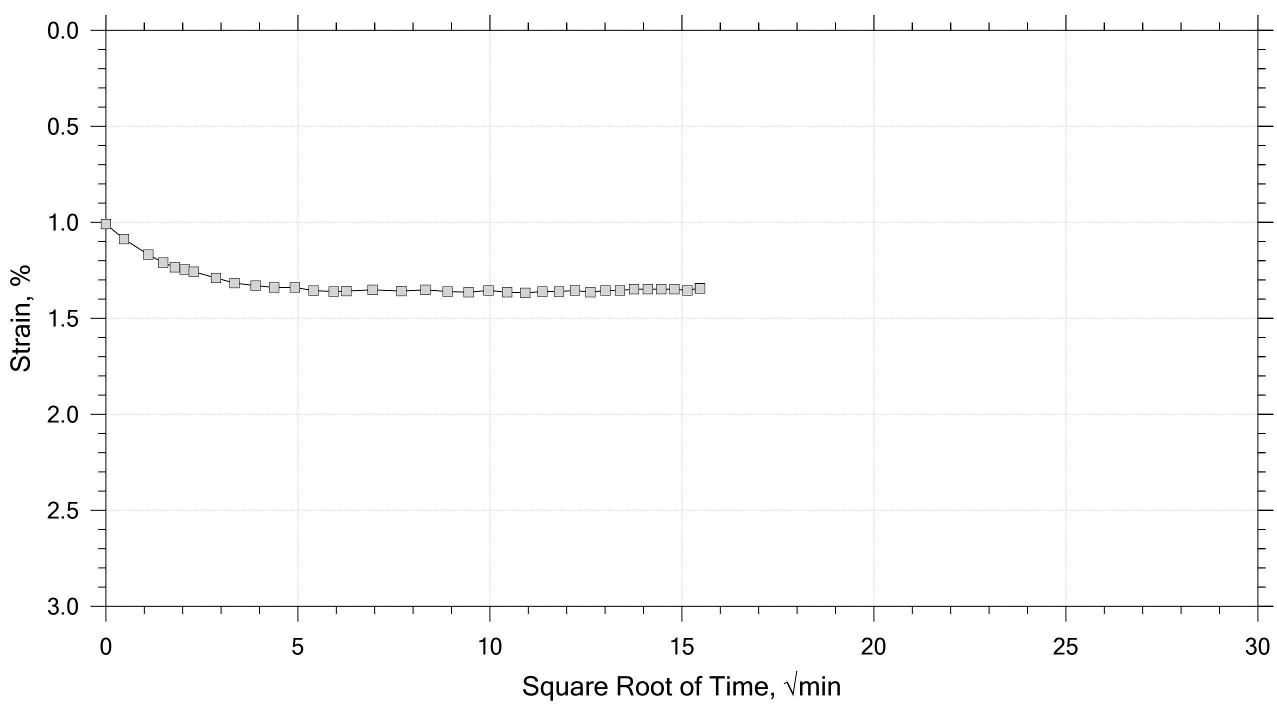
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 17

Constant Load Step

Stress: 0.125 tsf





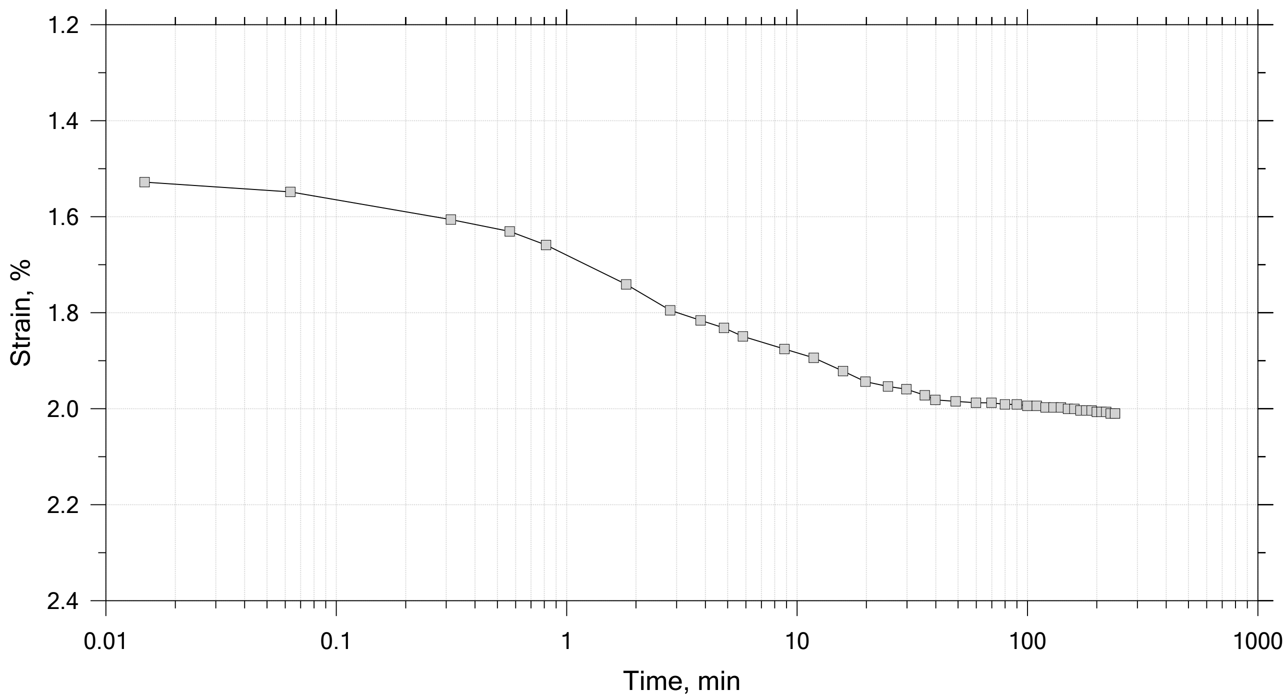
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

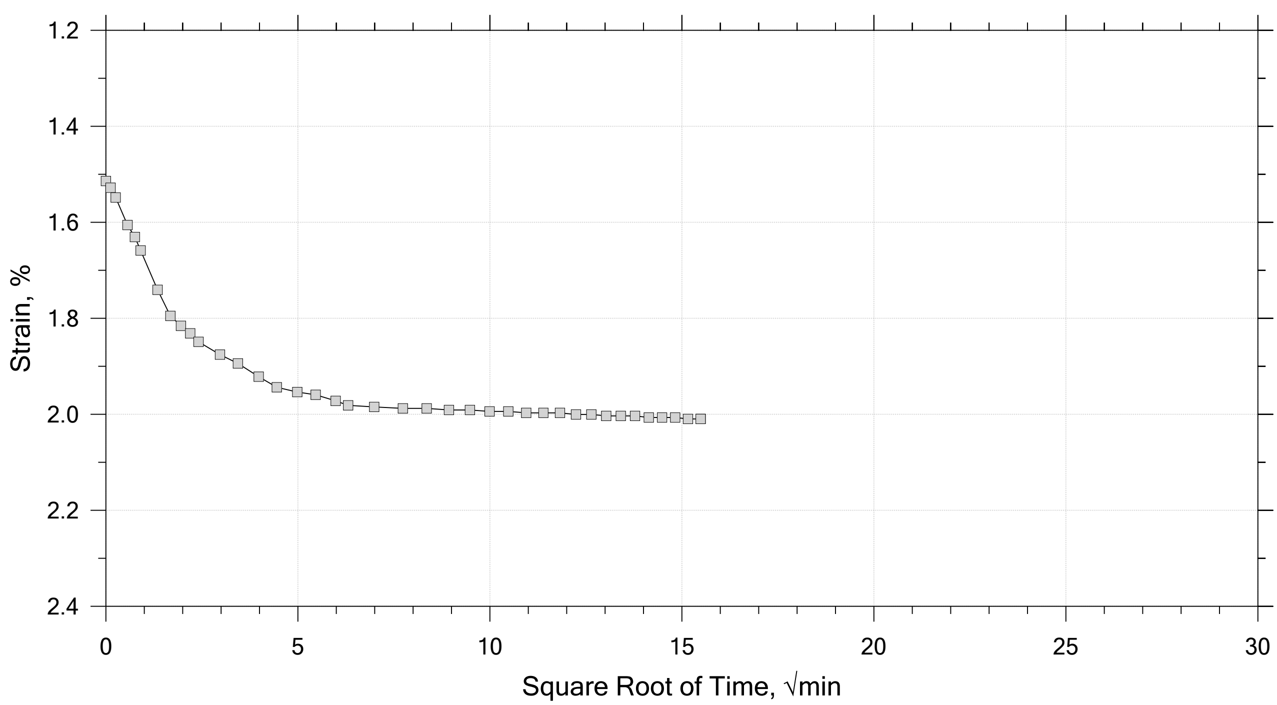
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 17

Constant Load Step

Stress: 0.25 tsf





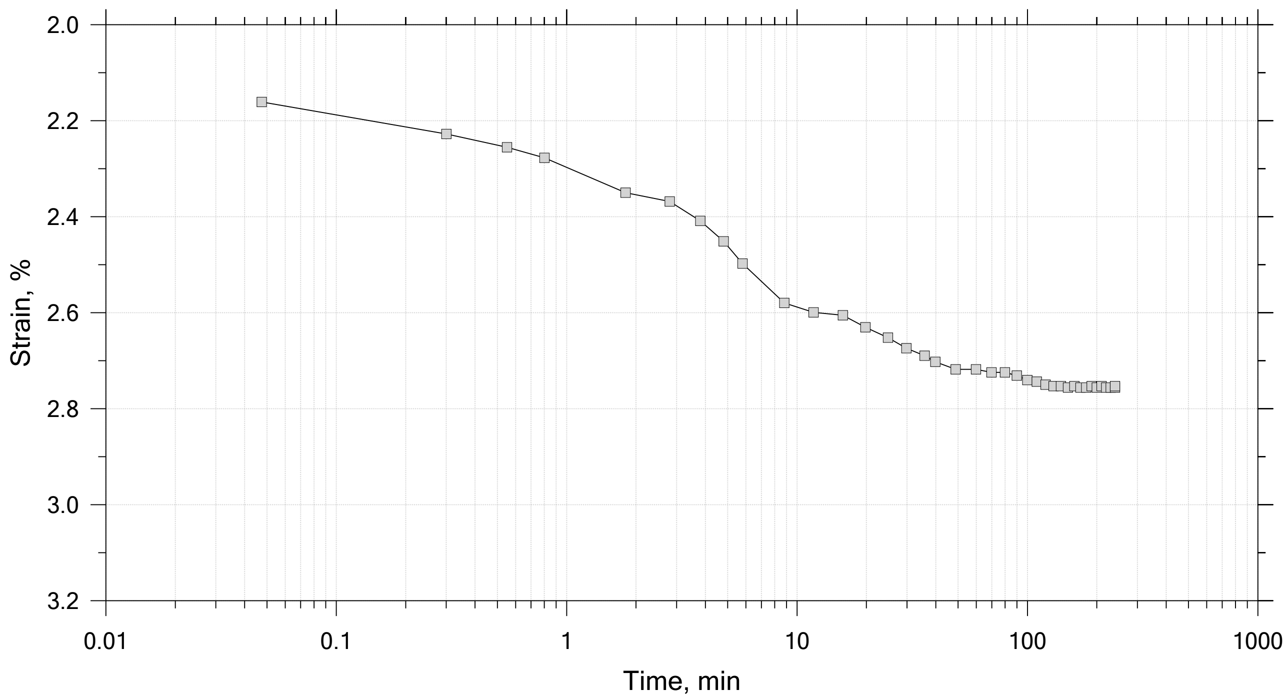
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
GeoTesting EXPRESS	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

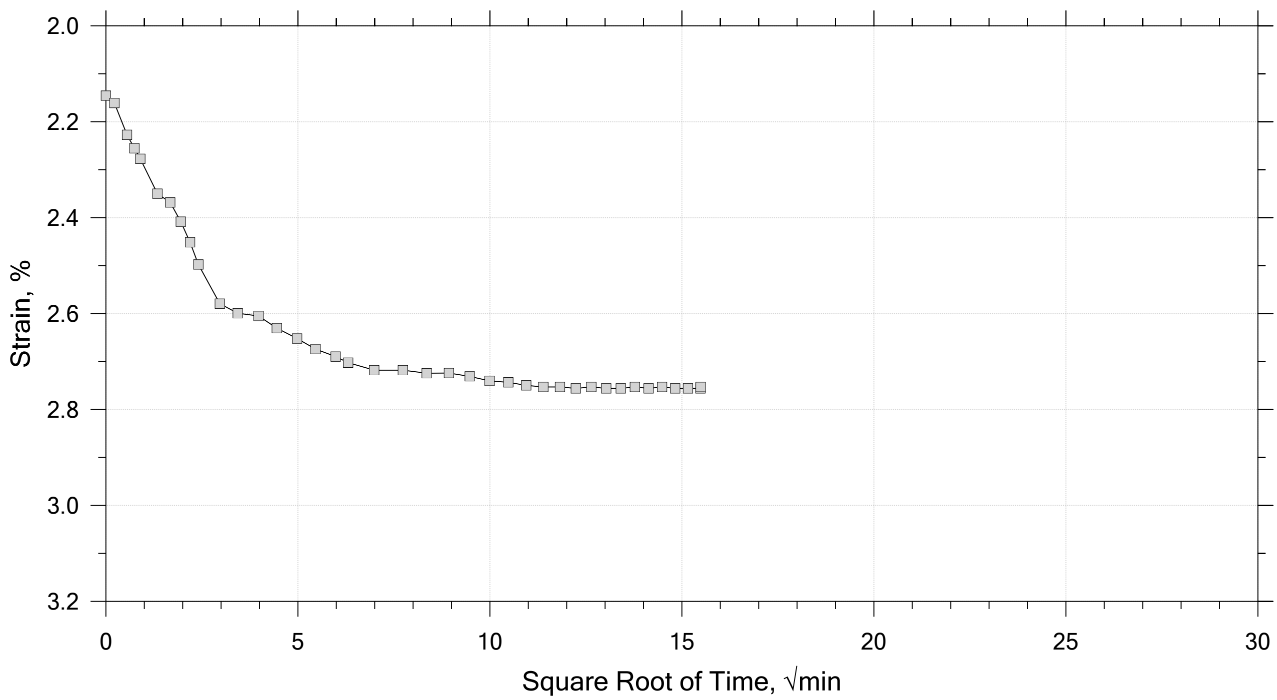
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 17

Constant Load Step

Stress: 0.5 tsf





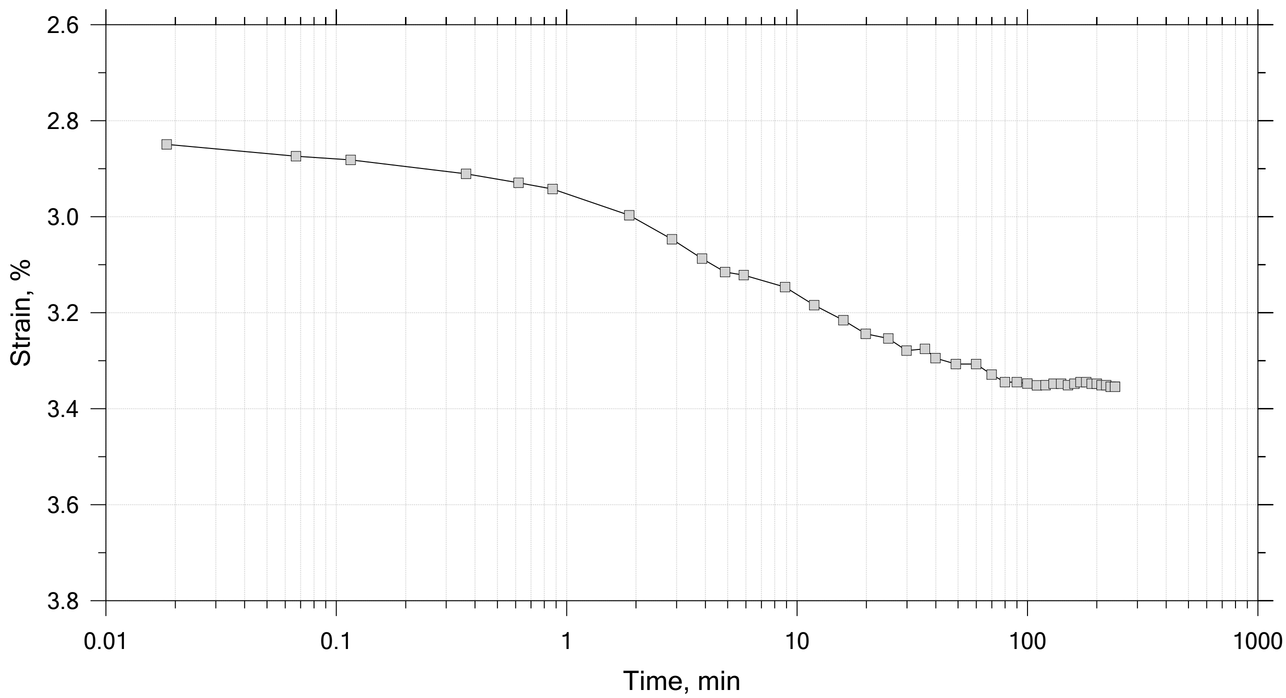
GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

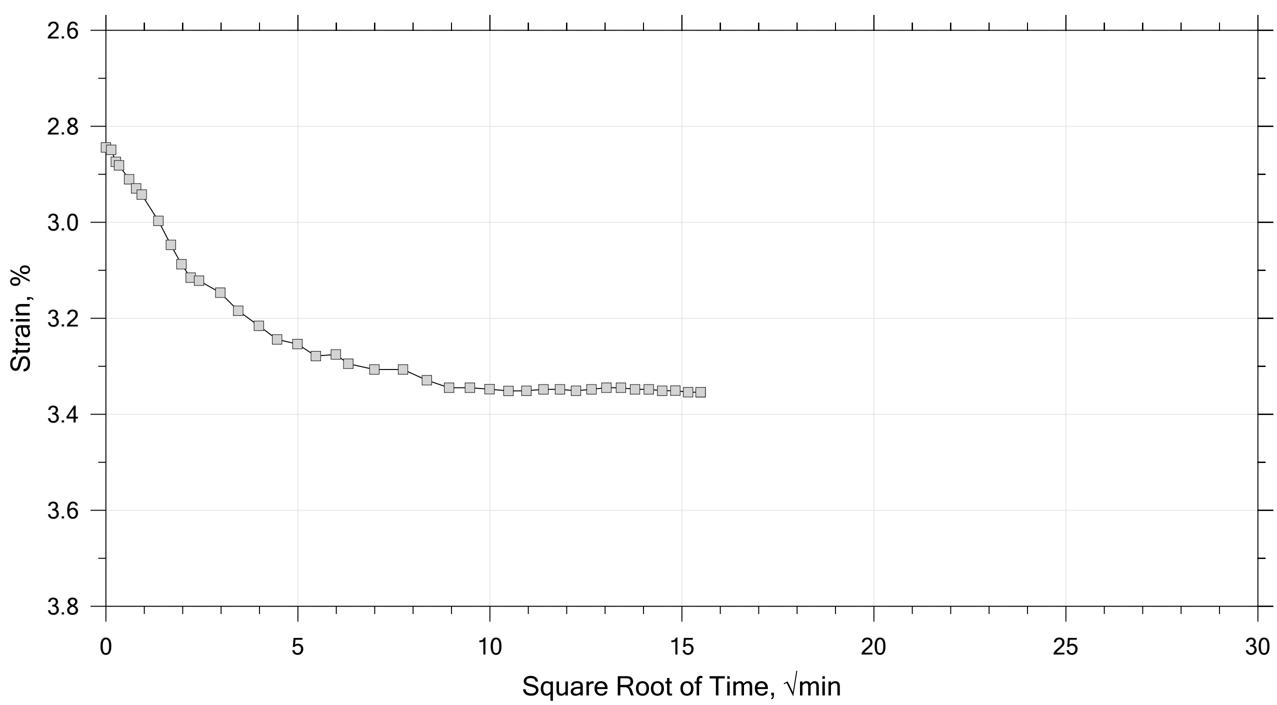
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 17

Constant Load Step

Stress: 0.75 tsf





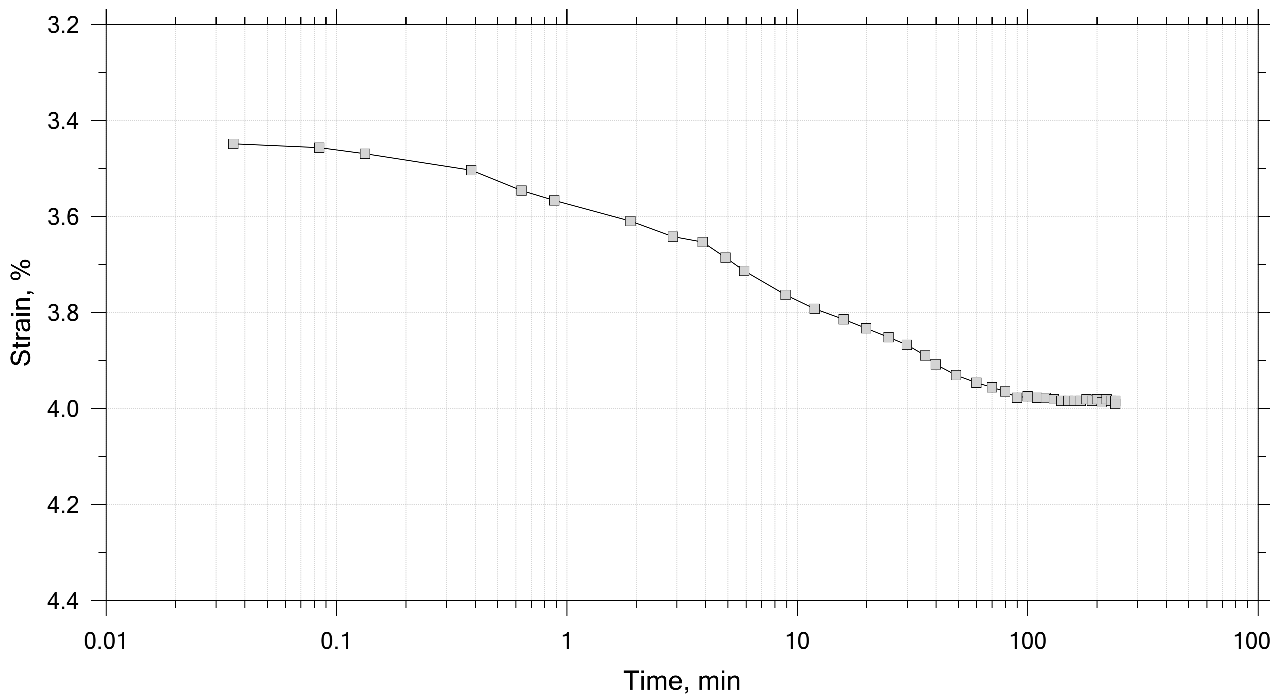
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

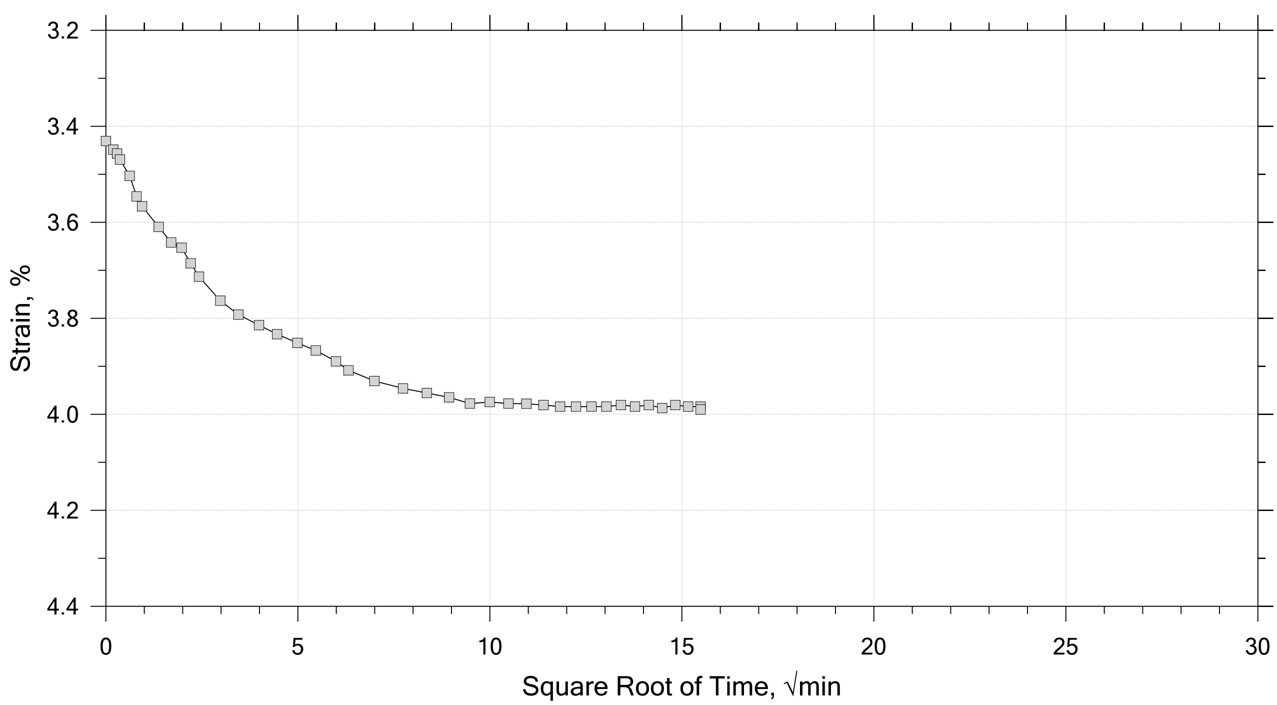
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 17

Constant Load Step

Stress: 1 tsf





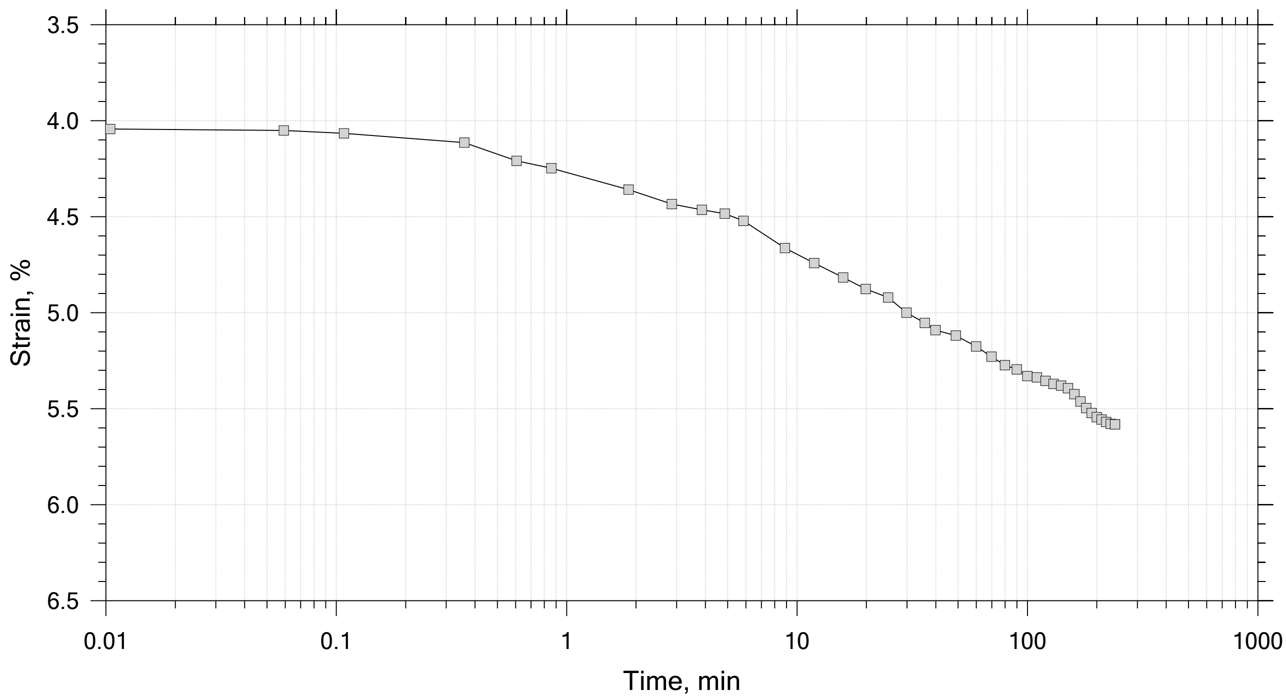
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

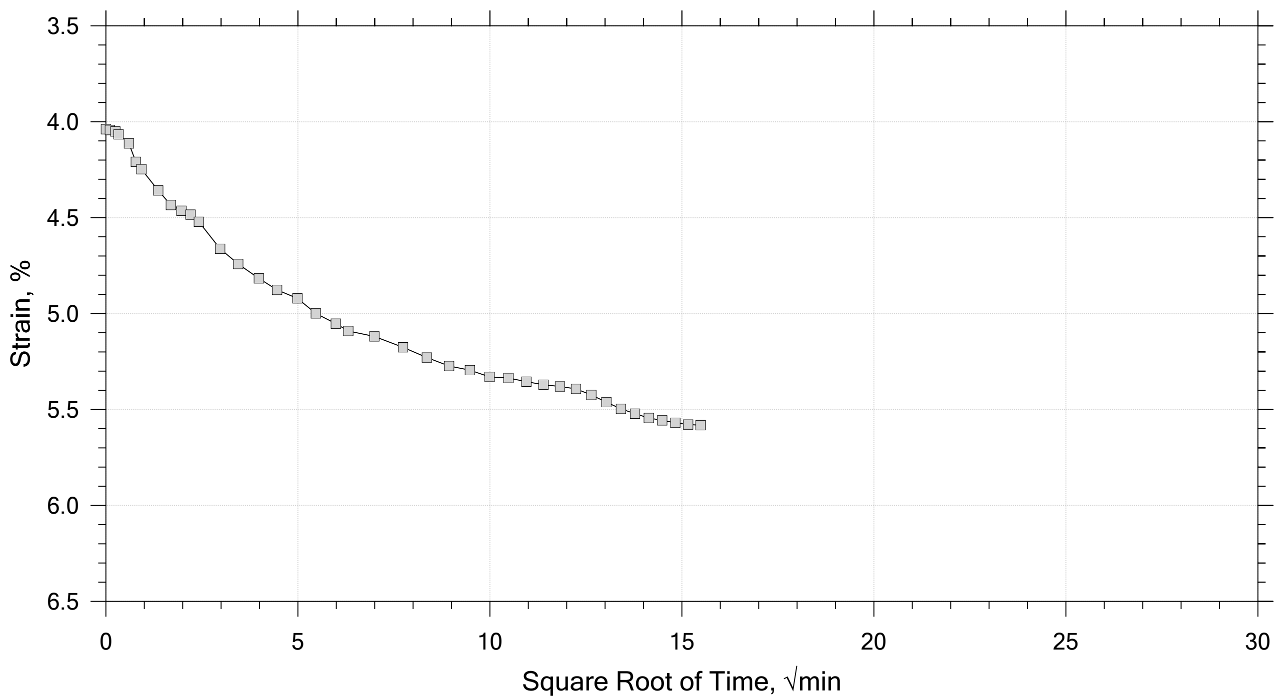
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 17

Constant Load Step

Stress: 1.5 tsf





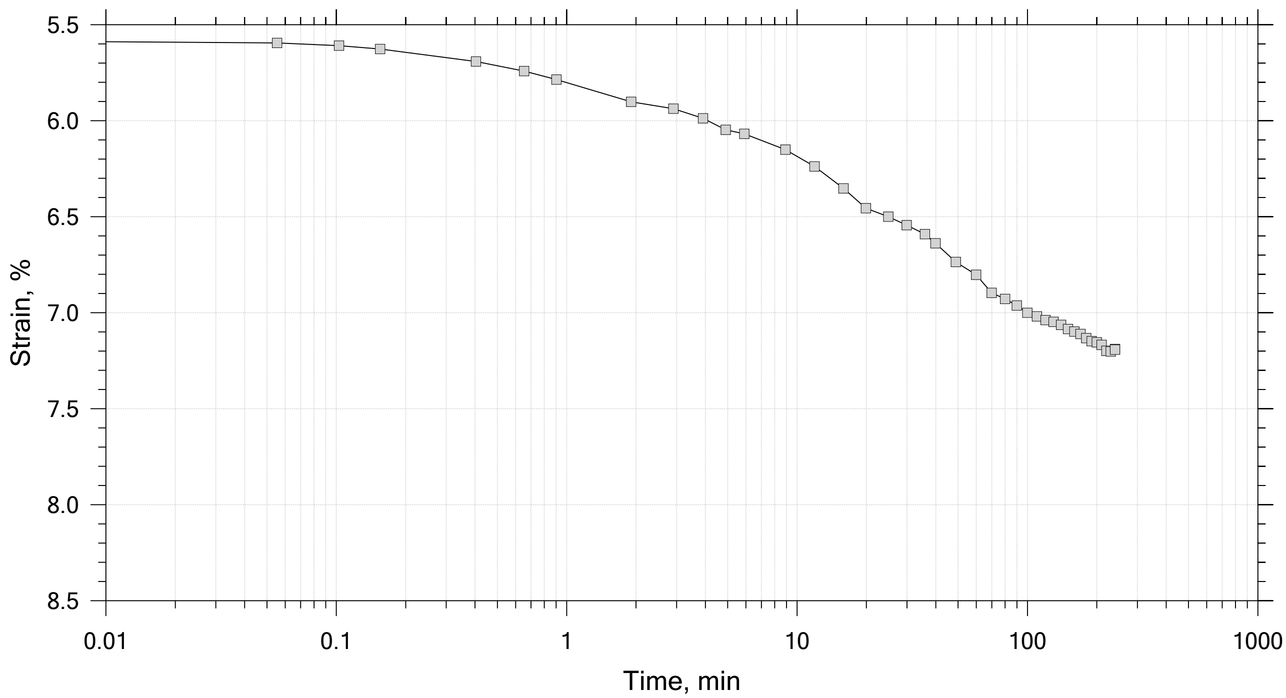
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System LTIII-A		

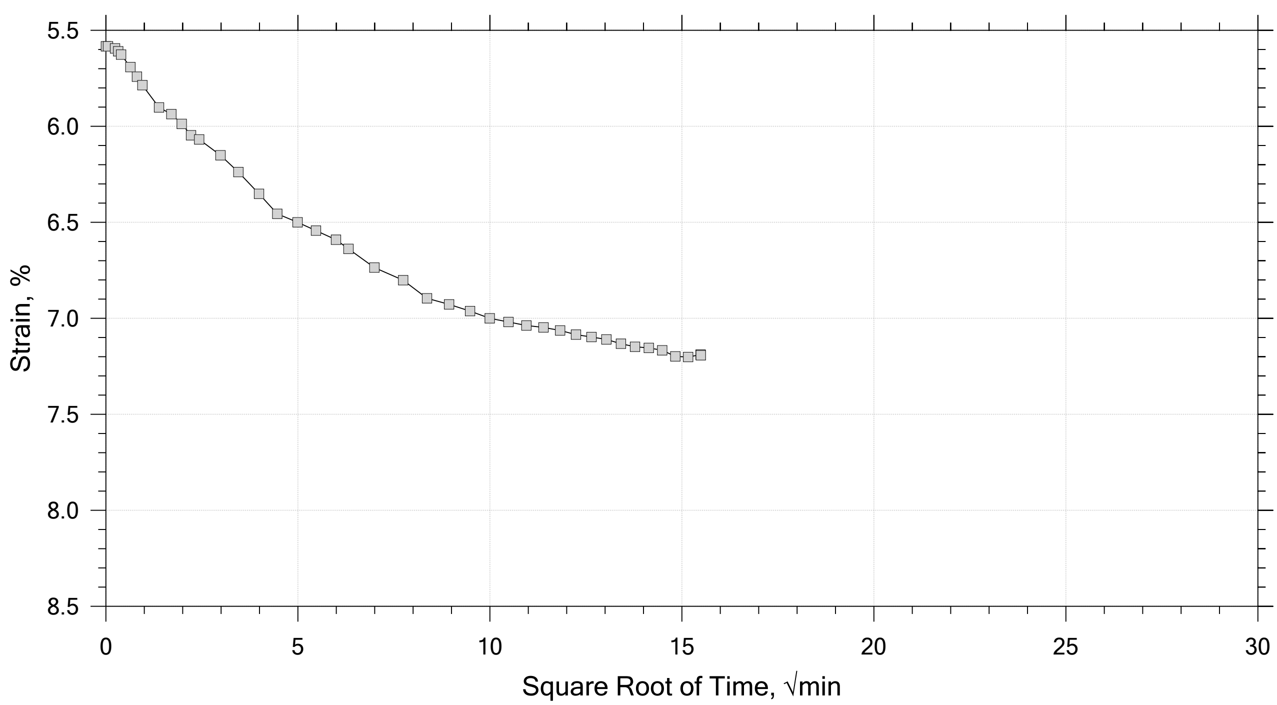
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 17

Constant Load Step

Stress: 2 tsf





Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System LTIII-A		

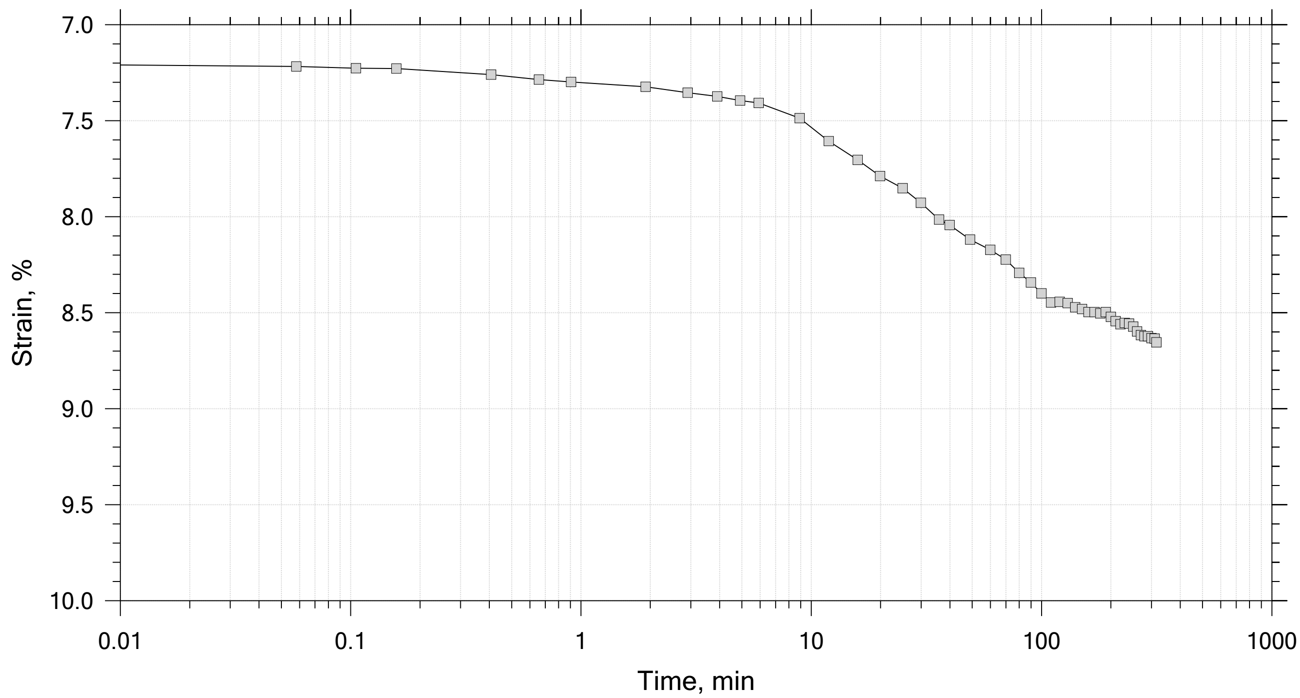
GeoTesting EXPRESS

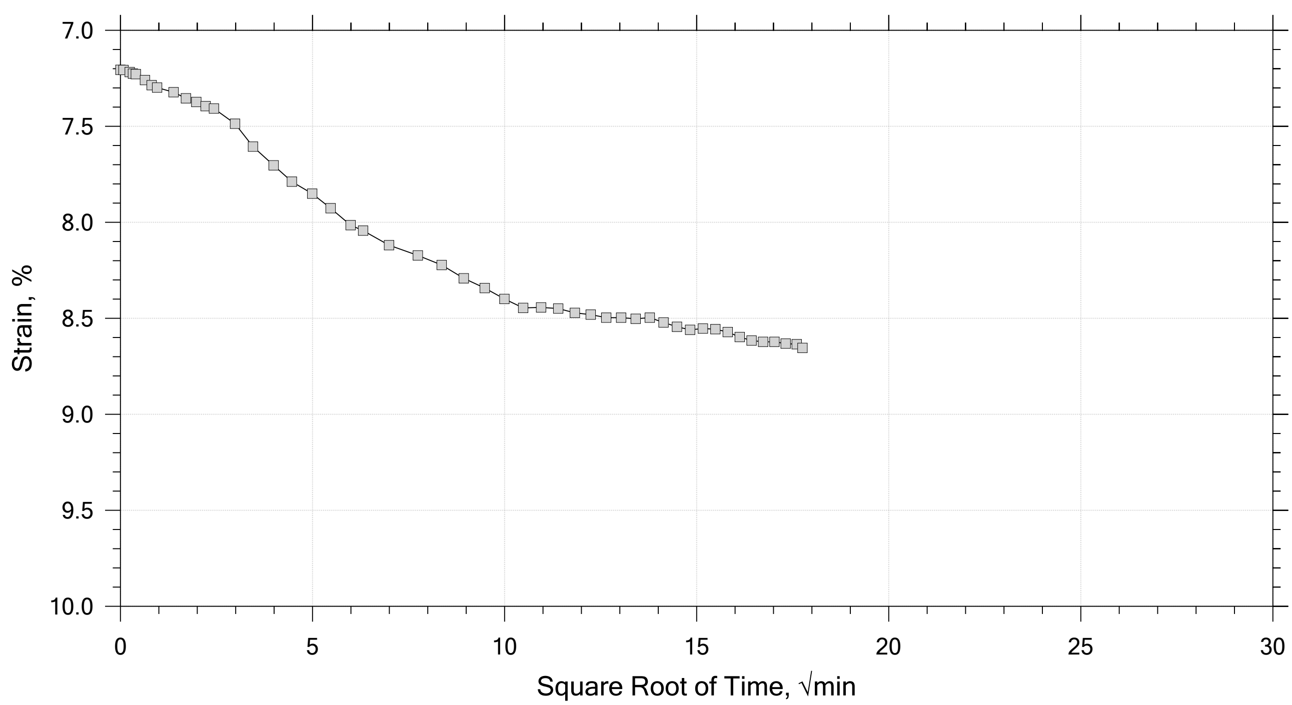
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 17

Constant Load Step

Stress: 2.5 tsf





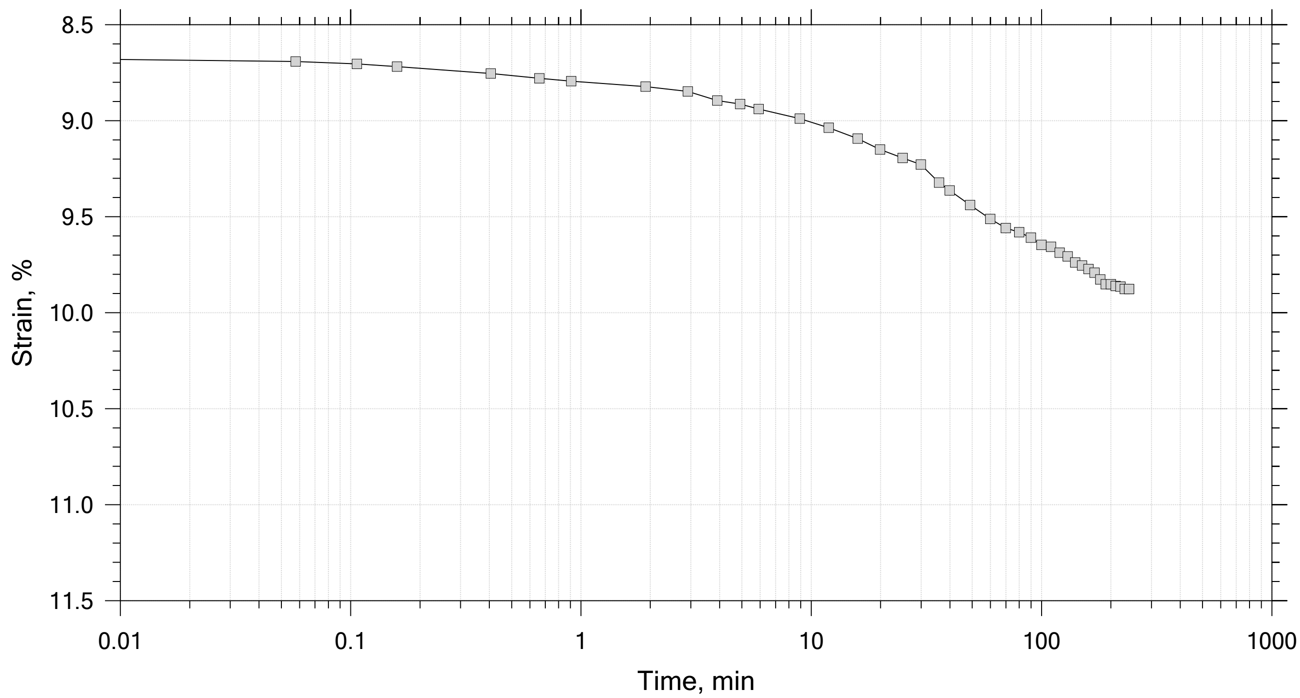
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

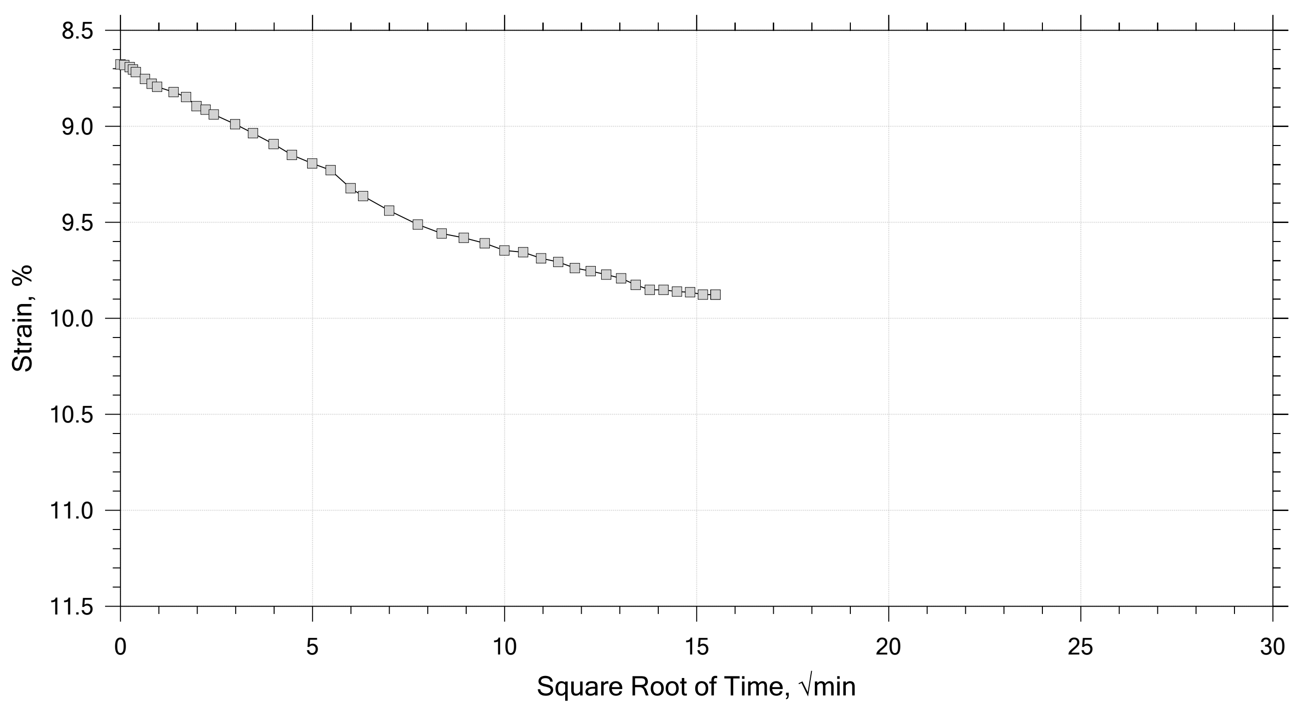
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 17

Constant Load Step

Stress: 3 tsf





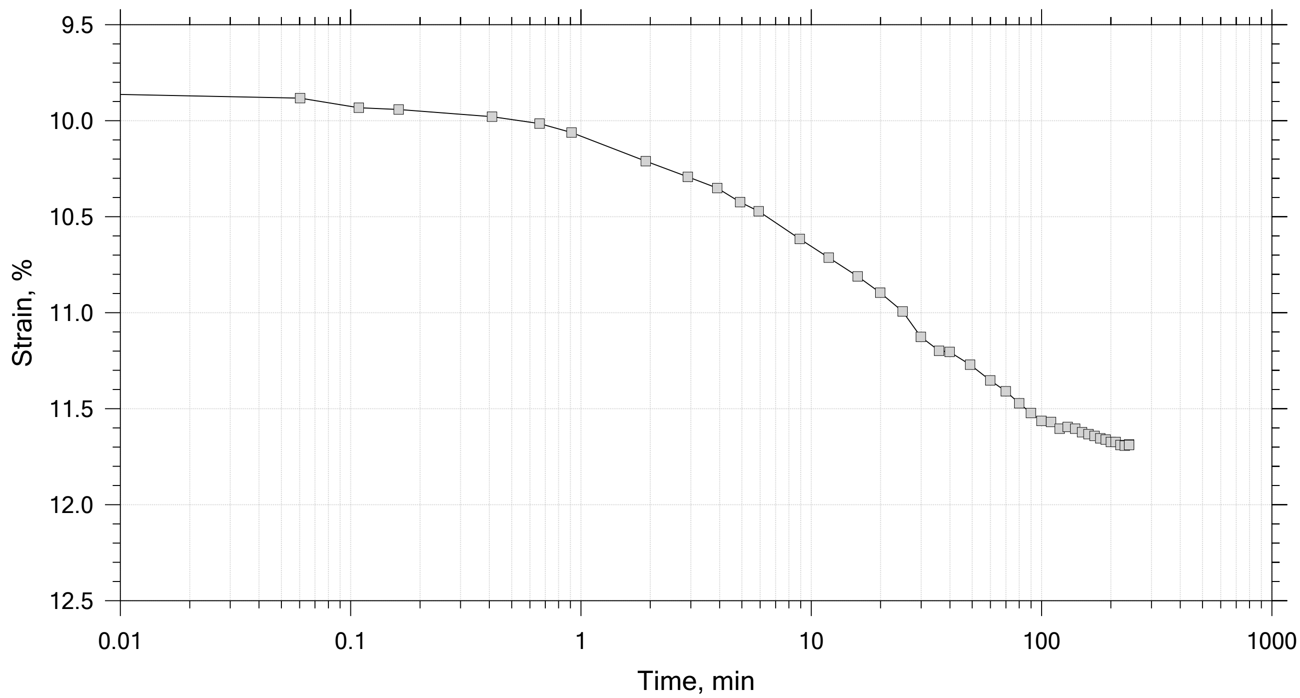
GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

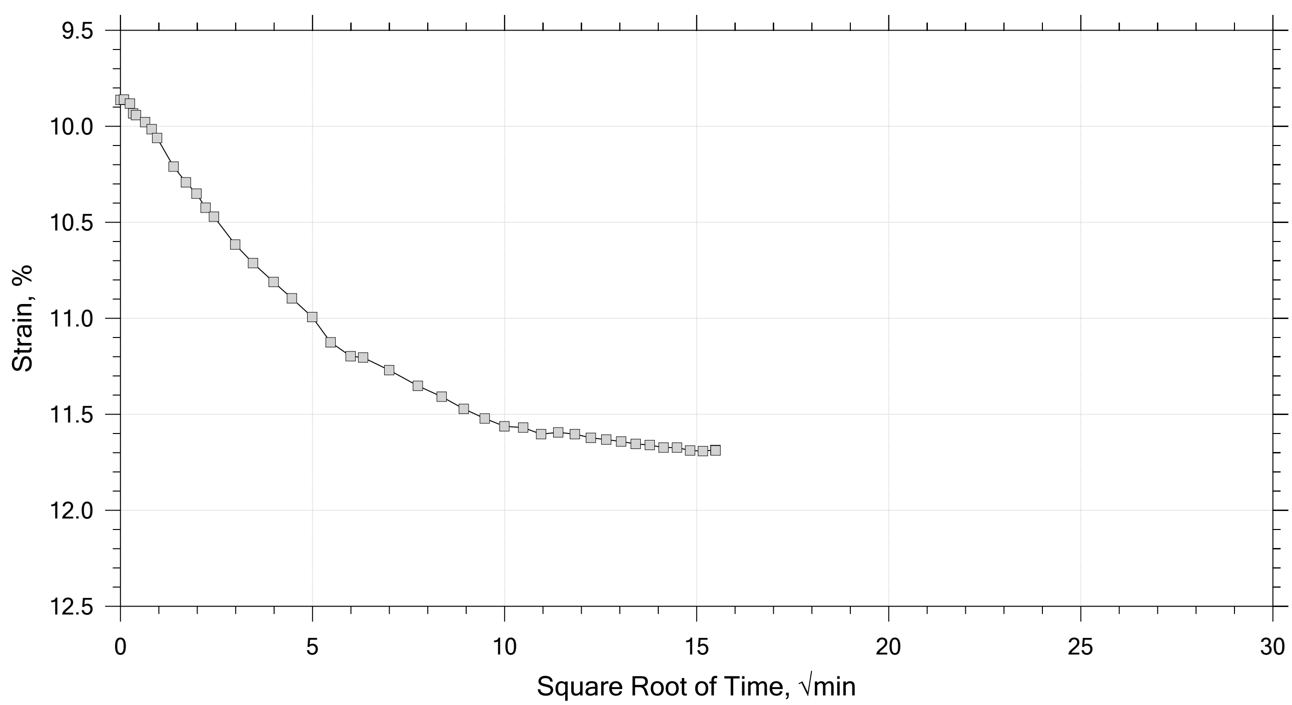
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 17

Constant Load Step

Stress: 4 tsf





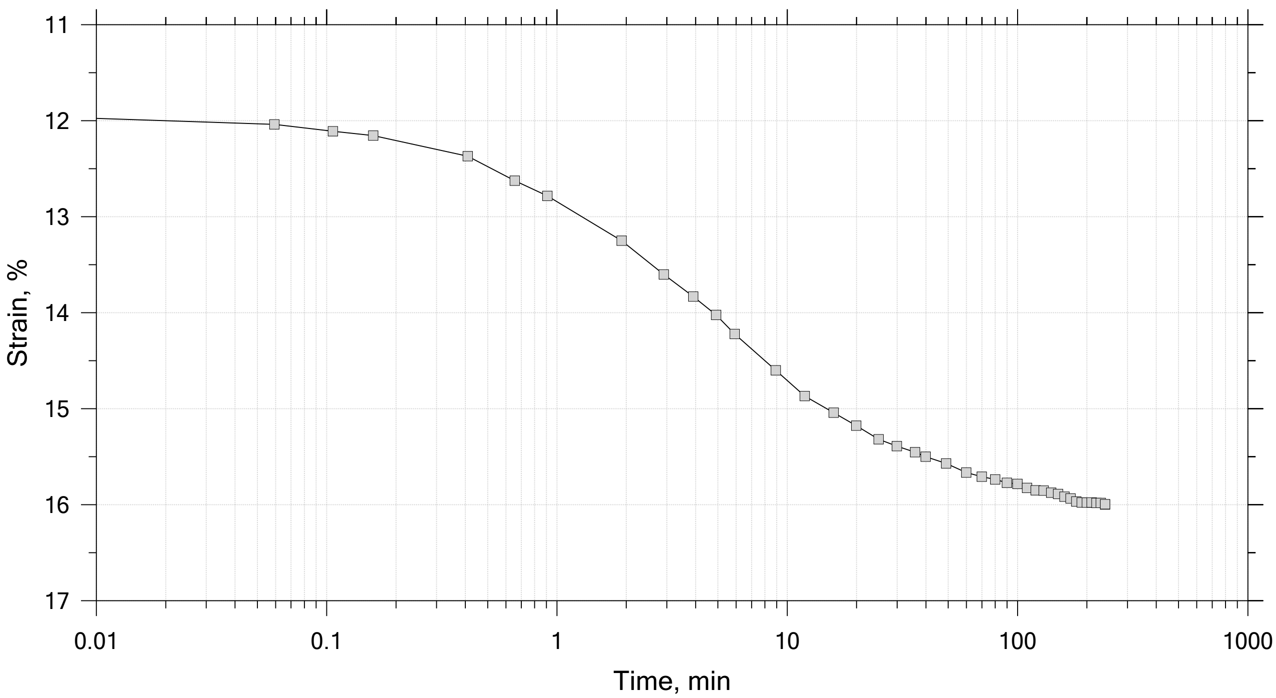
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

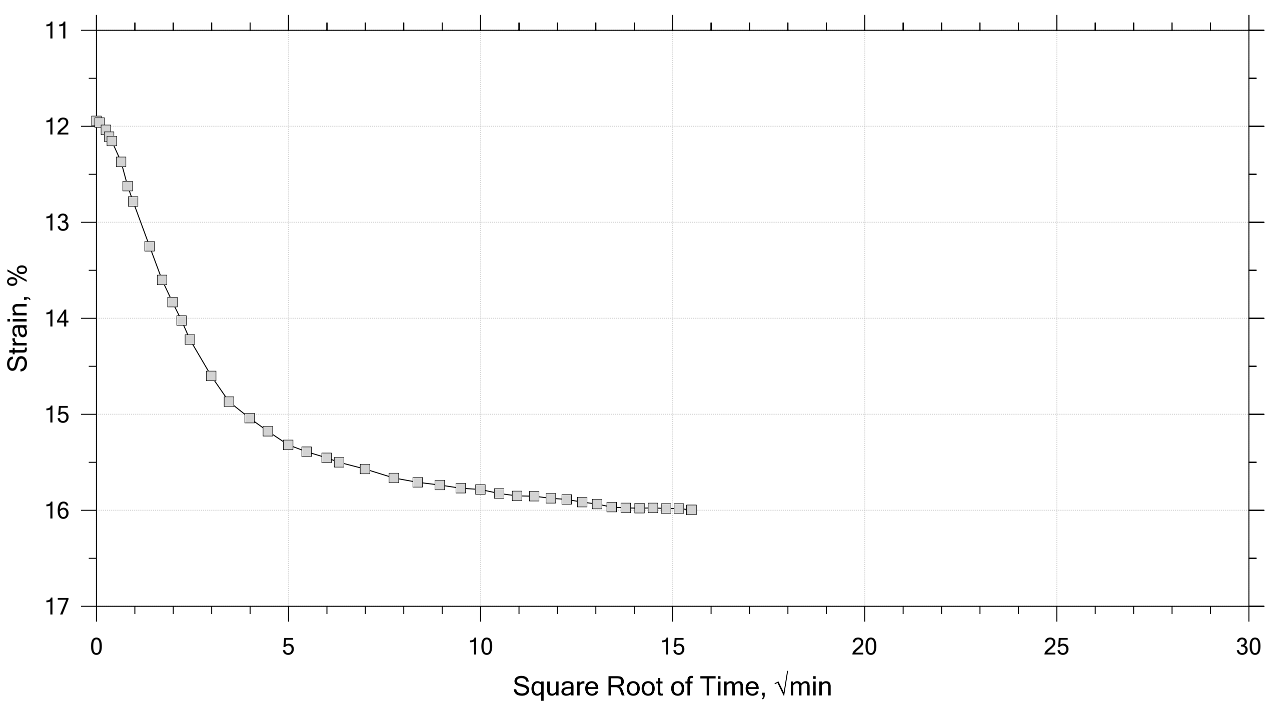
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 17

Constant Load Step

Stress: 8 tsf





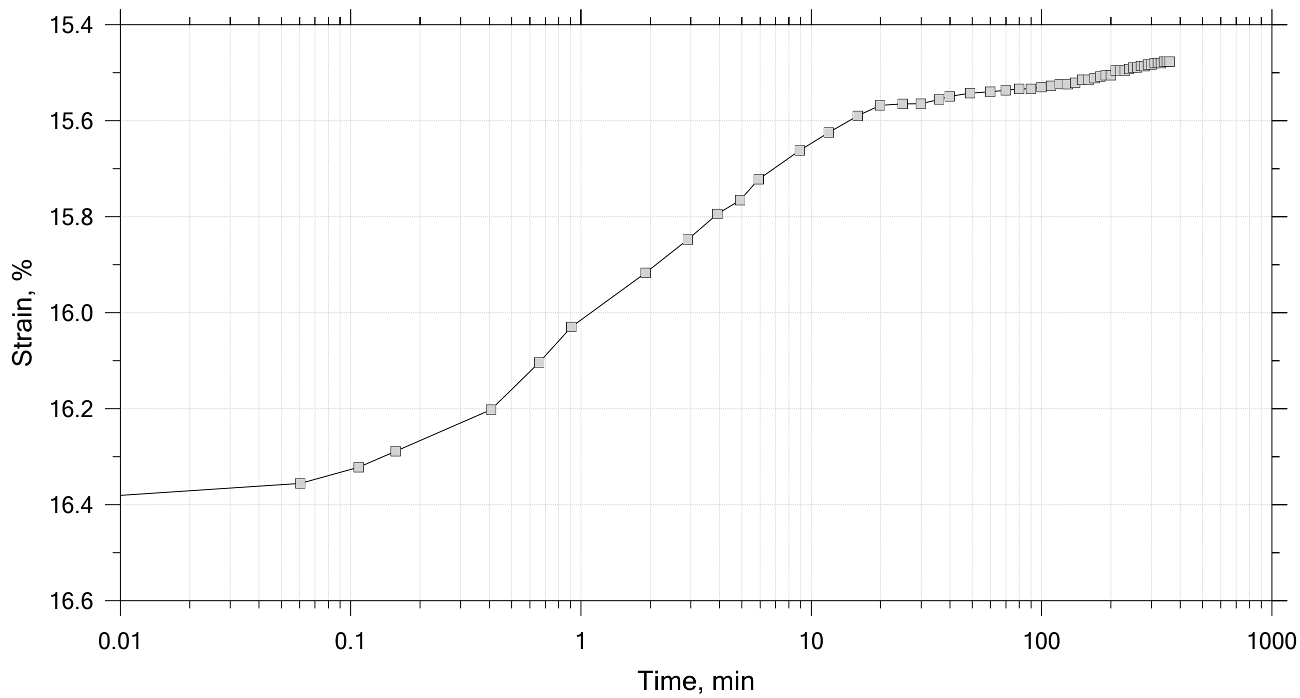
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System LTIII-A		

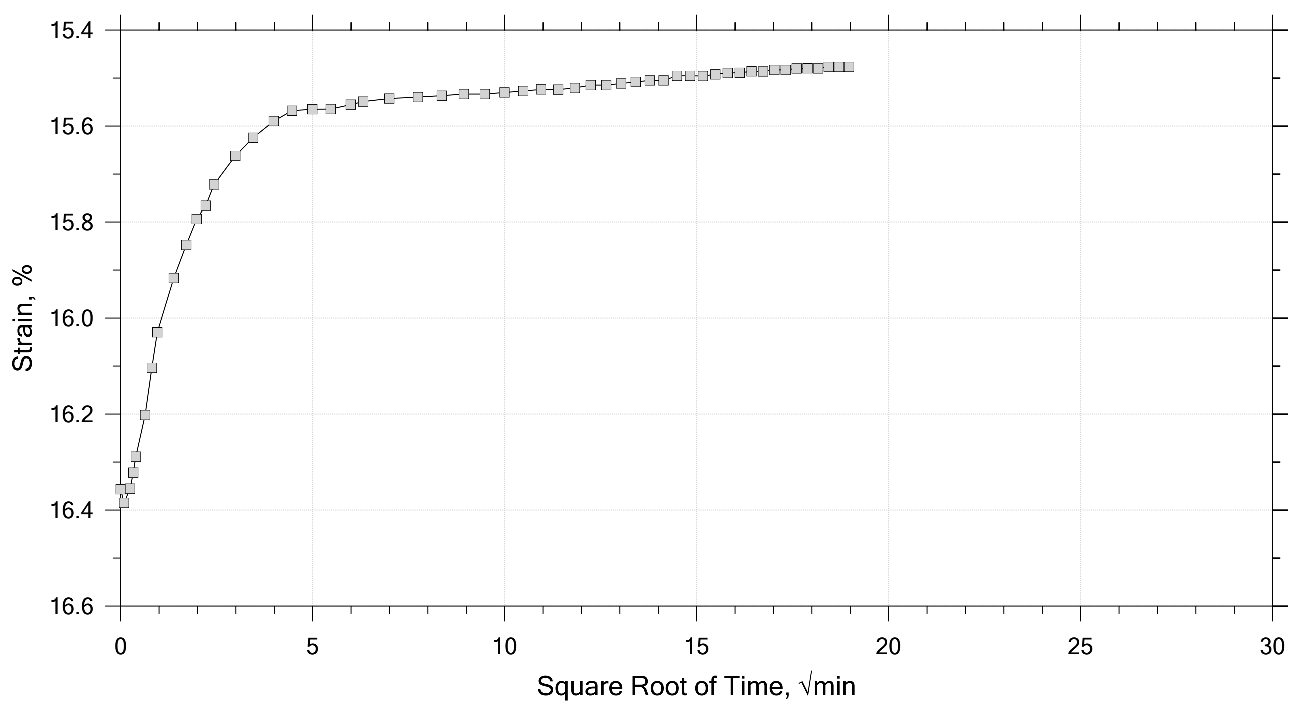
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 17

Constant Load Step

Stress: 1 tsf





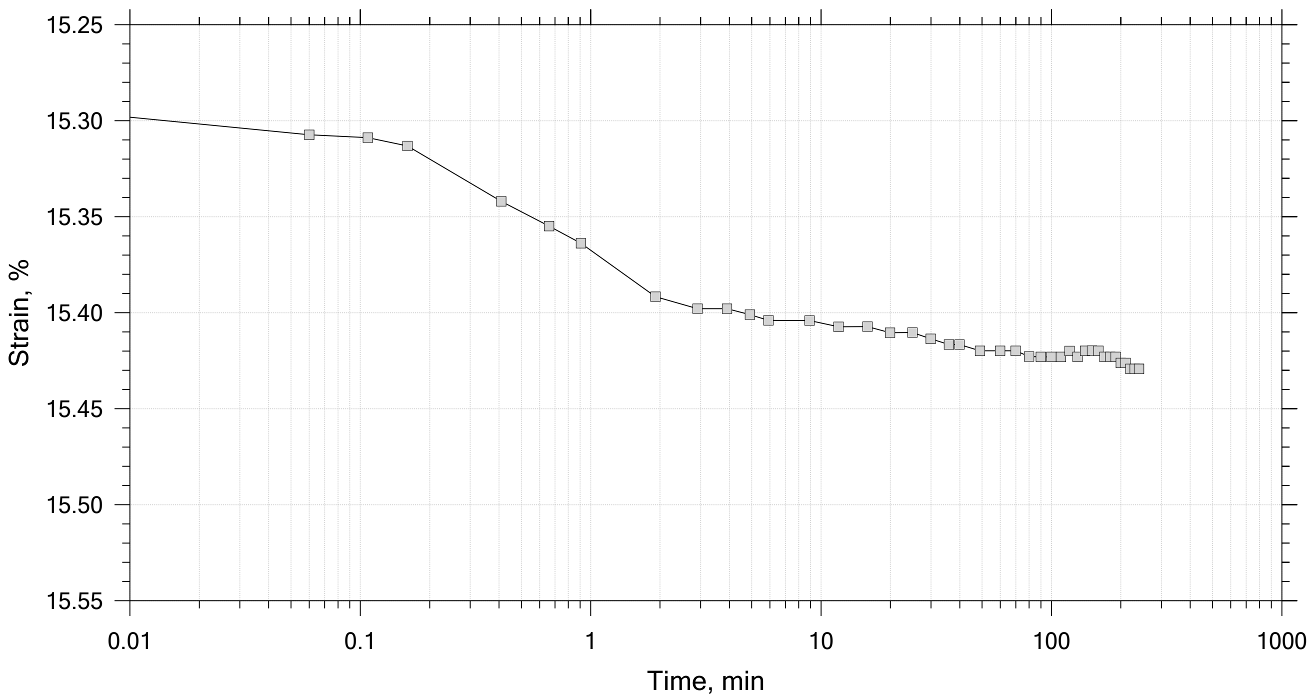
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System LTIII-A		

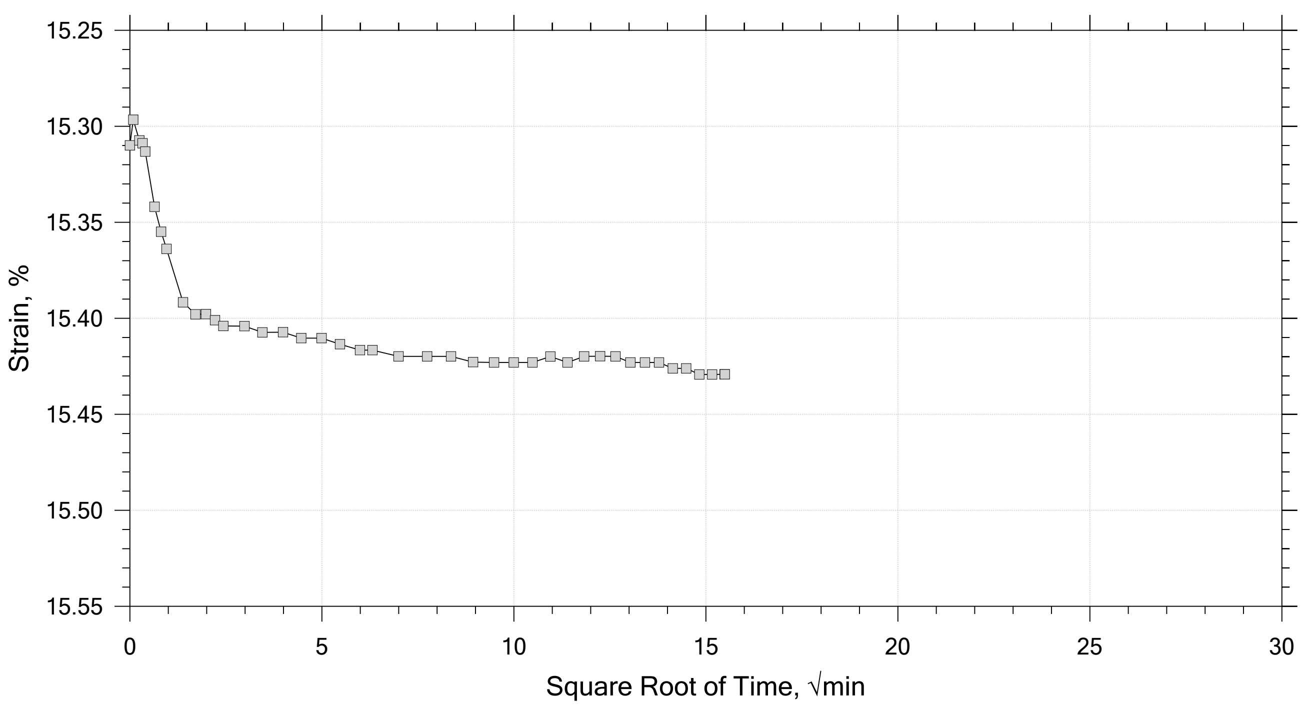
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 17

Constant Load Step

Stress: 2 tsf





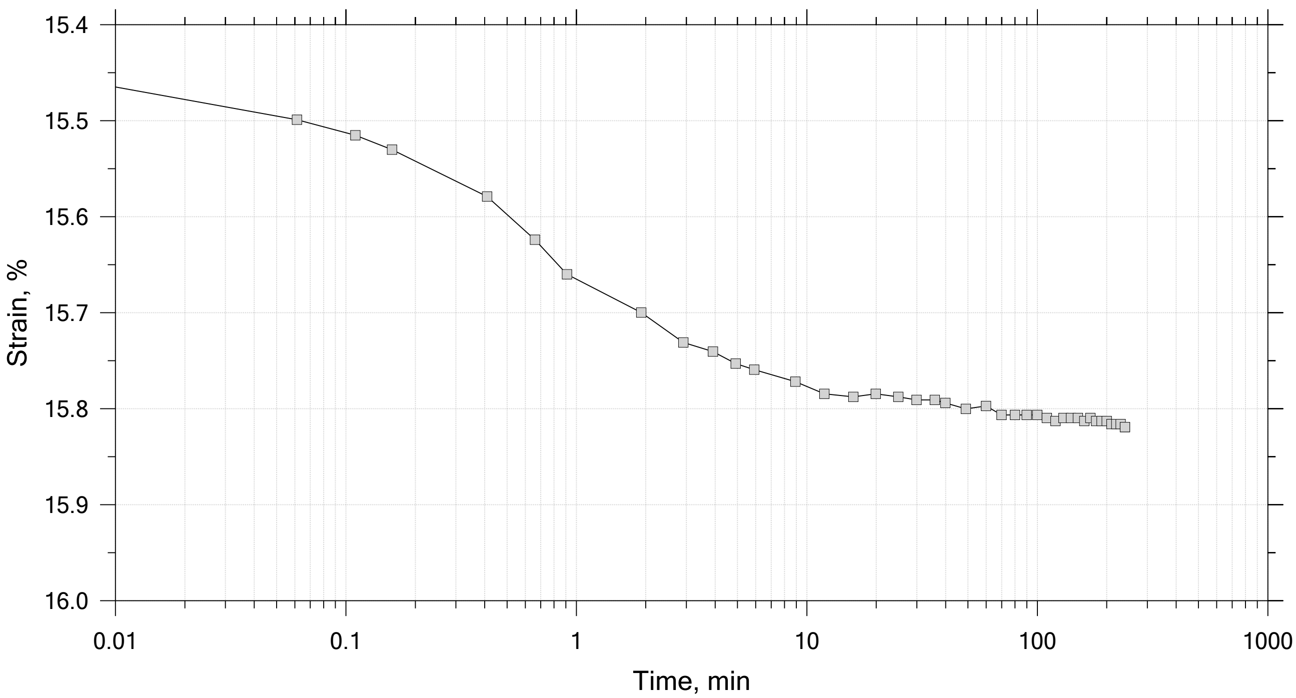
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

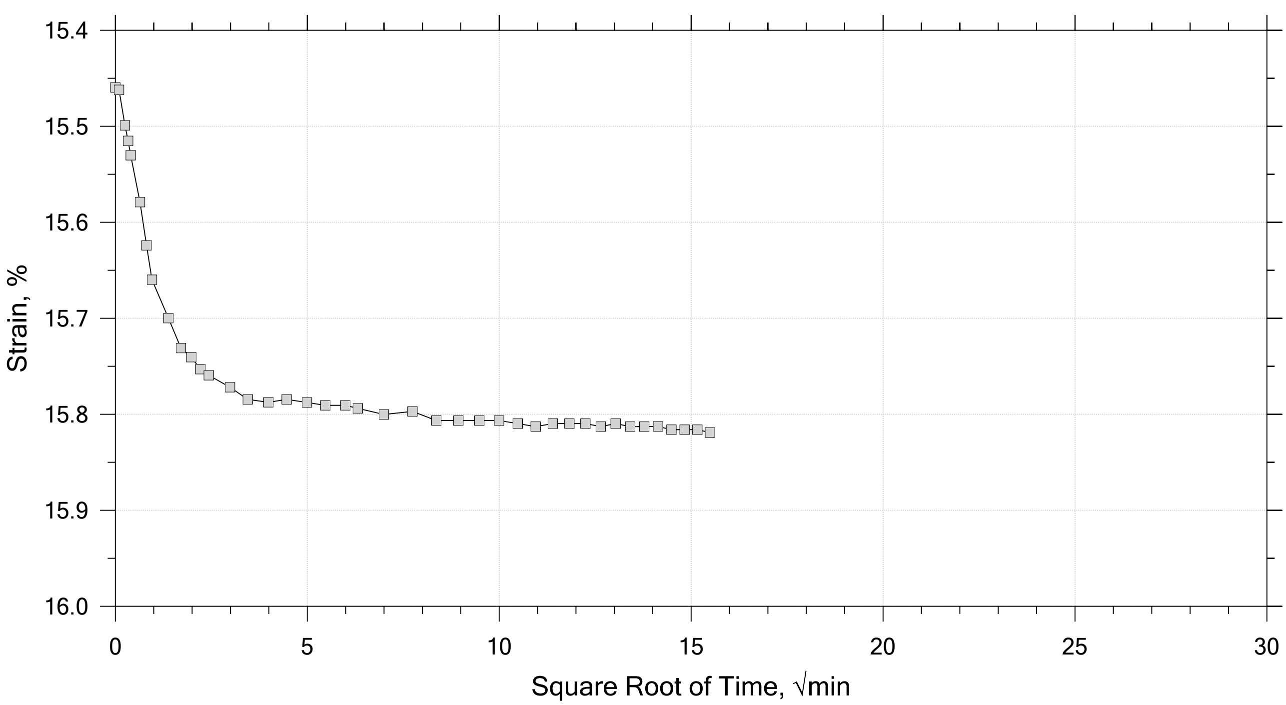
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 17

Constant Load Step

Stress: 4 tsf





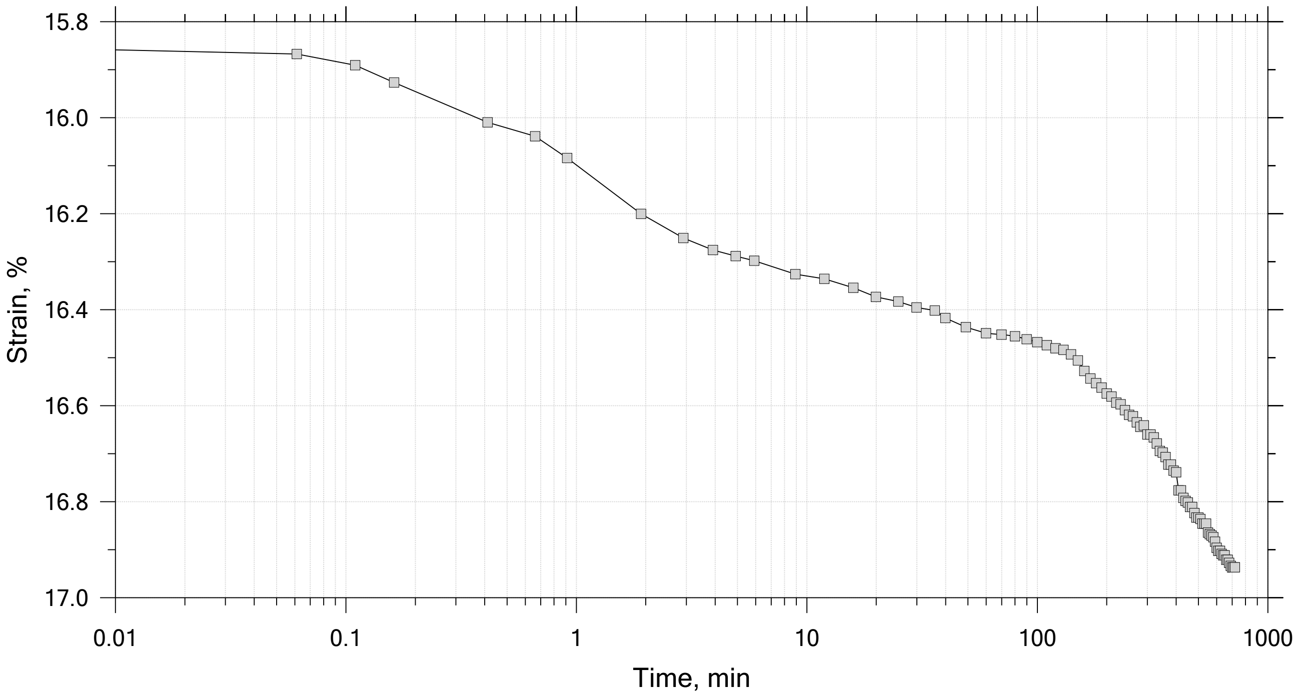
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

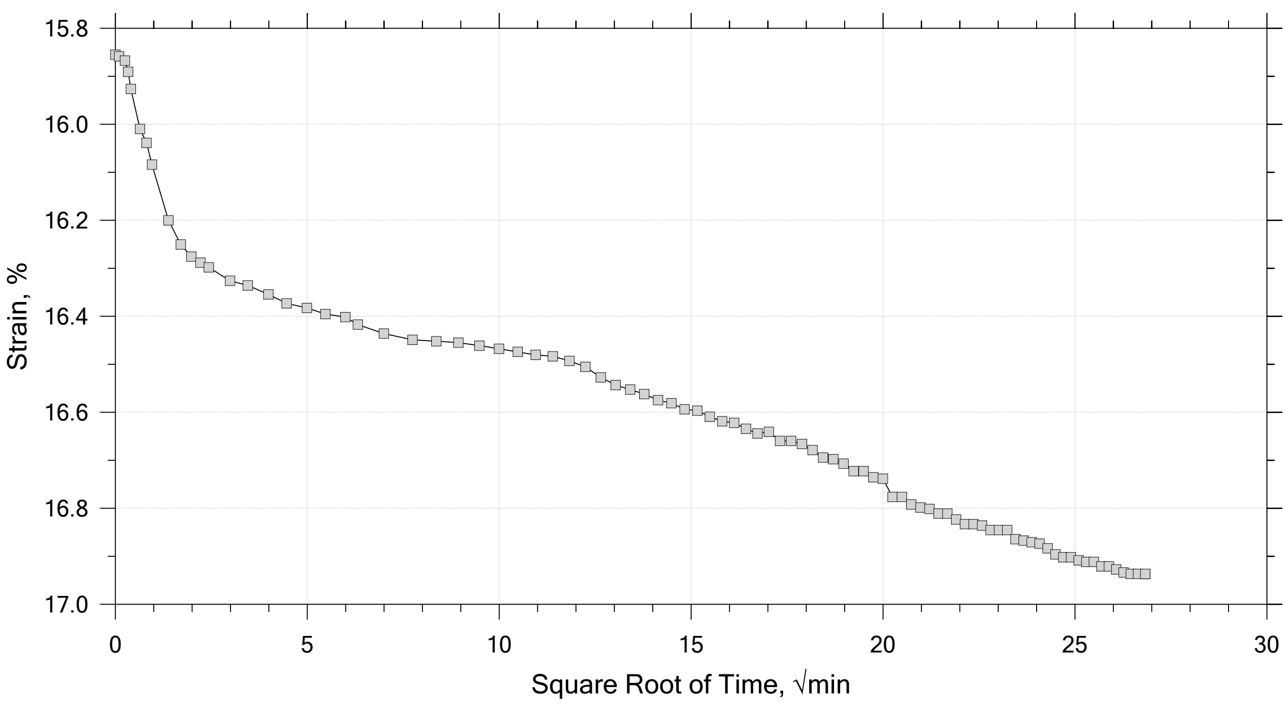
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 17

Constant Load Step

Stress: 7.5 tsf





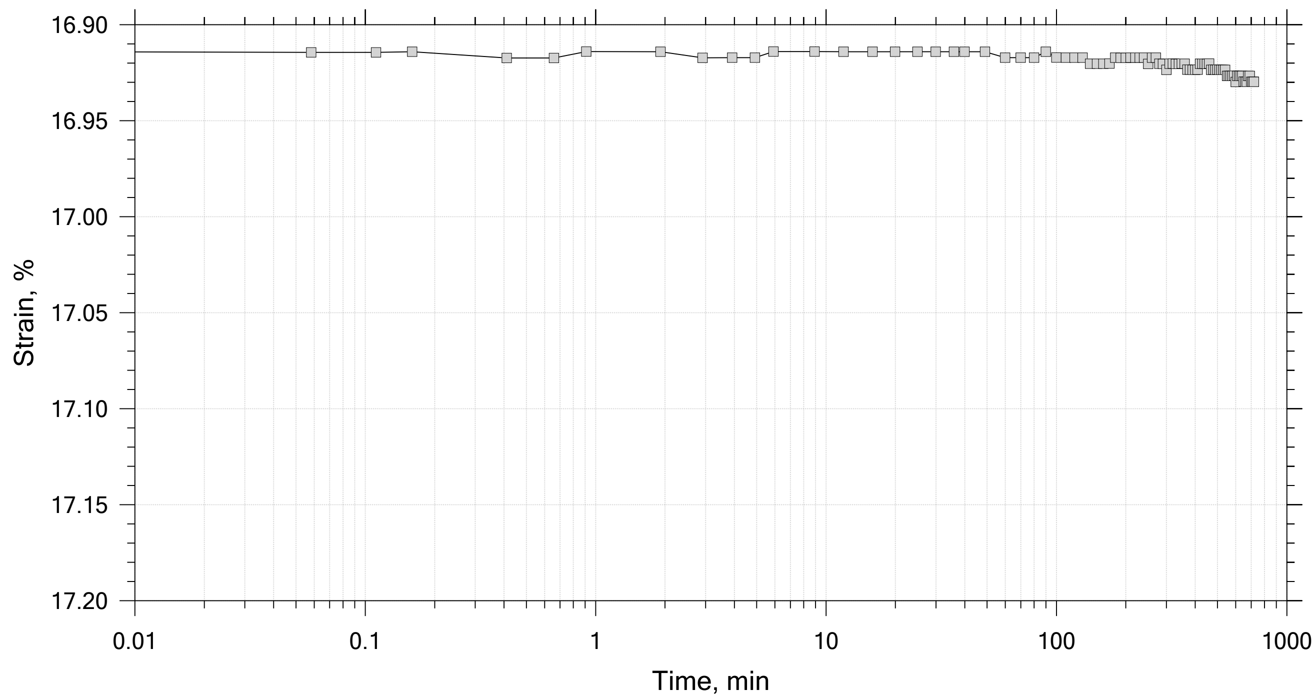
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

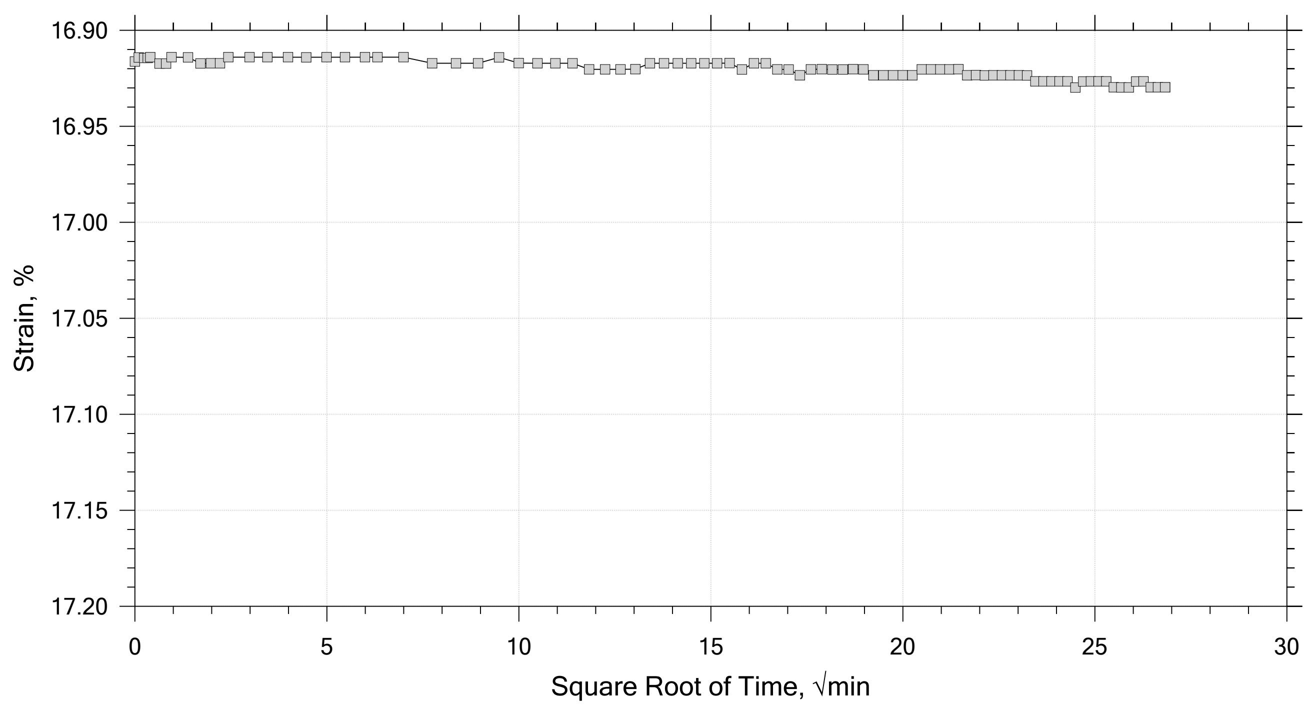
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 17

Constant Load Step

Stress: 8 tsf

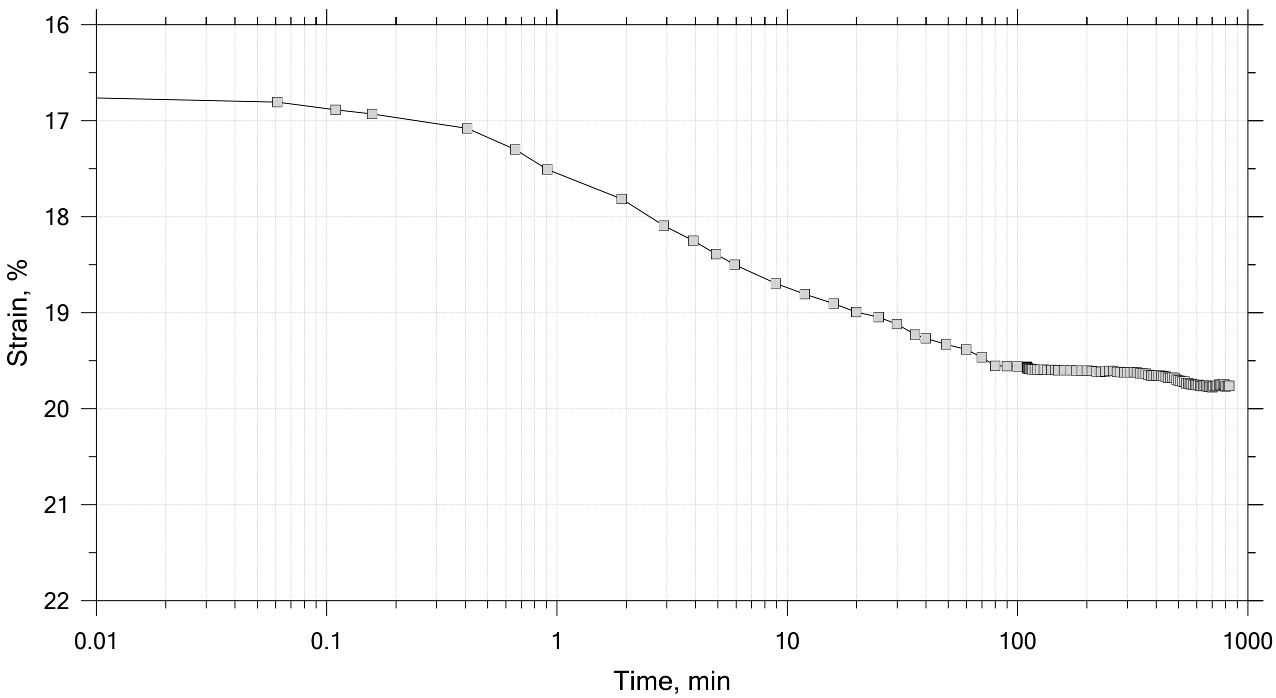


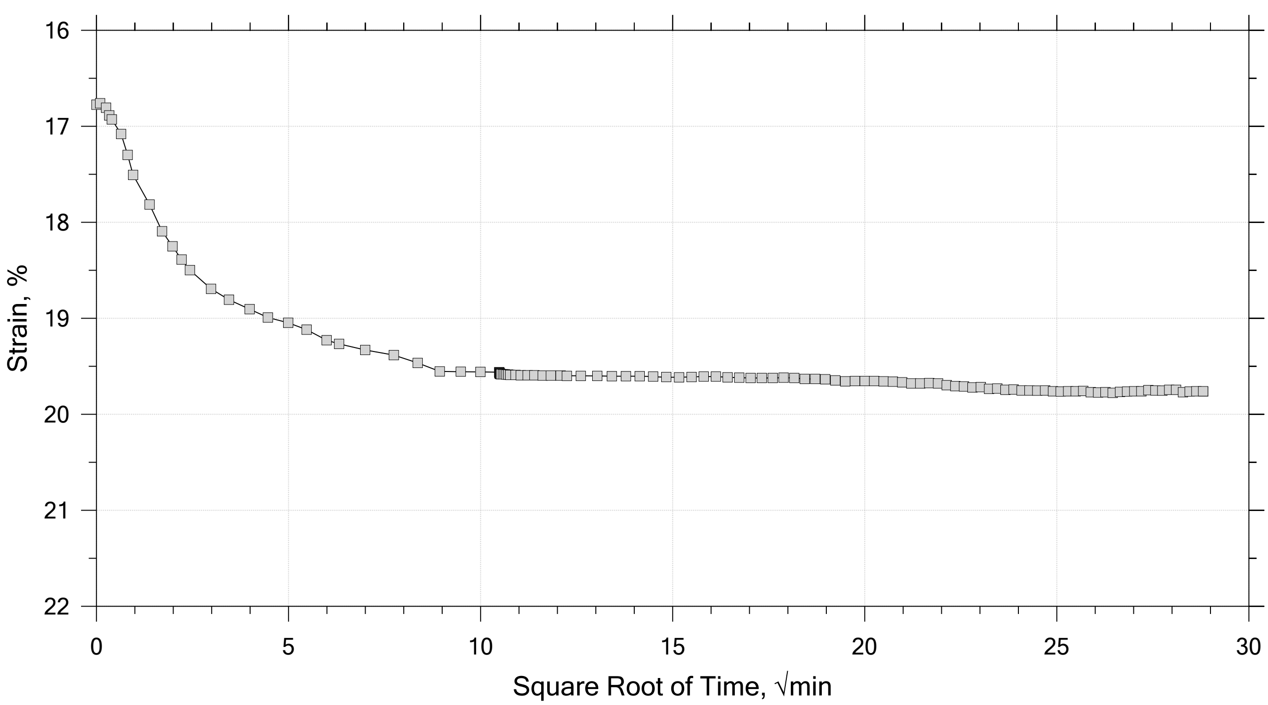


GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 17
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.75	Liquid Limit: 31
Initial Height: 1.00 in	Initial Void Ratio: 0.84	Plastic Limit: 17
Final Height: 0.85 in	Final Void Ratio: 0.564	Plasticity Index: 14

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-2254	RING		D-2576
Mass Container, gm	8.31	108.97	108.97	8.43
Mass Container + Wet Soil, gm	125.79	265.41	253.88	148.39
Mass Container + Dry Soil, gm	97.98	229.23	229.23	124.58
Mass Dry Soil, gm	89.67	120.26	120.26	116.15
Water Content, %	31.01	30.09	20.50	20.50
Void Ratio	---	0.84	0.56	---
Degree of Saturation, %	---	98.54	100.00	---
Dry Unit Weight, pcf	---	93.33	109.8	---

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: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		

One-Dimensional Consolidation by ASTM D2435 - Method B

Log of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/s	Mv 1/tsf	k ft/day	Ca %
1	0.125	0.01345	0.815	1.35	0.000	0.00e+00	1.08e-01	0.00e+00	0.00e+00
2	0.250	0.02010	0.803	2.01	1.460	3.77e-06	5.31e-02	5.41e-04	0.00e+00
3	0.500	0.02753	0.789	2.75	0.000	0.00e+00	2.97e-02	0.00e+00	0.00e+00
4	0.750	0.03354	0.778	3.35	0.000	0.00e+00	2.41e-02	0.00e+00	0.00e+00
5	1.00	0.03990	0.766	3.99	0.000	0.00e+00	2.54e-02	0.00e+00	0.00e+00
6	1.50	0.05582	0.737	5.58	0.000	0.00e+00	3.18e-02	0.00e+00	0.00e+00
7	2.00	0.07192	0.707	7.19	0.000	0.00e+00	3.22e-02	0.00e+00	0.00e+00
8	2.50	0.08654	0.681	8.65	0.000	0.00e+00	2.92e-02	0.00e+00	0.00e+00
9	3.00	0.09877	0.658	9.88	0.000	0.00e+00	2.45e-02	0.00e+00	0.00e+00
10	4.00	0.1169	0.625	11.7	13.865	3.27e-07	1.81e-02	1.60e-05	0.00e+00
11	8.00	0.1599	0.546	16.0	2.823	1.50e-06	1.08e-02	4.35e-05	0.00e+00
12	1.00	0.1548	0.555	15.5	1.409	2.87e-06	7.40e-04	5.73e-06	0.00e+00
13	2.00	0.1543	0.556	15.4	0.000	0.00e+00	-4.75e-04	-0.00e+00	0.00e+00
14	4.00	0.1582	0.549	15.8	0.709	5.73e-06	1.95e-03	3.01e-05	0.00e+00
15	7.50	0.1694	0.528	16.9	0.000	0.00e+00	3.19e-03	0.00e+00	0.00e+00
16	8.00	0.1693	0.528	16.9	291.198	1.35e-08	-1.42e-04	-5.17e-09	0.00e+00
17	16.0	0.1976	0.476	19.8	0.000	0.00e+00	3.54e-03	0.00e+00	0.00e+00

Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System LTIII-A		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

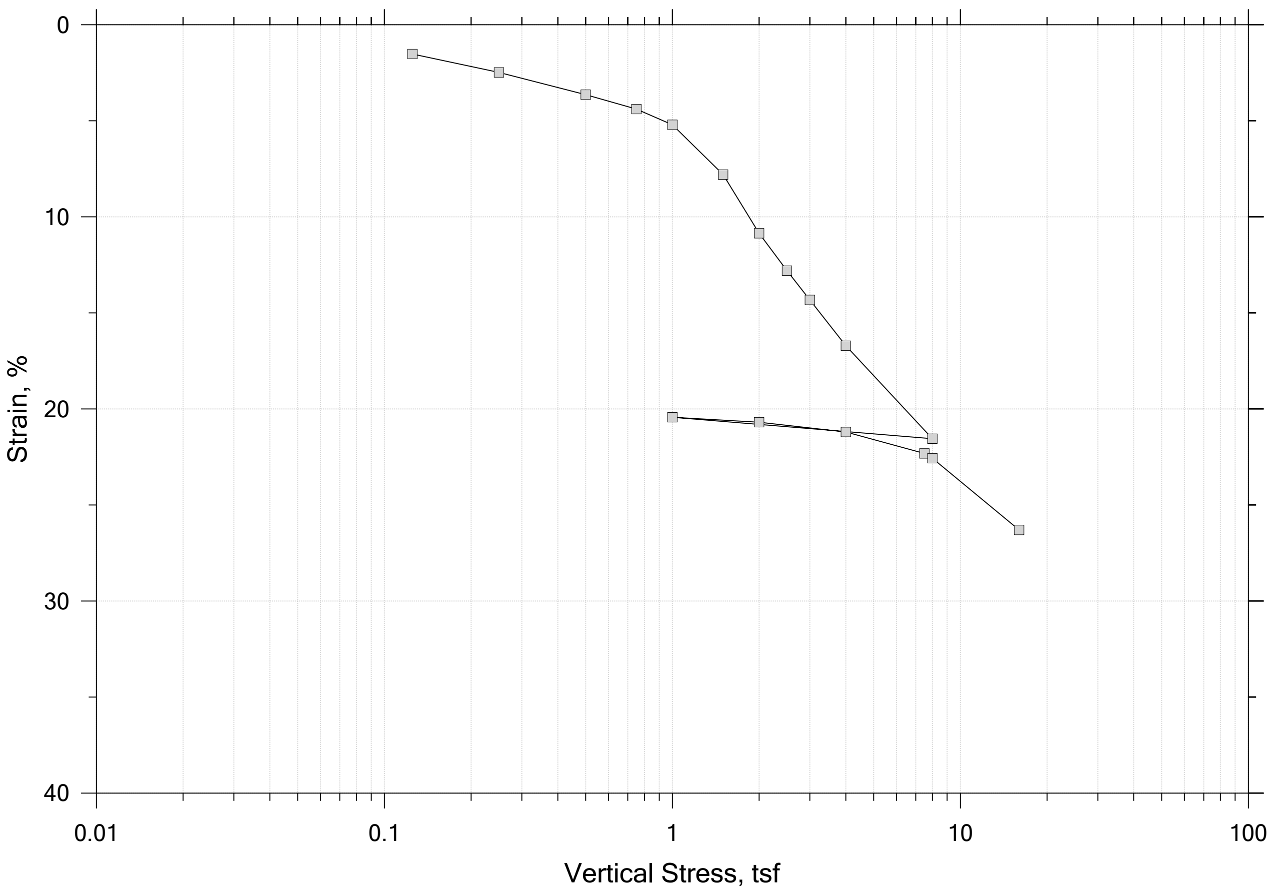
Square Root of Time Coefficients

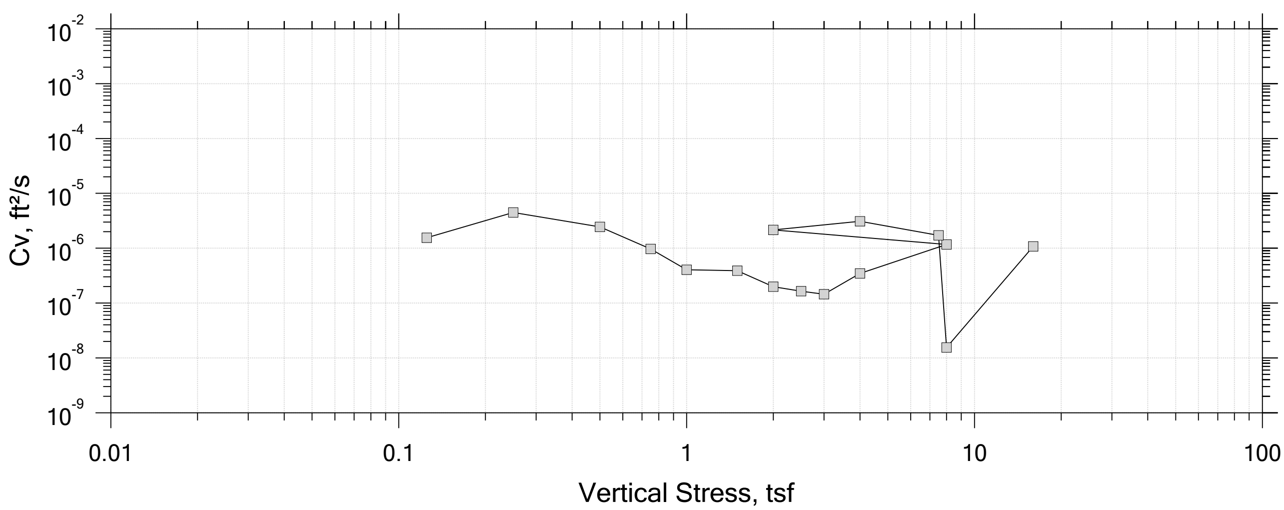
Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv ft²/s	Mv 1/tsf	k ft/day
1	0.125	0.01345	0.815	1.35	14.914	1.62e-06	1.08e-01	4.71e-04
2	0.250	0.02010	0.803	2.01	7.906	3.00e-06	5.31e-02	4.30e-04
3	0.500	0.02753	0.789	2.75	20.195	1.16e-06	2.97e-02	9.28e-05
4	0.750	0.03354	0.778	3.35	19.338	1.19e-06	2.41e-02	7.74e-05
5	1.00	0.03990	0.766	3.99	18.546	1.23e-06	2.54e-02	8.42e-05
6	1.50	0.05582	0.737	5.58	46.506	4.78e-07	3.18e-02	4.11e-05
7	2.00	0.07192	0.707	7.19	49.190	4.37e-07	3.22e-02	3.80e-05
8	2.50	0.08654	0.681	8.65	109.463	1.90e-07	2.92e-02	1.50e-05
9	3.00	0.09877	0.658	9.88	125.610	1.61e-07	2.45e-02	1.06e-05
10	4.00	0.1169	0.625	11.7	42.202	4.63e-07	1.81e-02	2.26e-05
11	8.00	0.1599	0.546	16.0	16.604	1.10e-06	1.08e-02	3.18e-05
12	1.00	0.1548	0.555	15.5	7.462	2.33e-06	7.40e-04	4.66e-06
13	2.00	0.1543	0.556	15.4	2.896	6.06e-06	-4.75e-04	-7.76e-06
14	4.00	0.1582	0.549	15.8	3.397	5.14e-06	1.95e-03	2.70e-05
15	7.50	0.1694	0.528	16.9	595.700	2.88e-08	3.19e-03	2.48e-07
16	8.00	0.1693	0.528	16.9	0.000	0.00e+00	-1.42e-04	-0.00e+00
17	16.0	0.1976	0.476	19.8	47.805	3.42e-07	3.54e-03	3.27e-06

GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-103	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 55-57 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System LTIII-A		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report





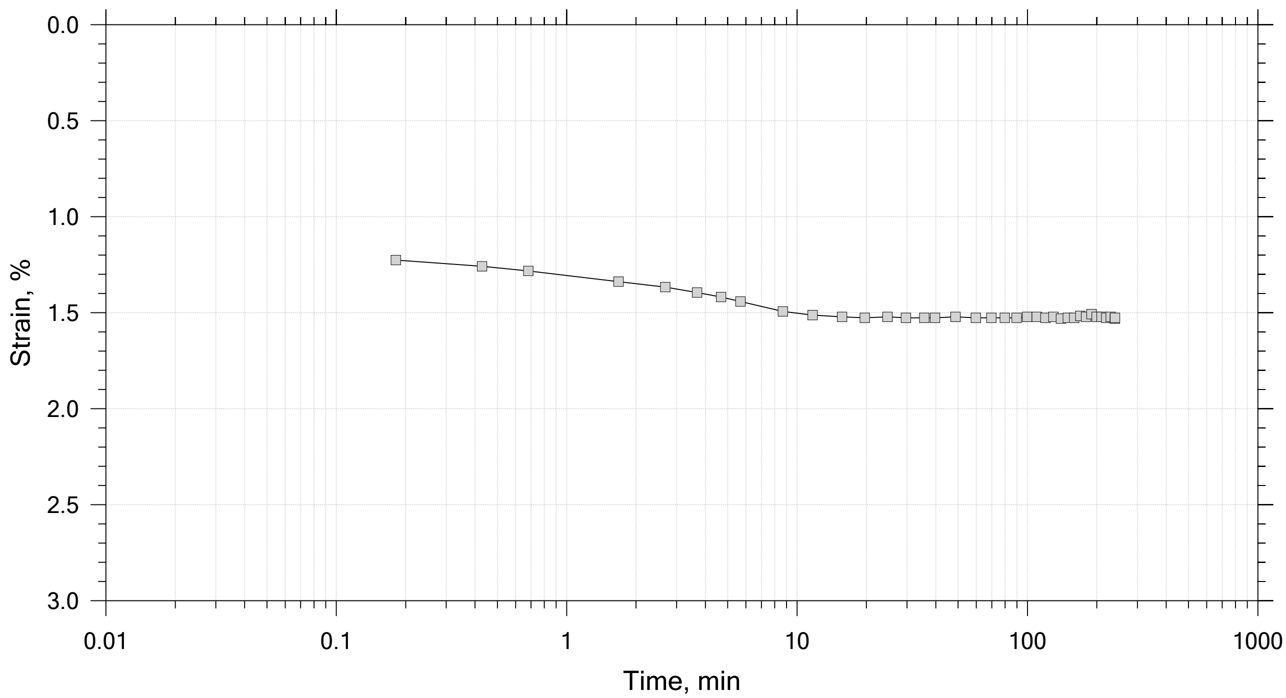
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		
	Displacement at End of Increment		

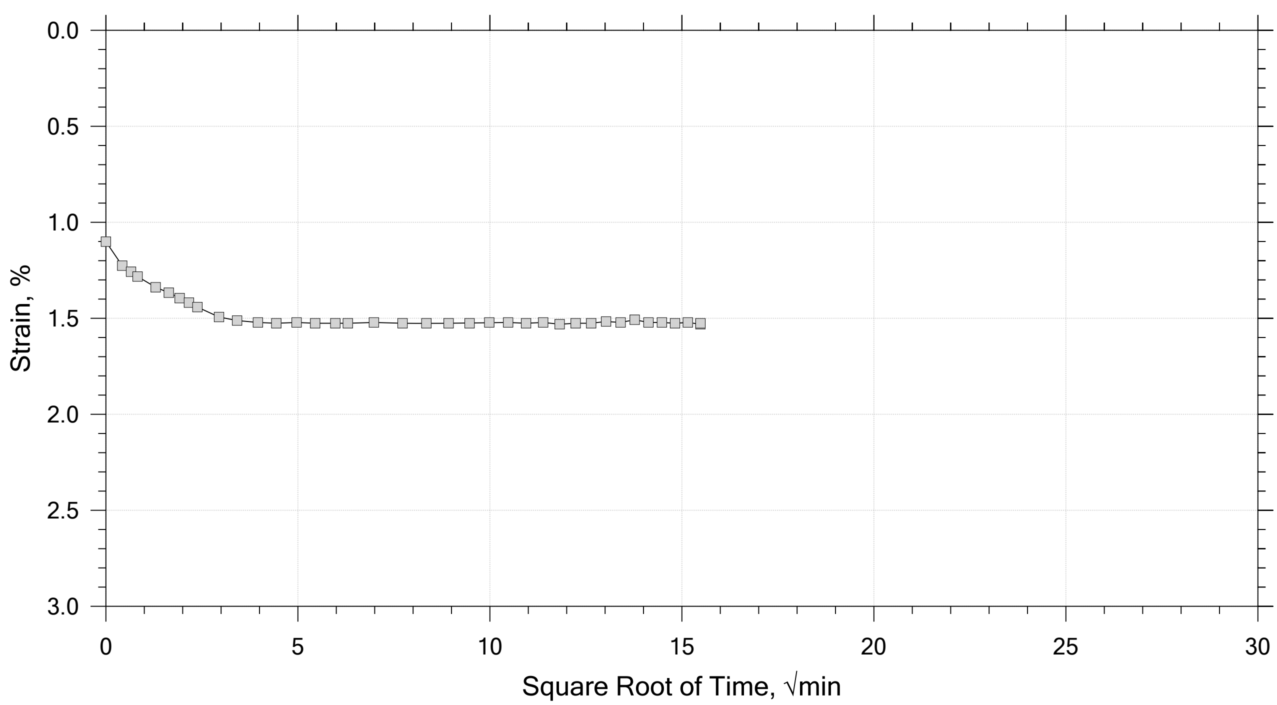
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 17

Constant Load Step

Stress: 0.125 tsf





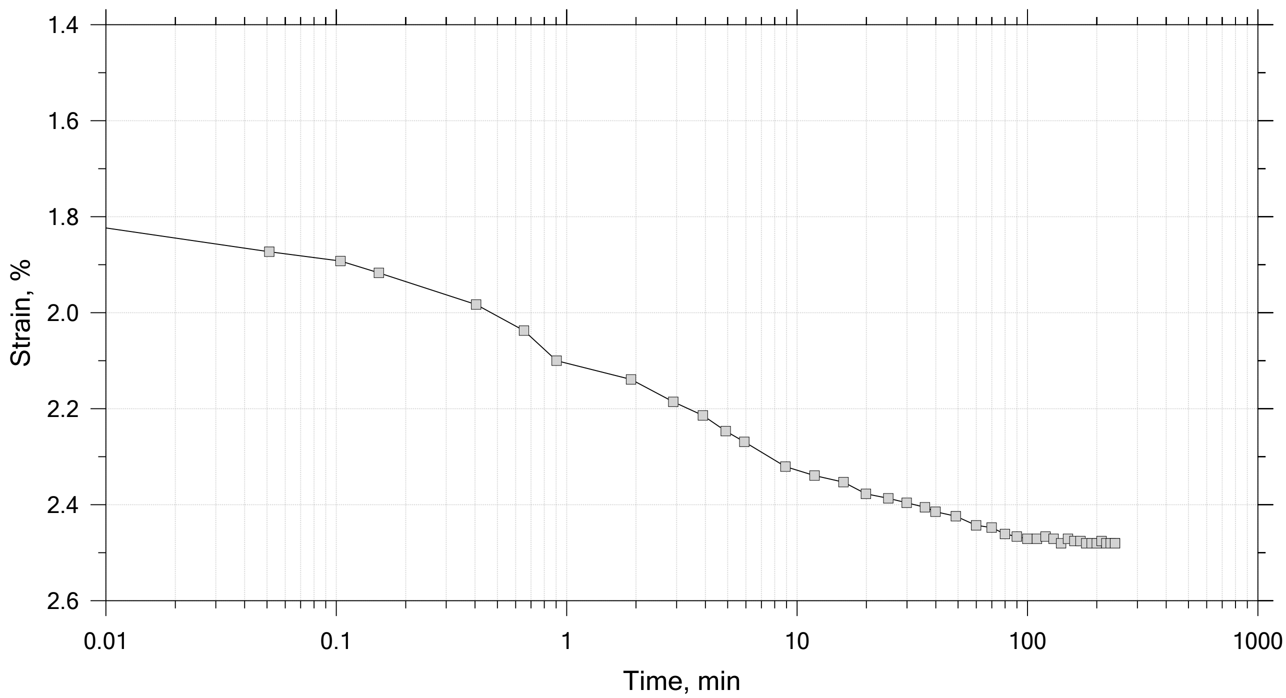
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

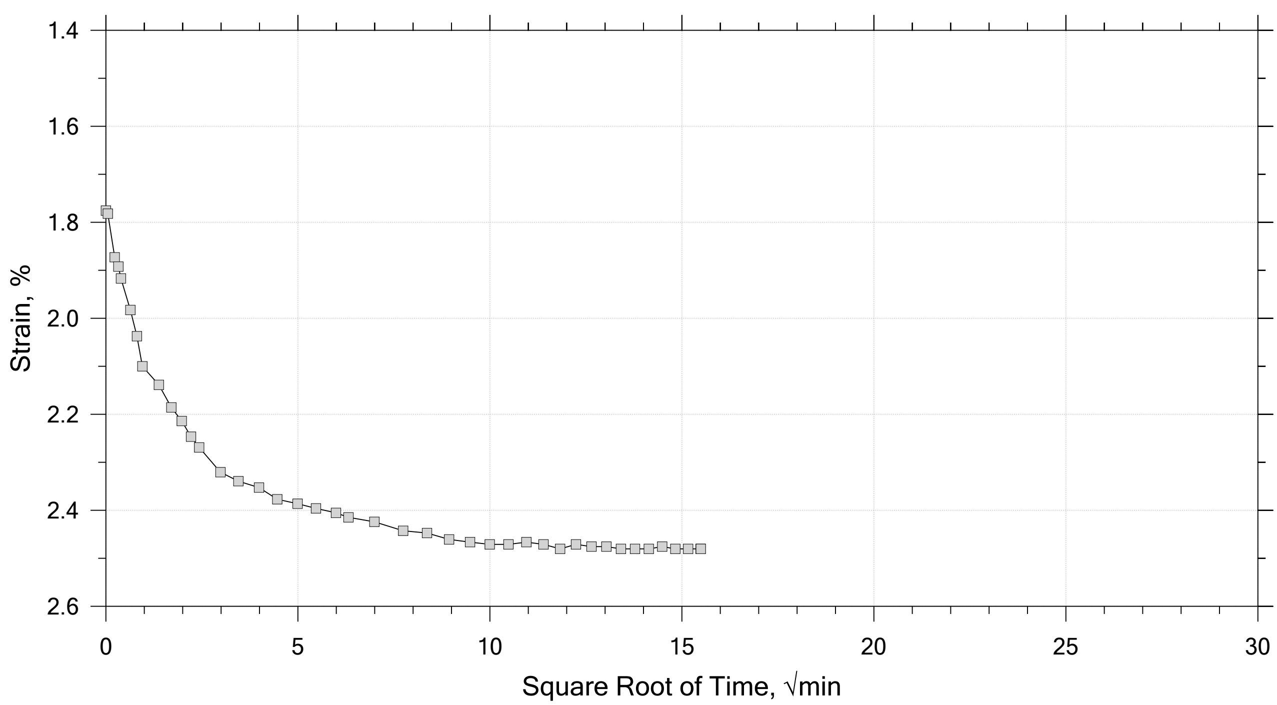
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 17

Constant Load Step

Stress: 0.25 tsf

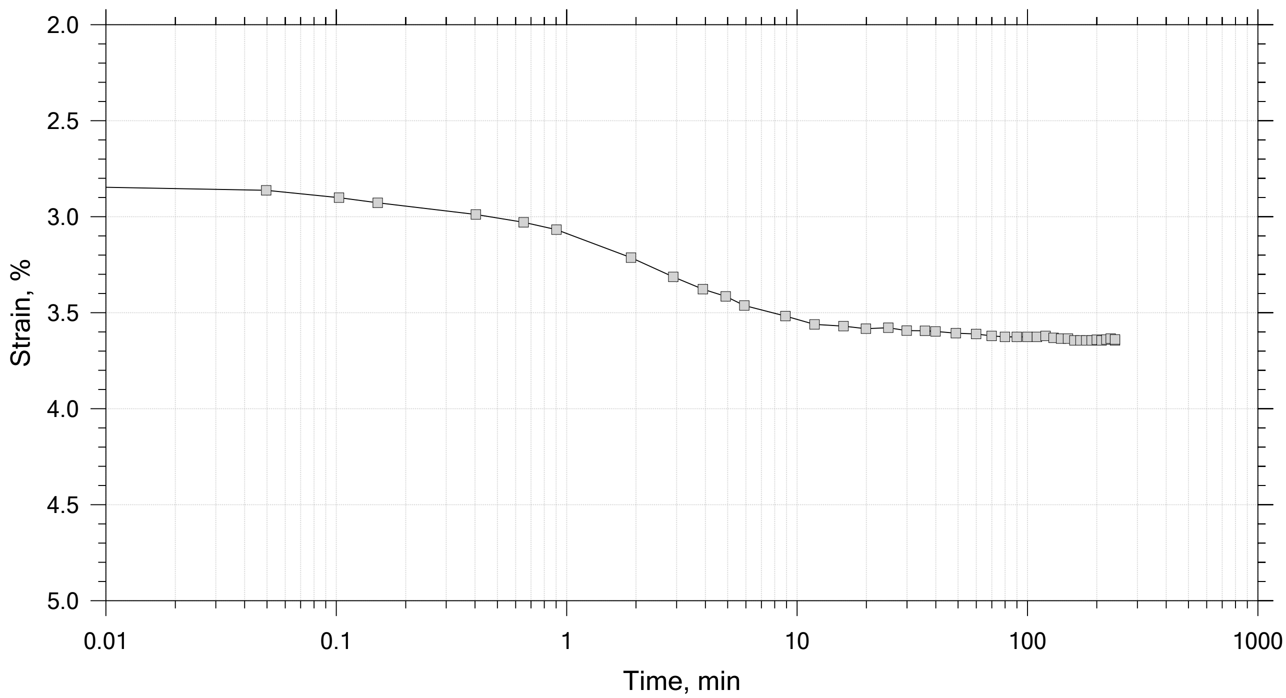


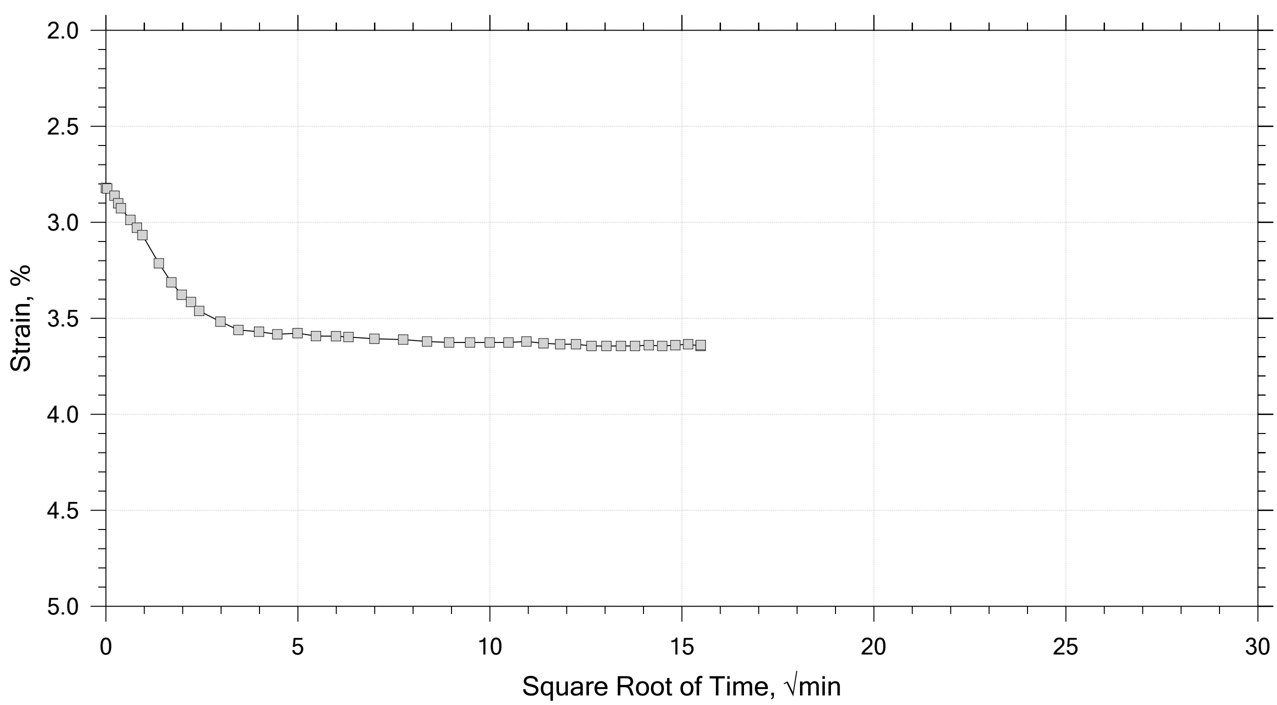


GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 17
Constant Load Step
Stress: 0.5 tsf





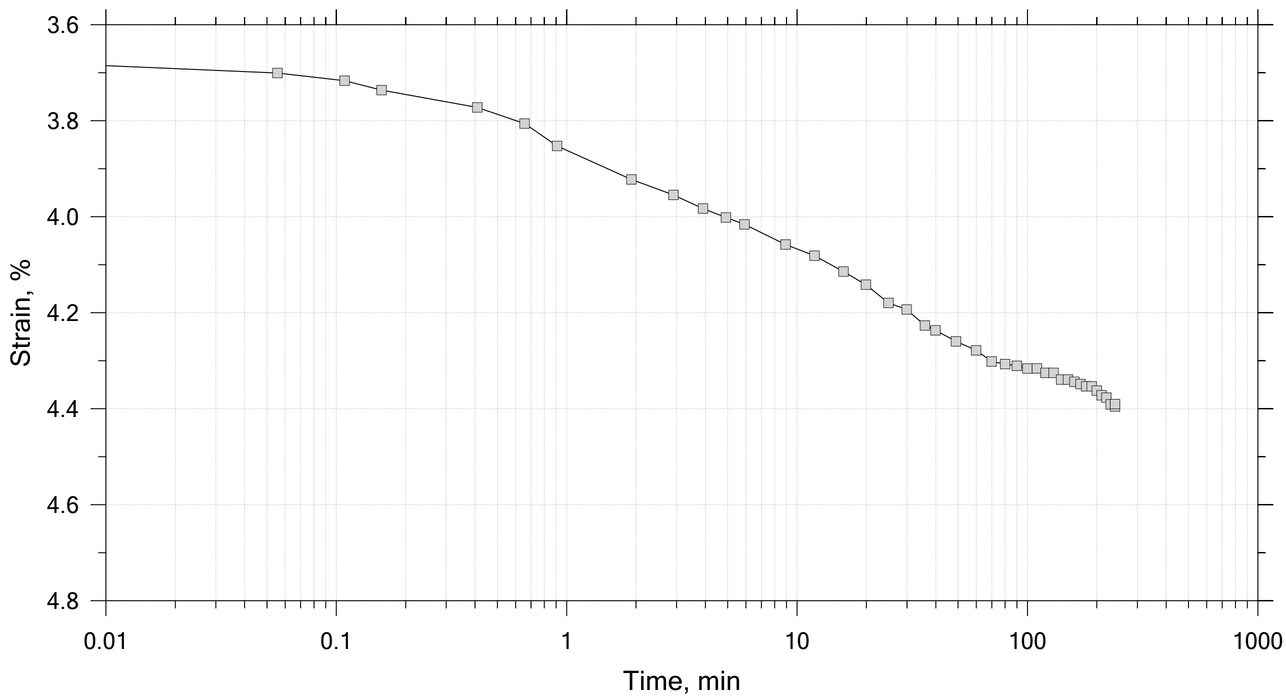
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		

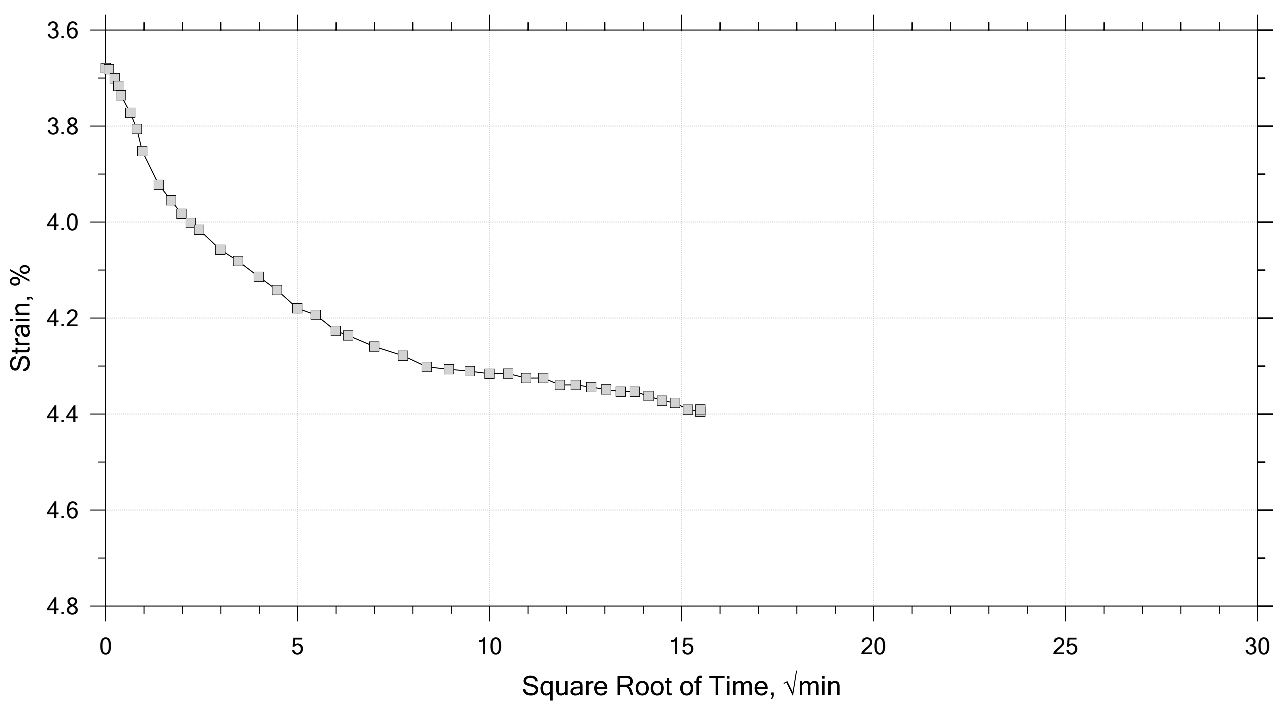
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 17

Constant Load Step

Stress: 0.75 tsf





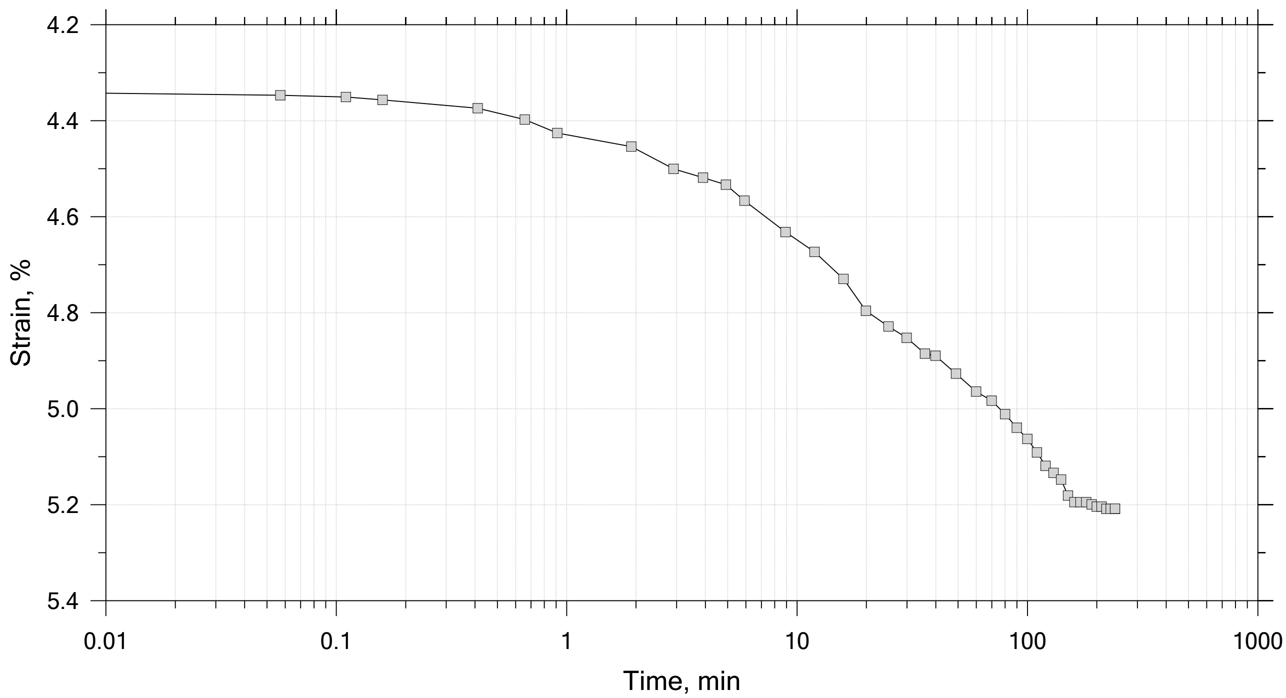
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

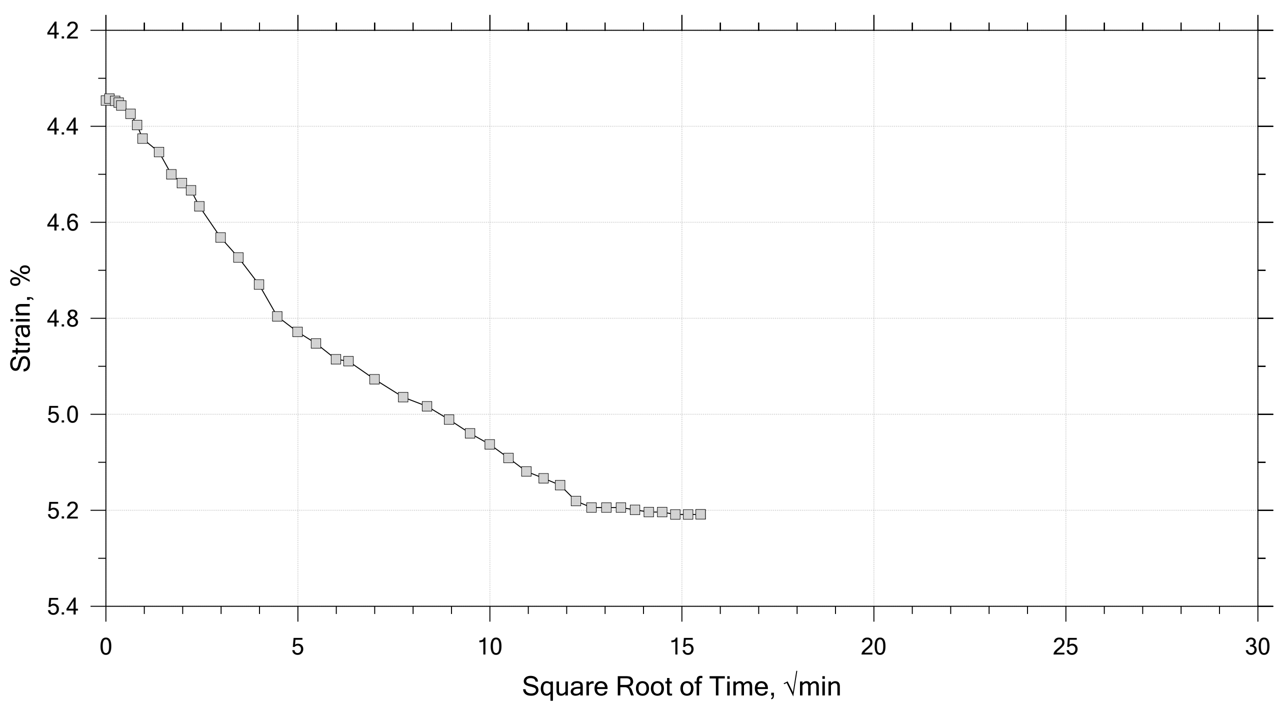
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 17

Constant Load Step

Stress: 1 tsf





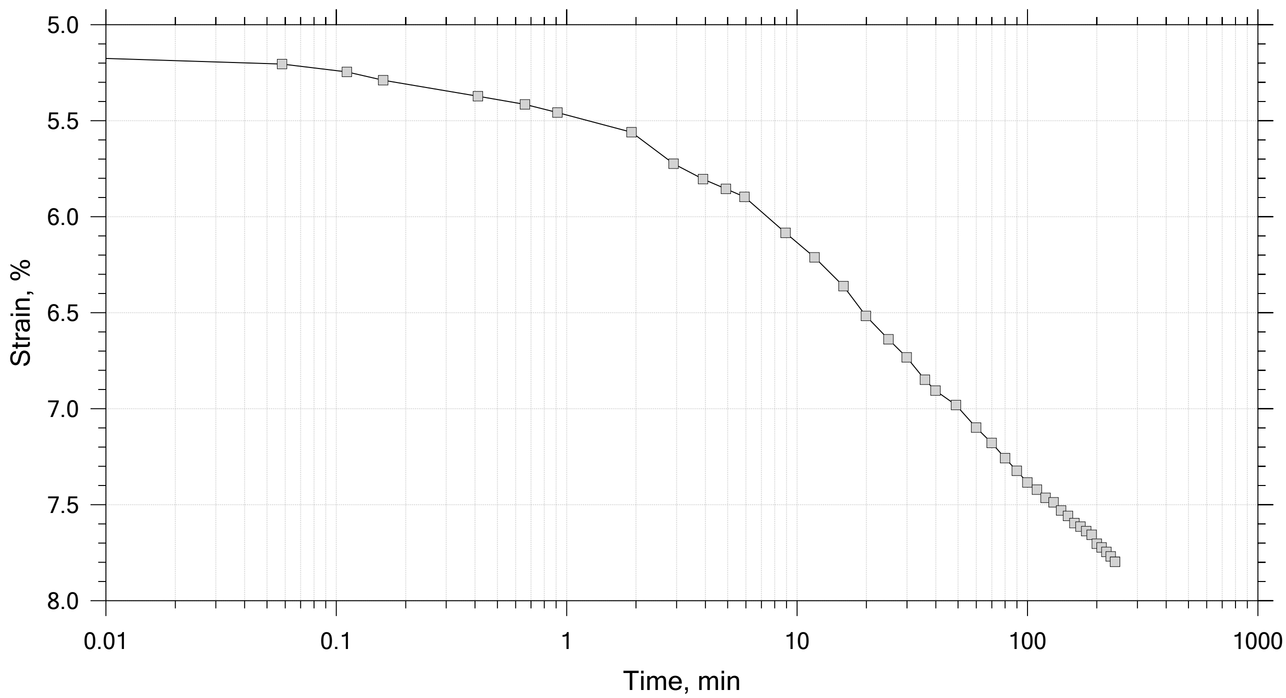
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

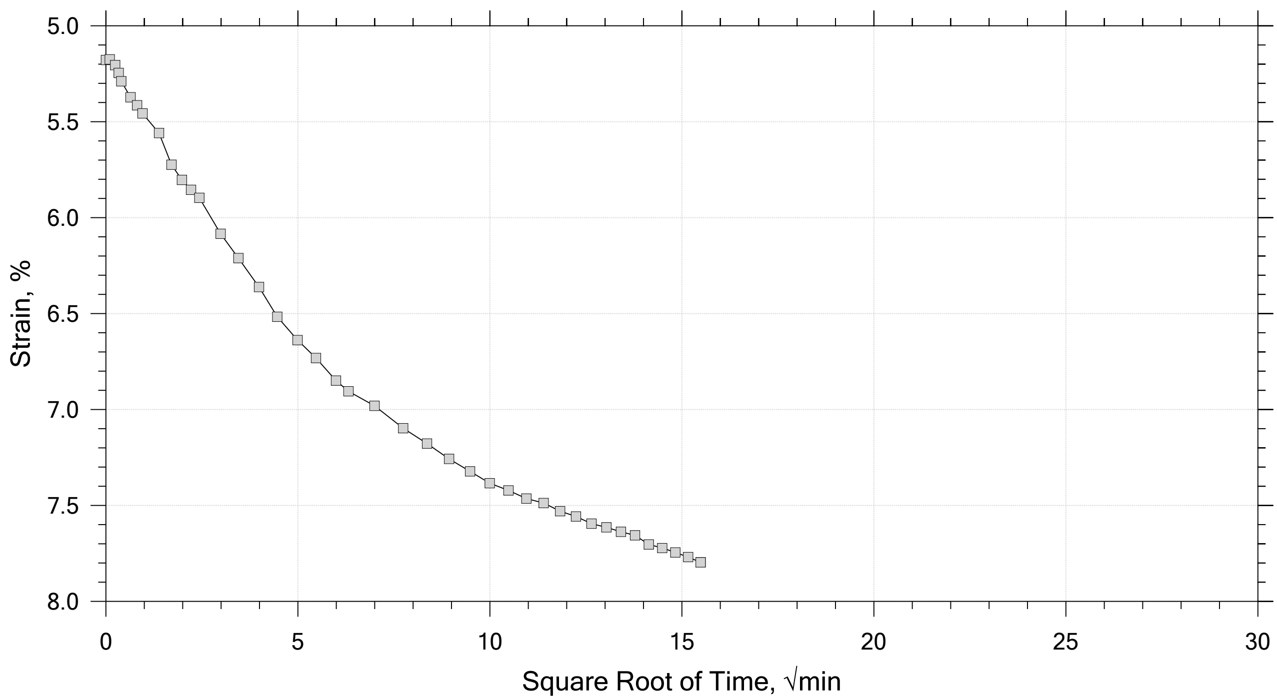
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 17

Constant Load Step

Stress: 1.5 tsf





Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		

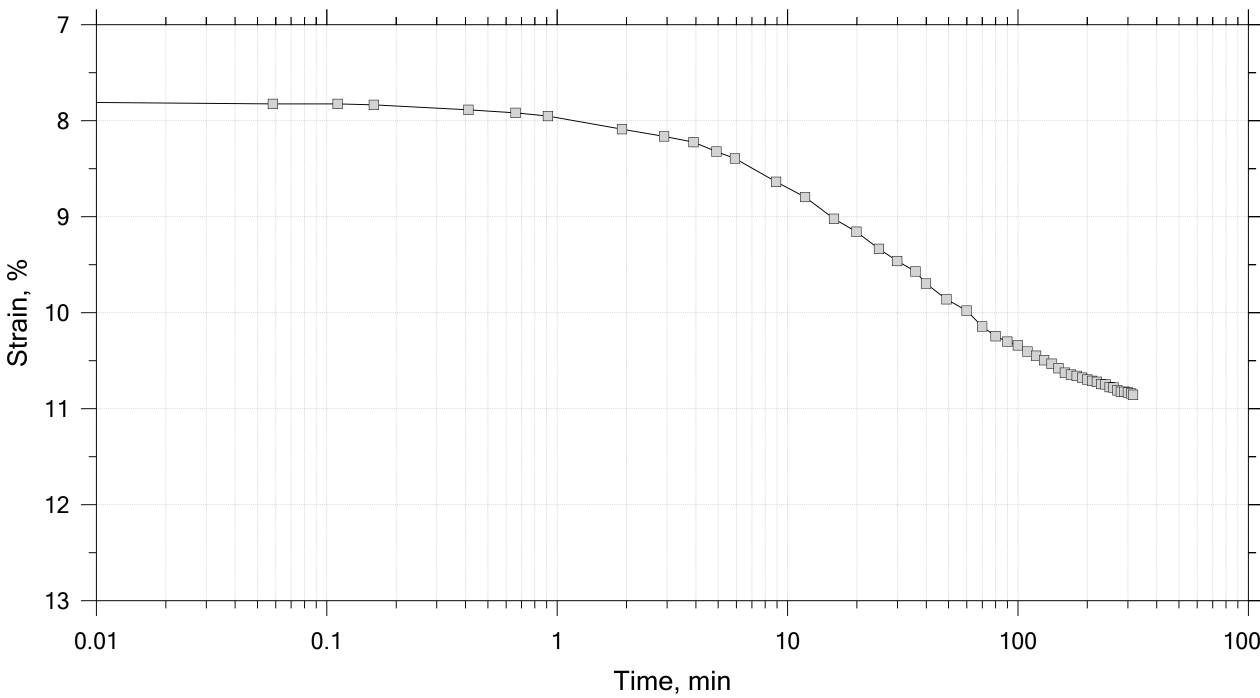
GeoTesting EXPRESS

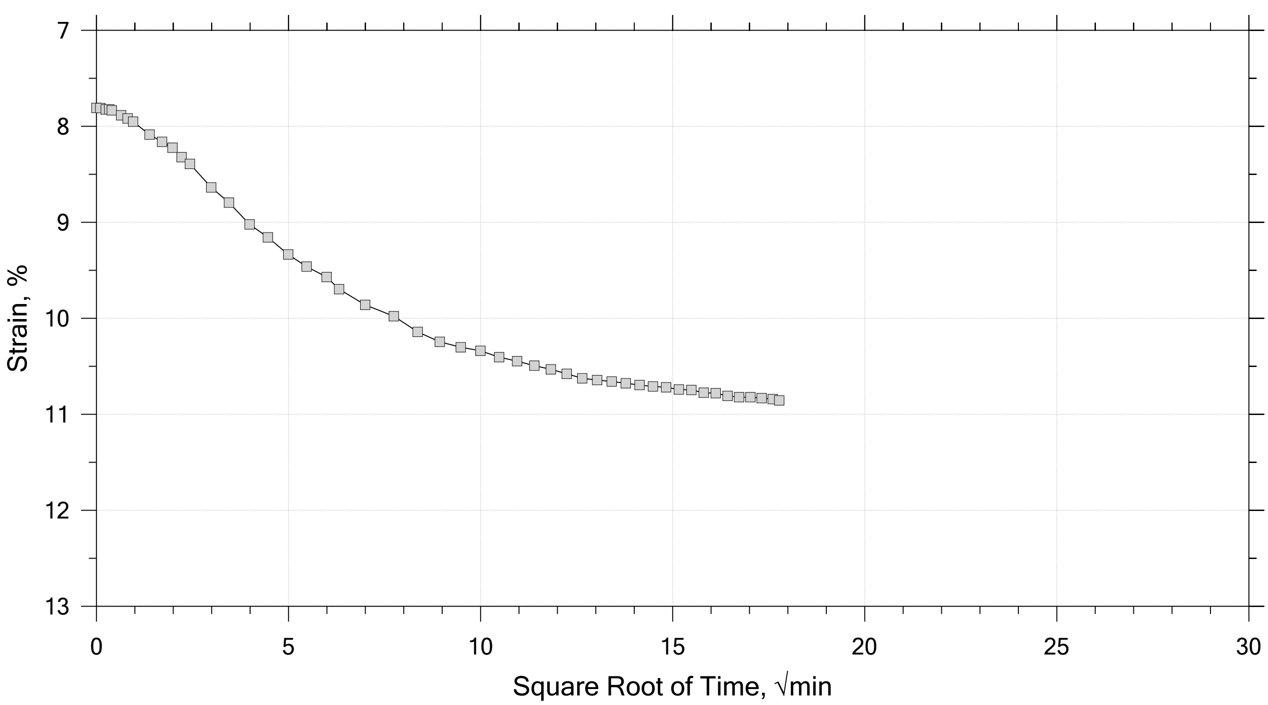
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 17

Constant Load Step

Stress: 2 tsf

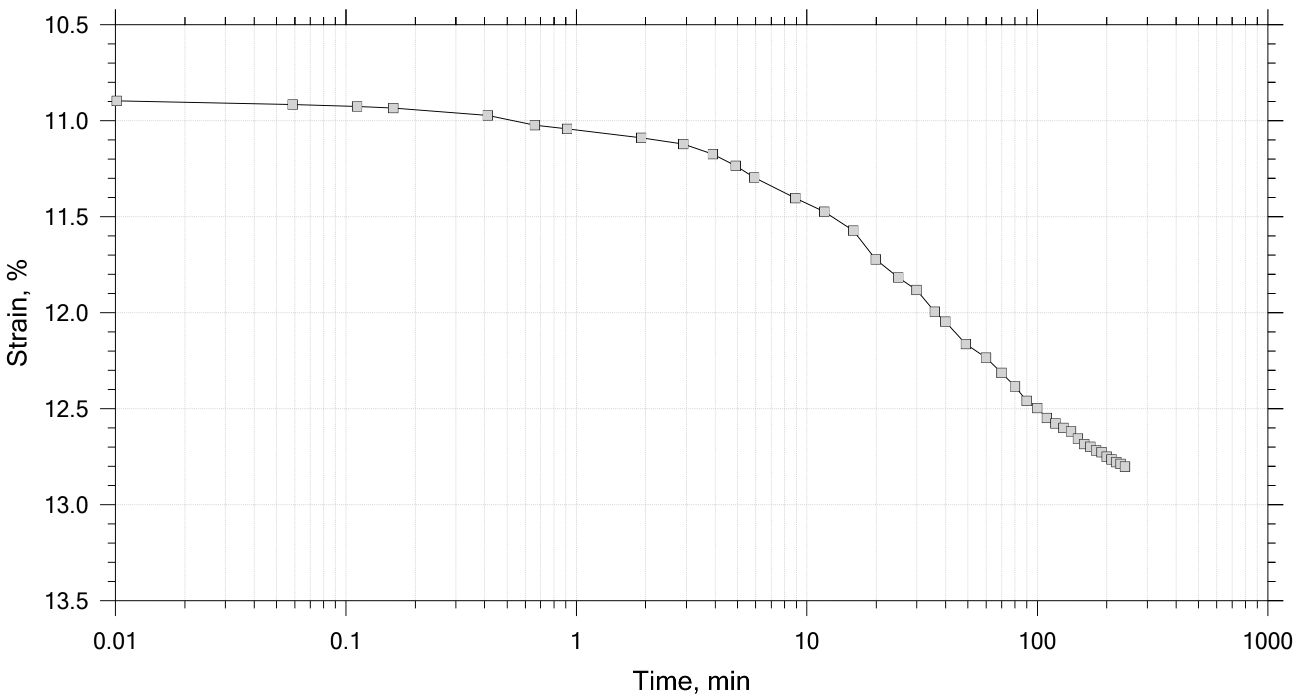


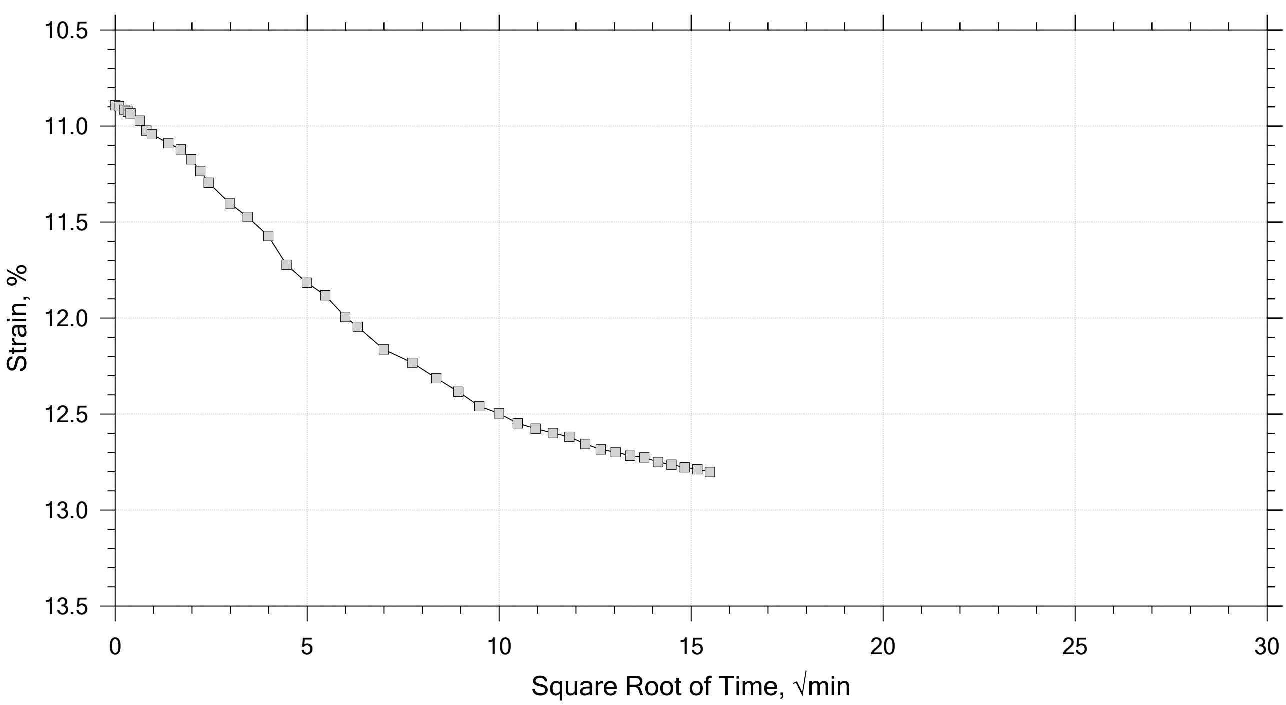


GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 17
Constant Load Step
Stress: 2.5 tsf





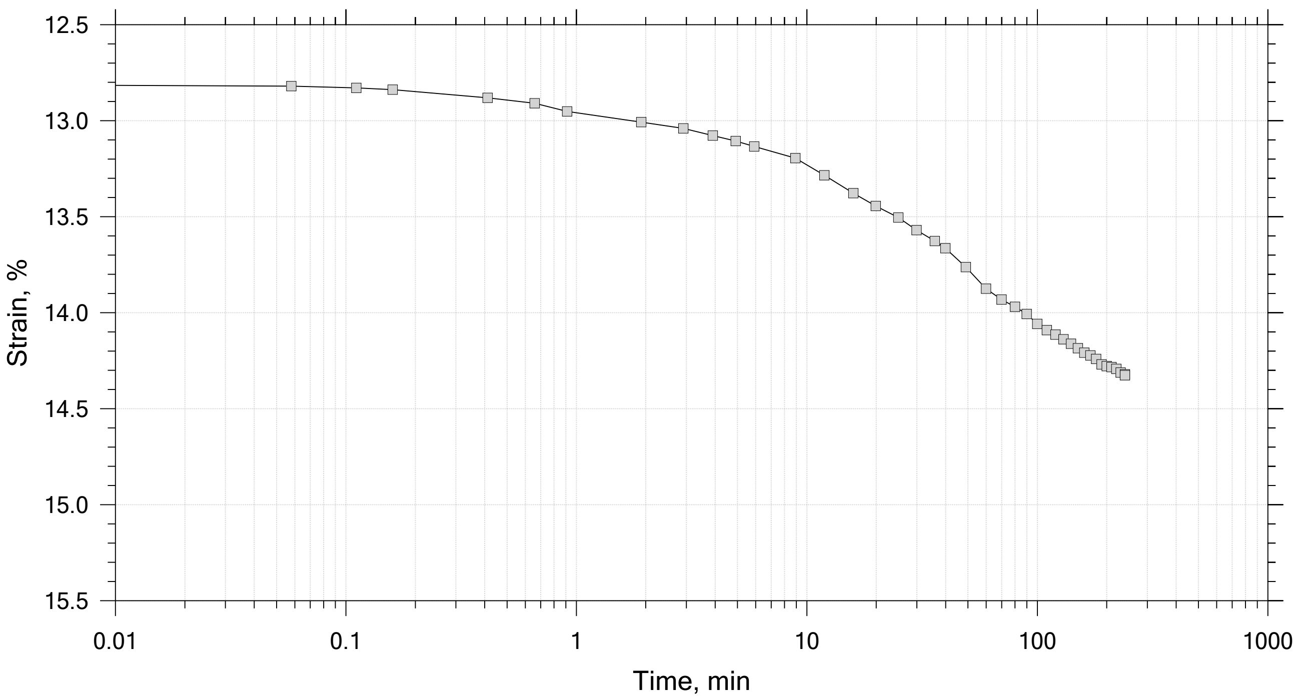
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

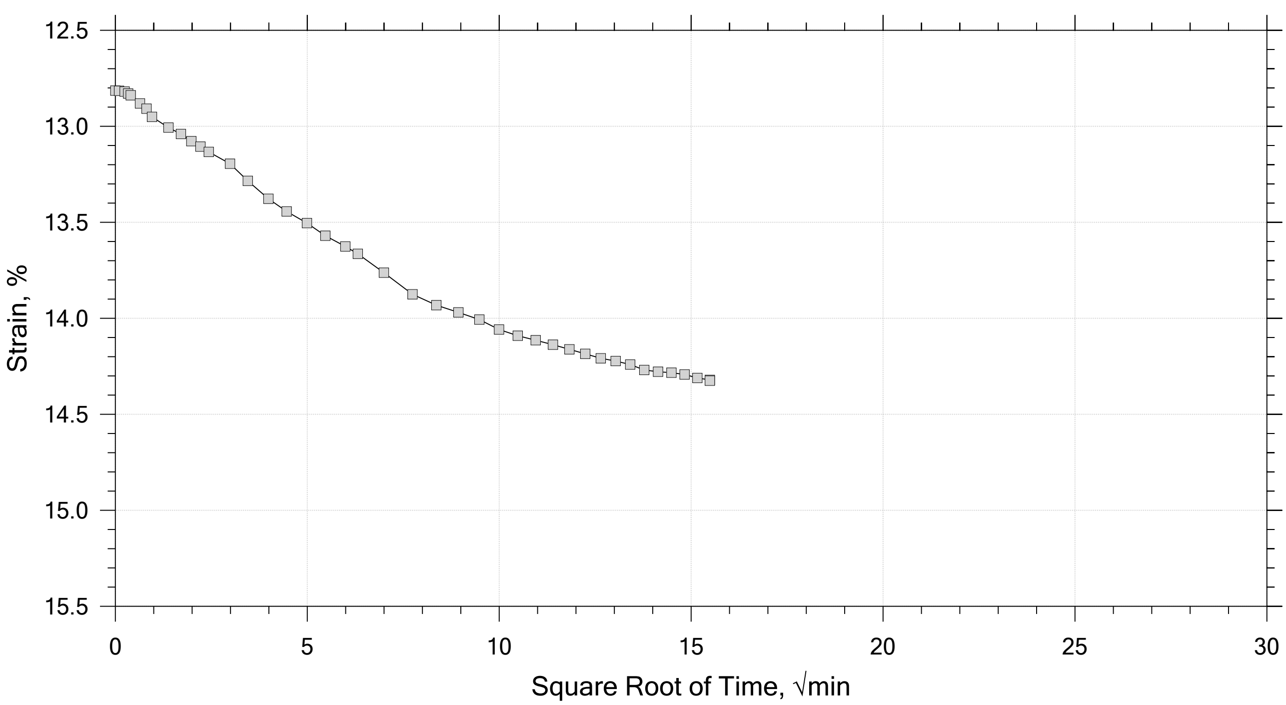
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 17

Constant Load Step

Stress: 3 tsf





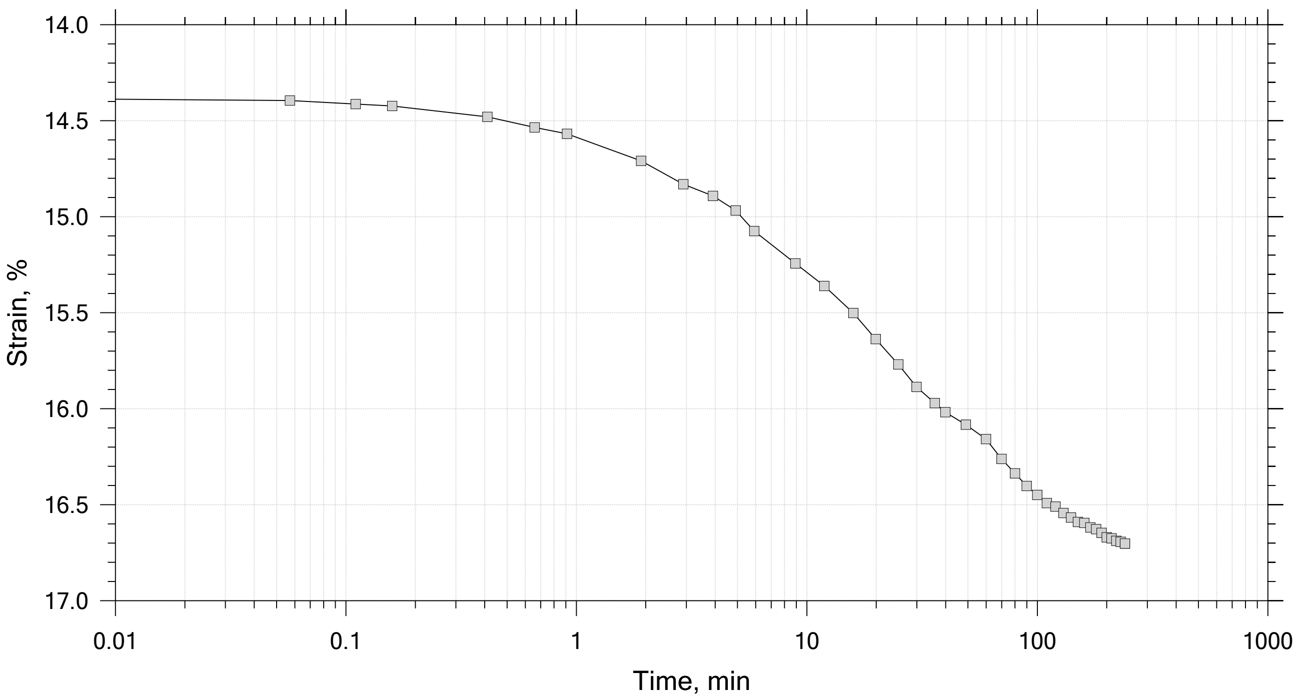
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

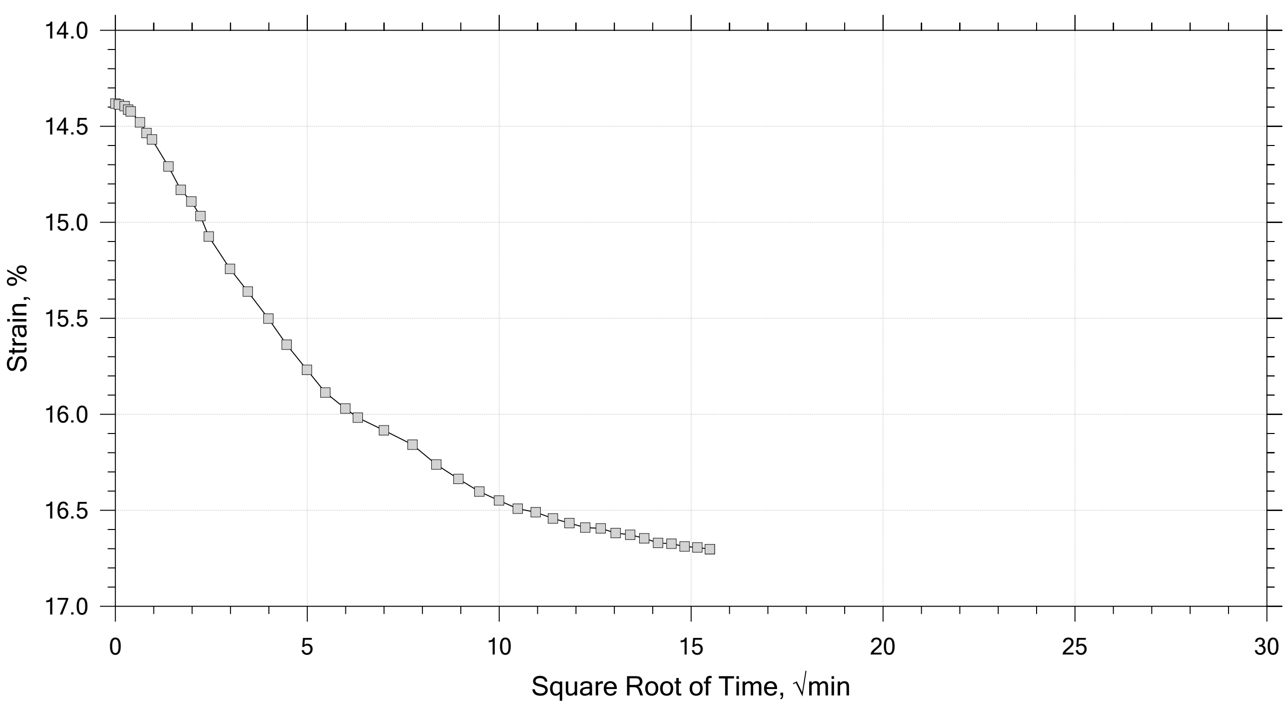
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 17

Constant Load Step

Stress: 4 tsf





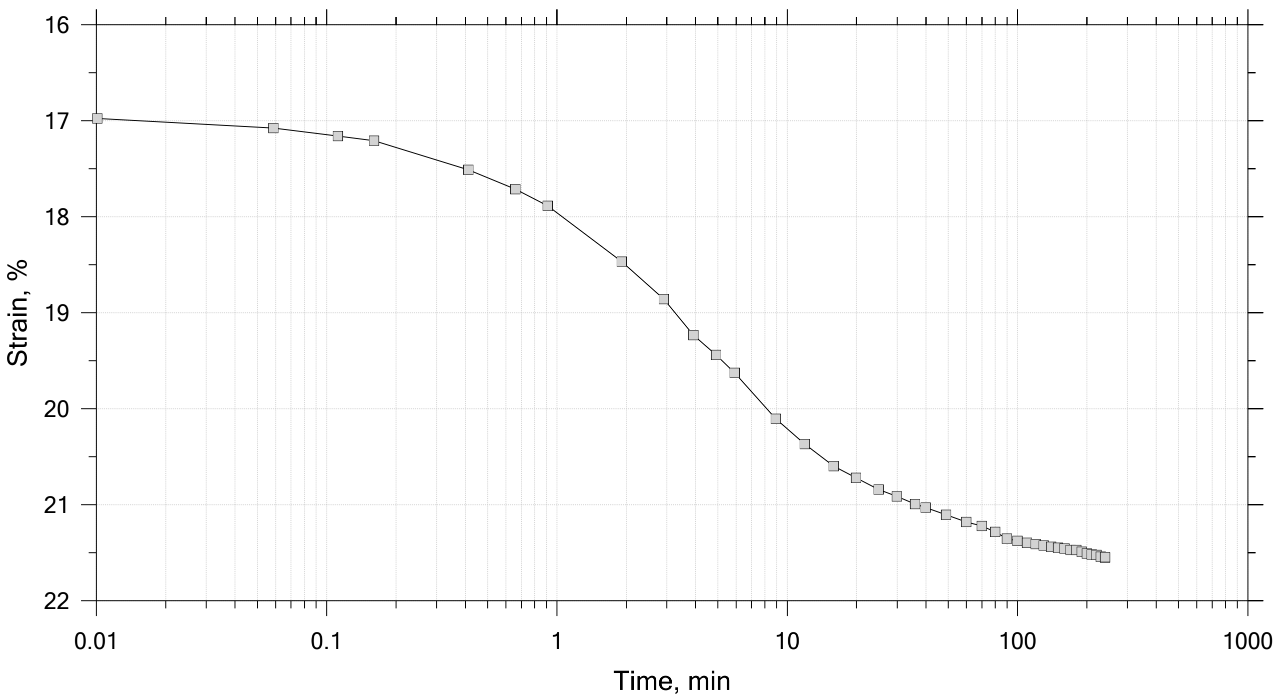
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		

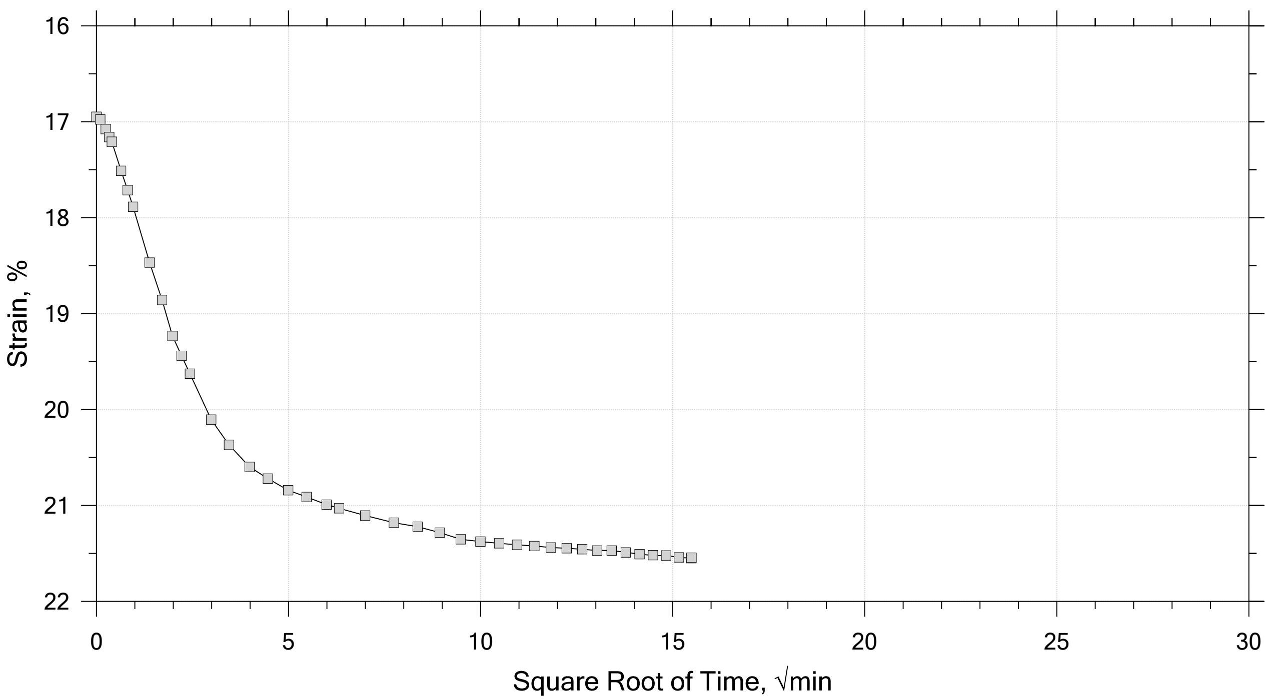
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 17

Constant Load Step

Stress: 8 tsf





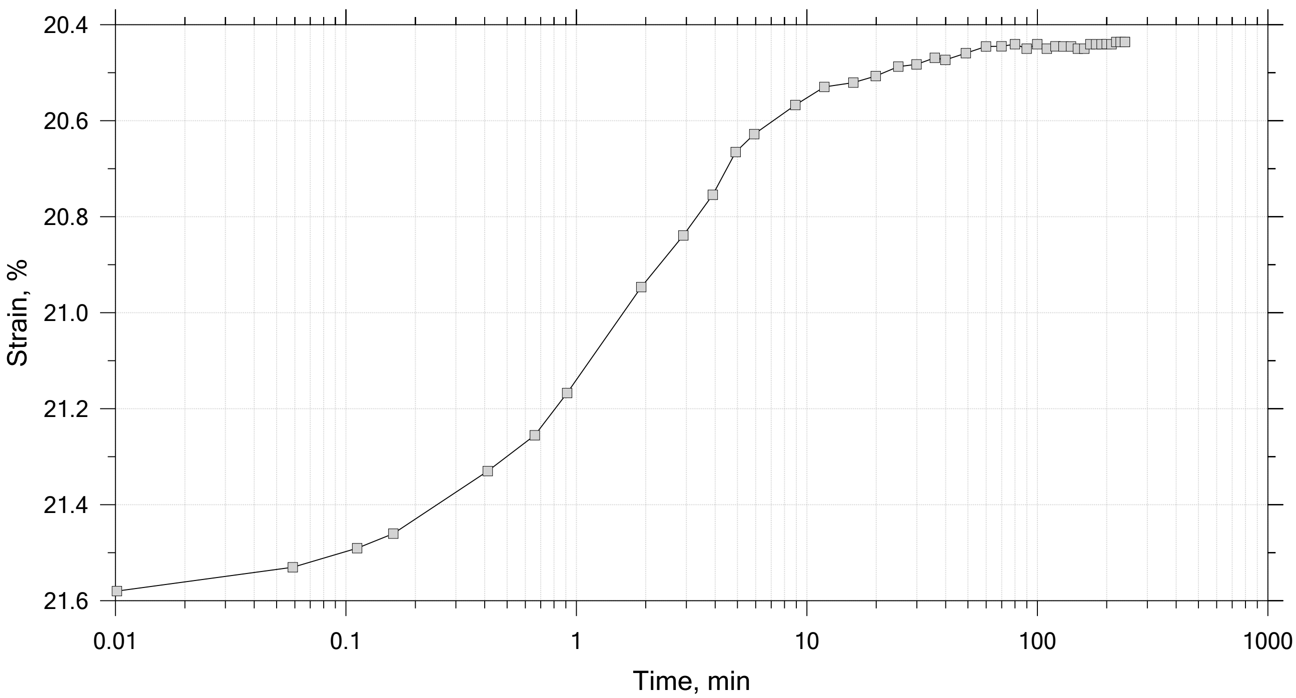
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

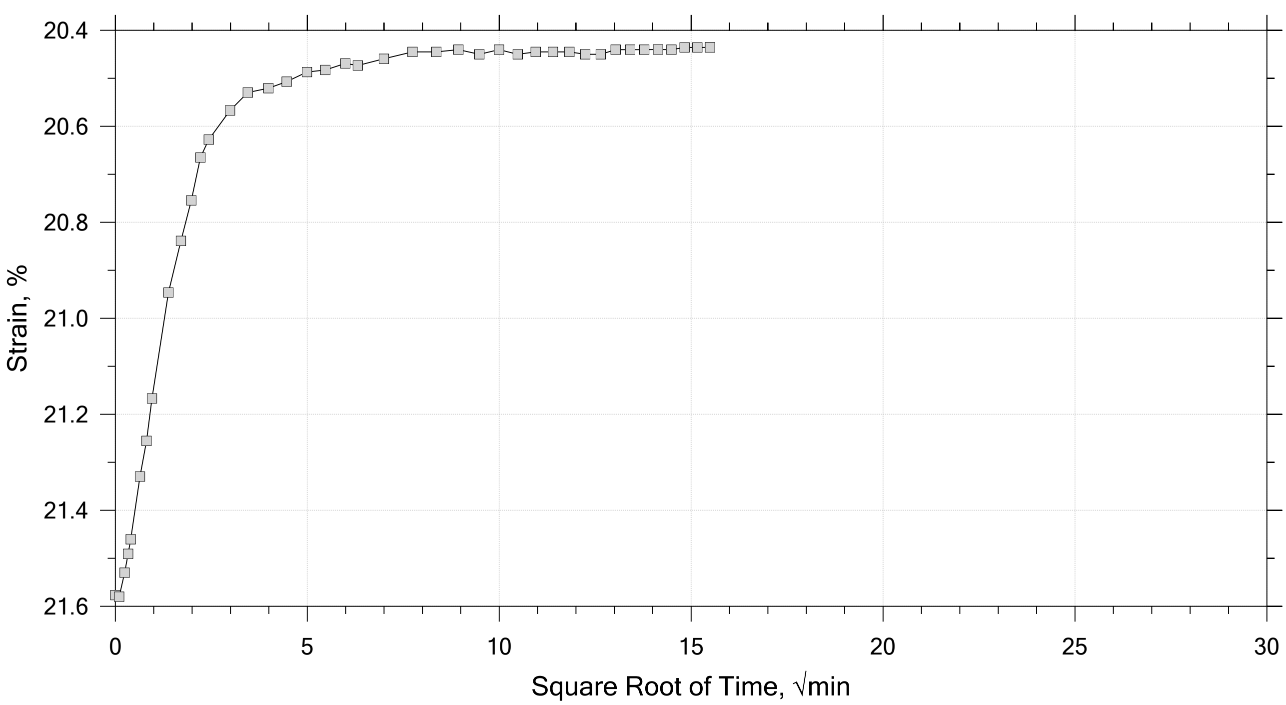
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 17

Constant Load Step

Stress: 1 tsf





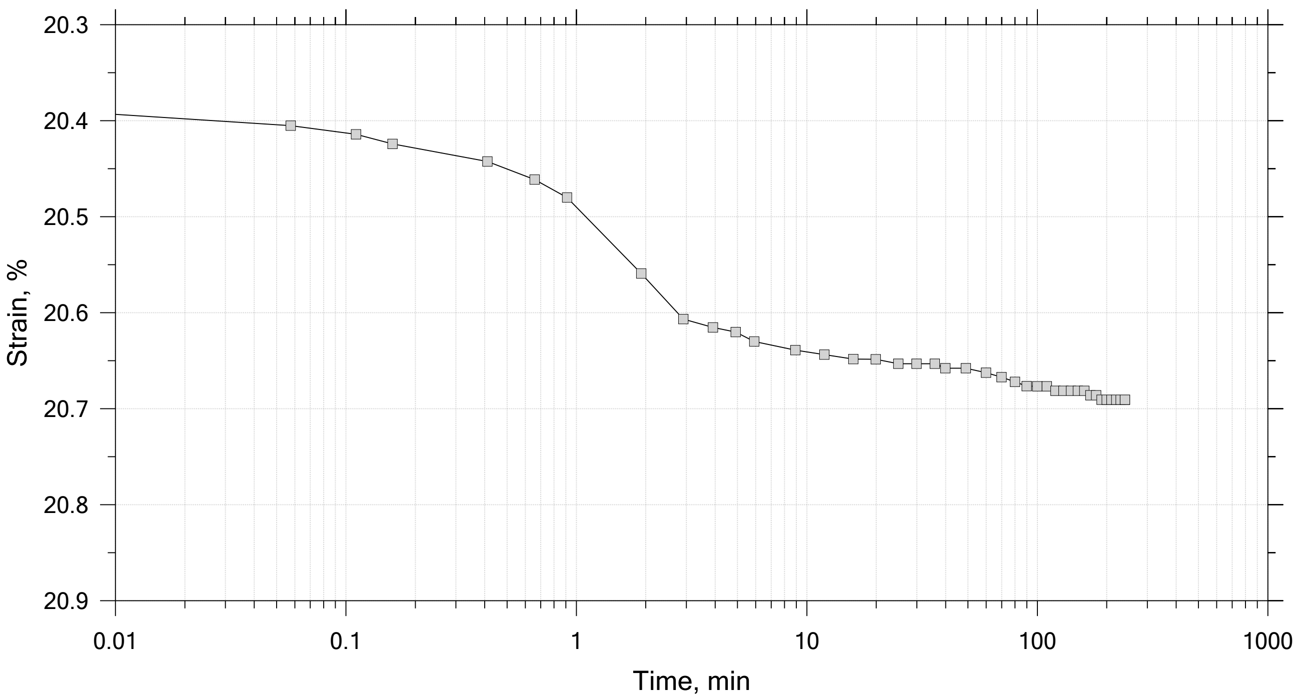
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		

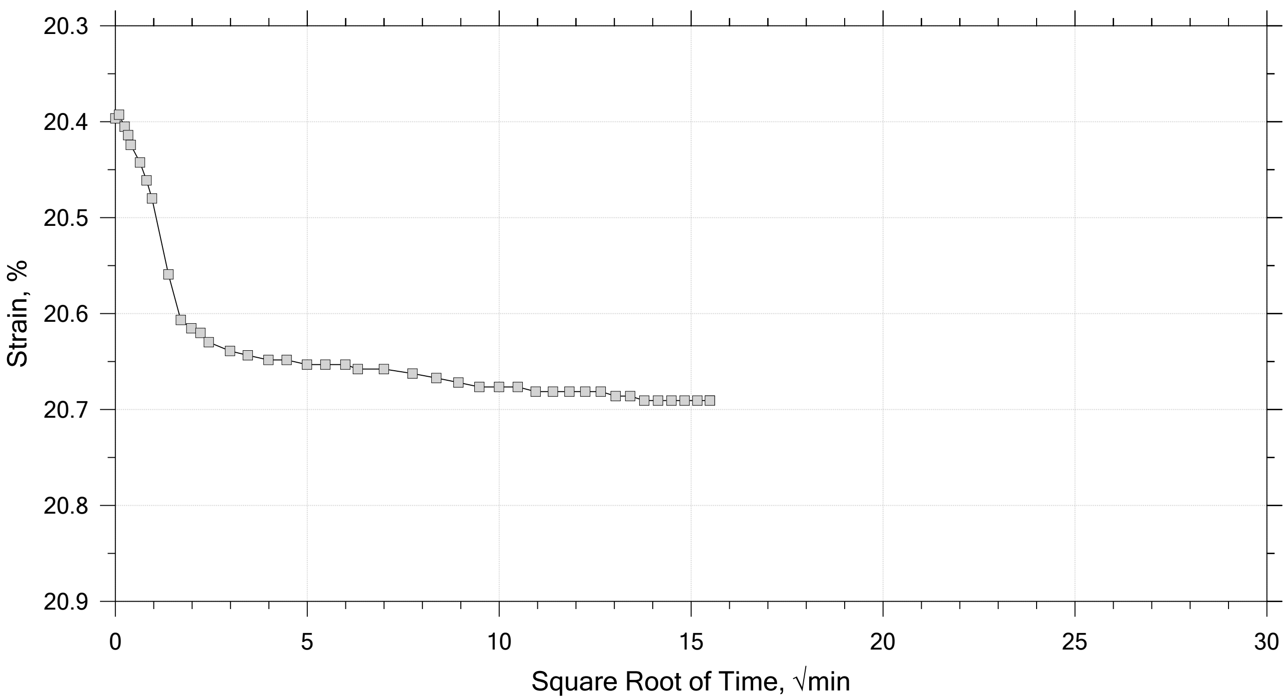
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 17

Constant Load Step

Stress: 2 tsf





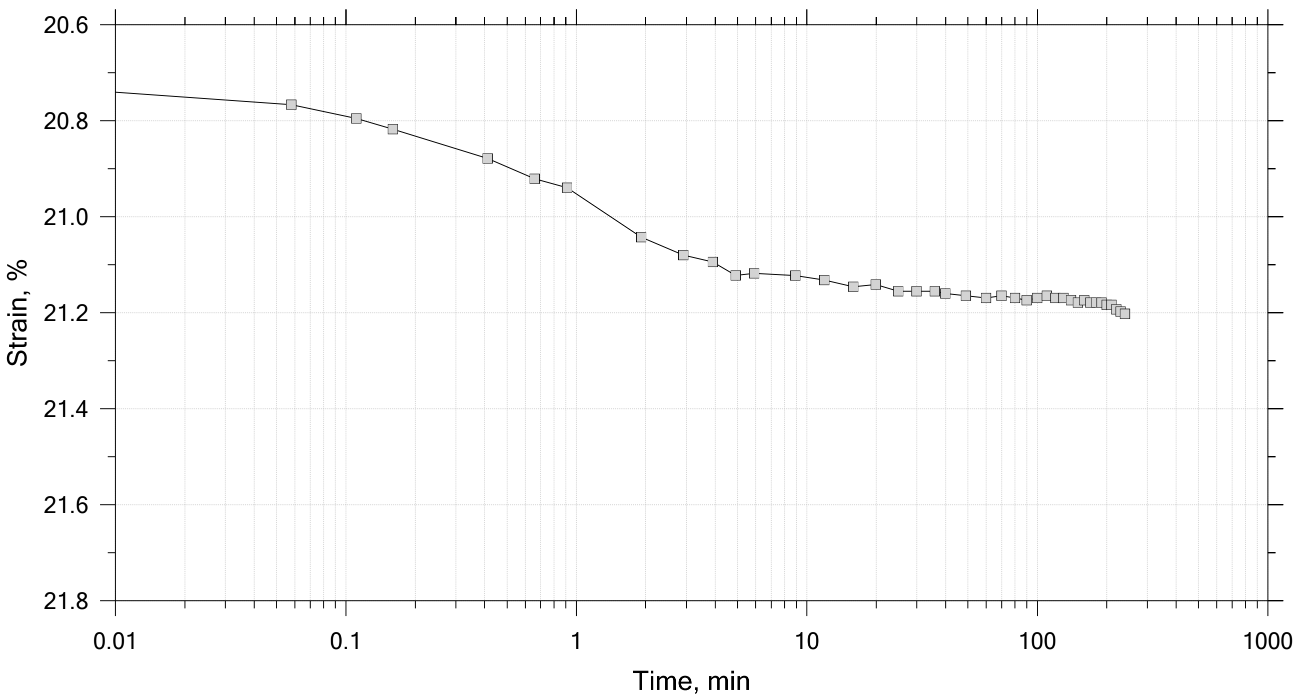
Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		

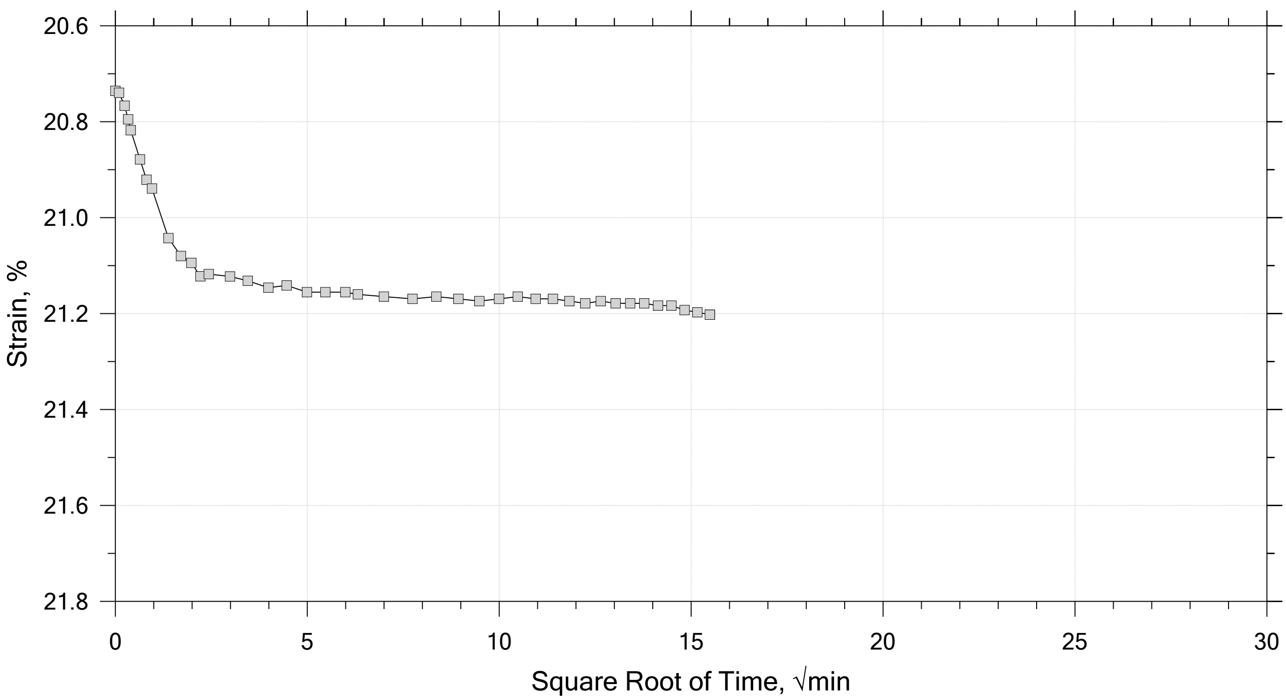
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 17

Constant Load Step

Stress: 4 tsf





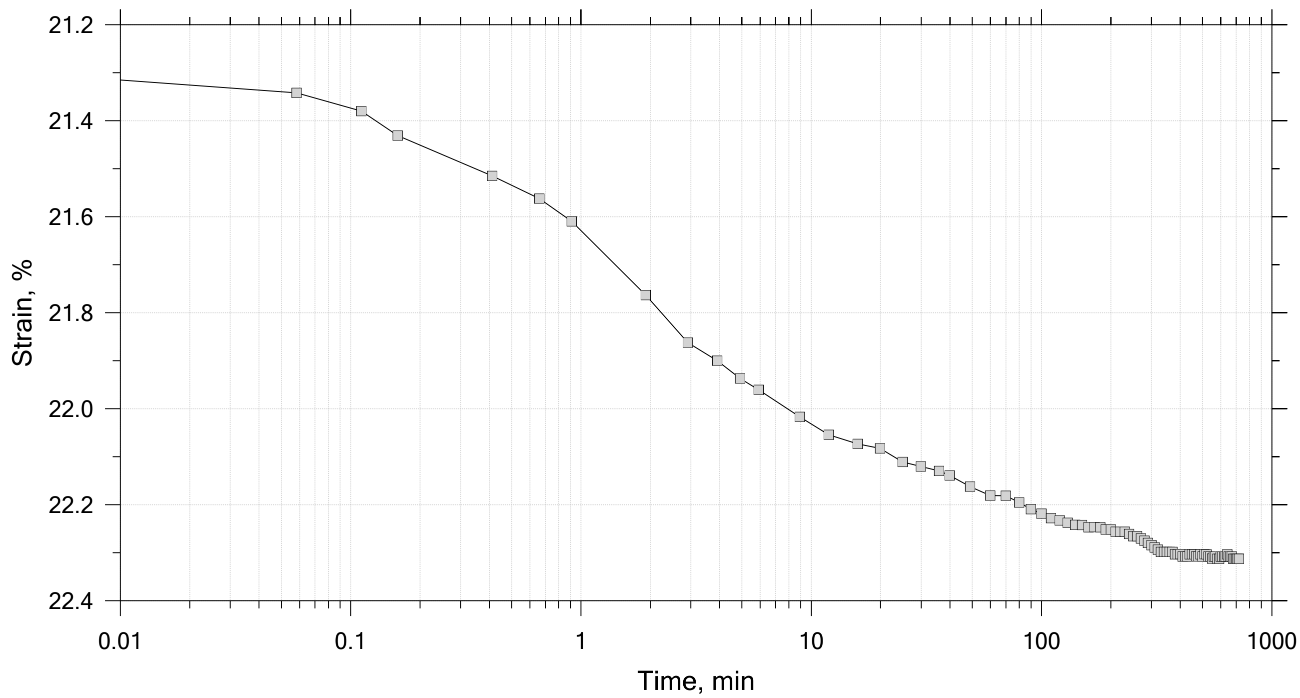
GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

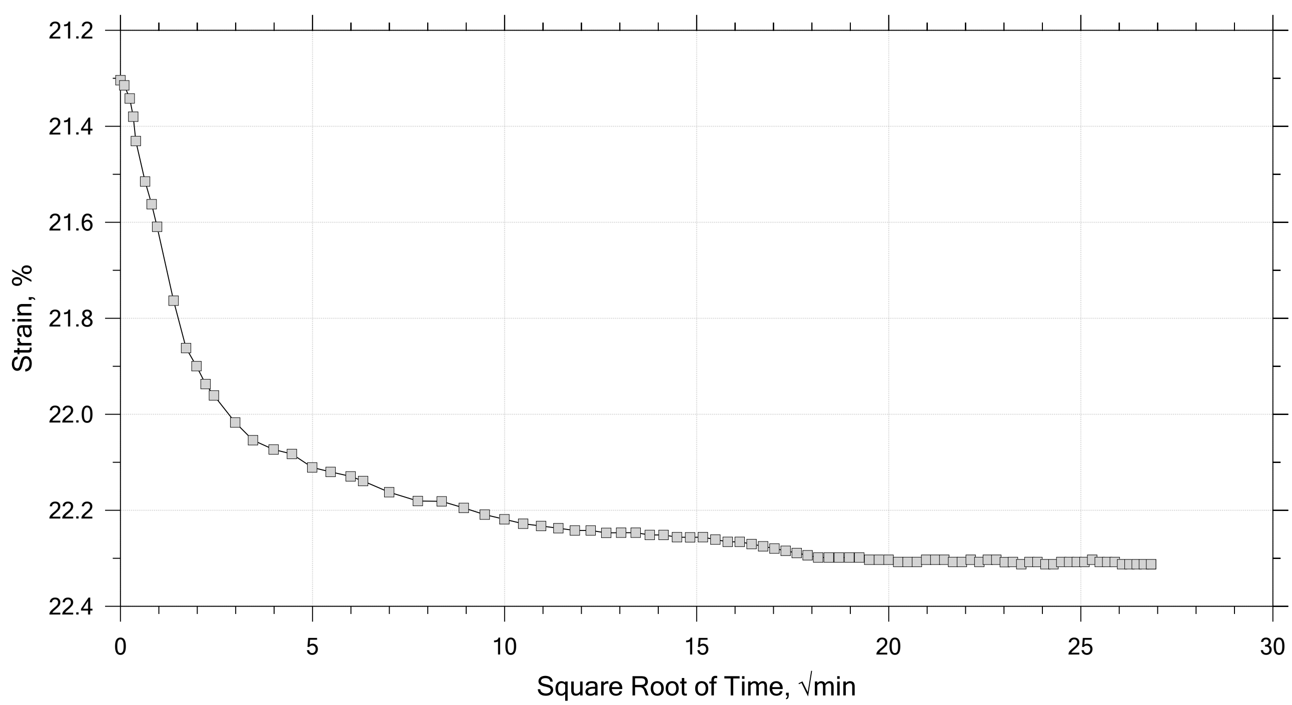
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 17

Constant Load Step

Stress: 7.5 tsf





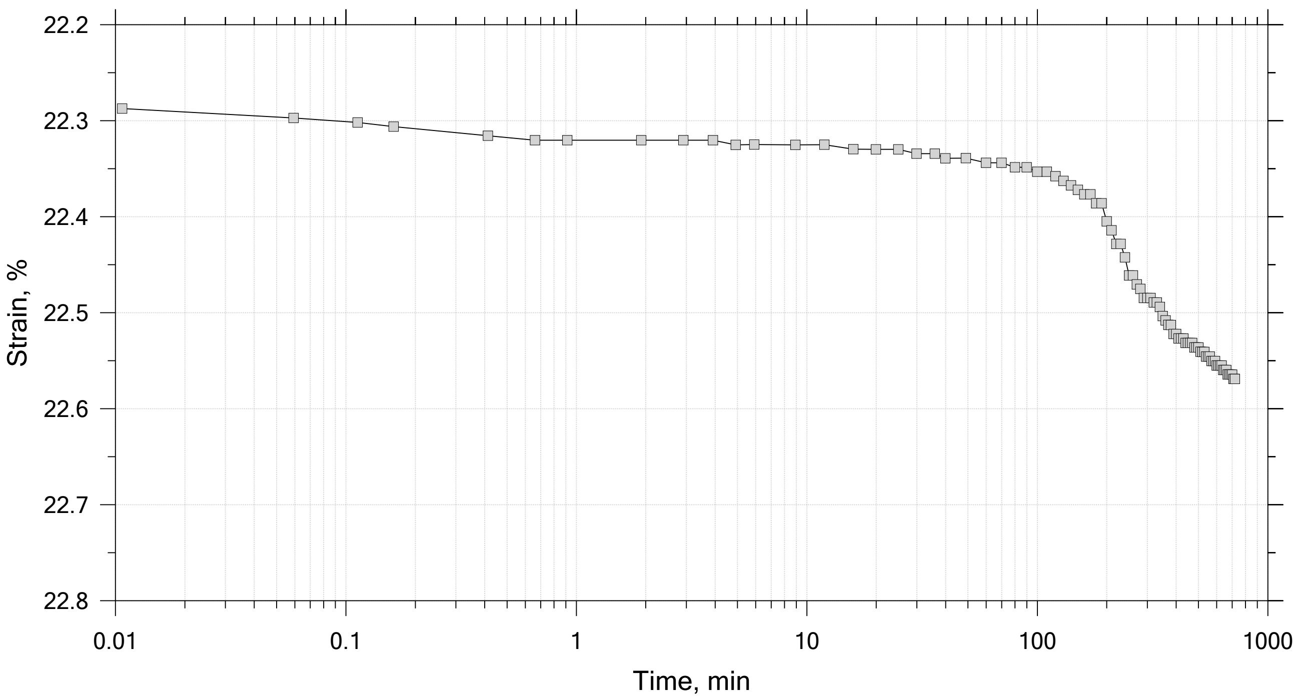
GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

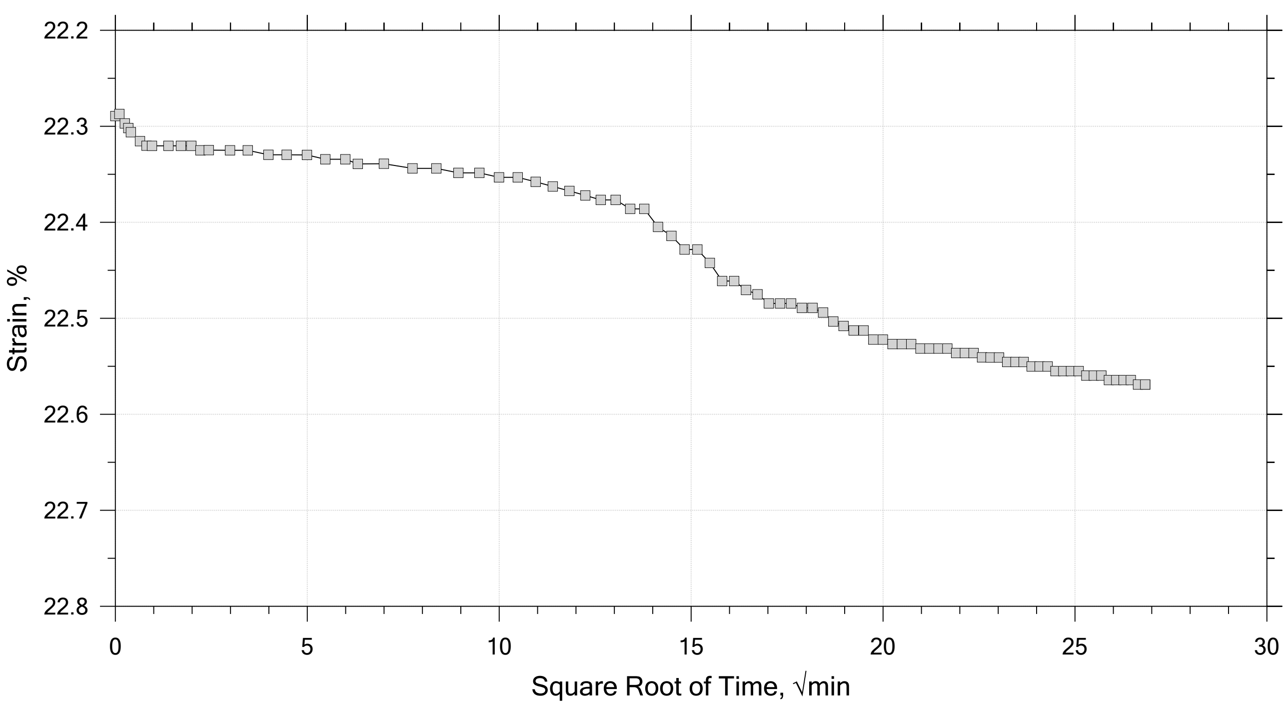
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 17

Constant Load Step

Stress: 8 tsf





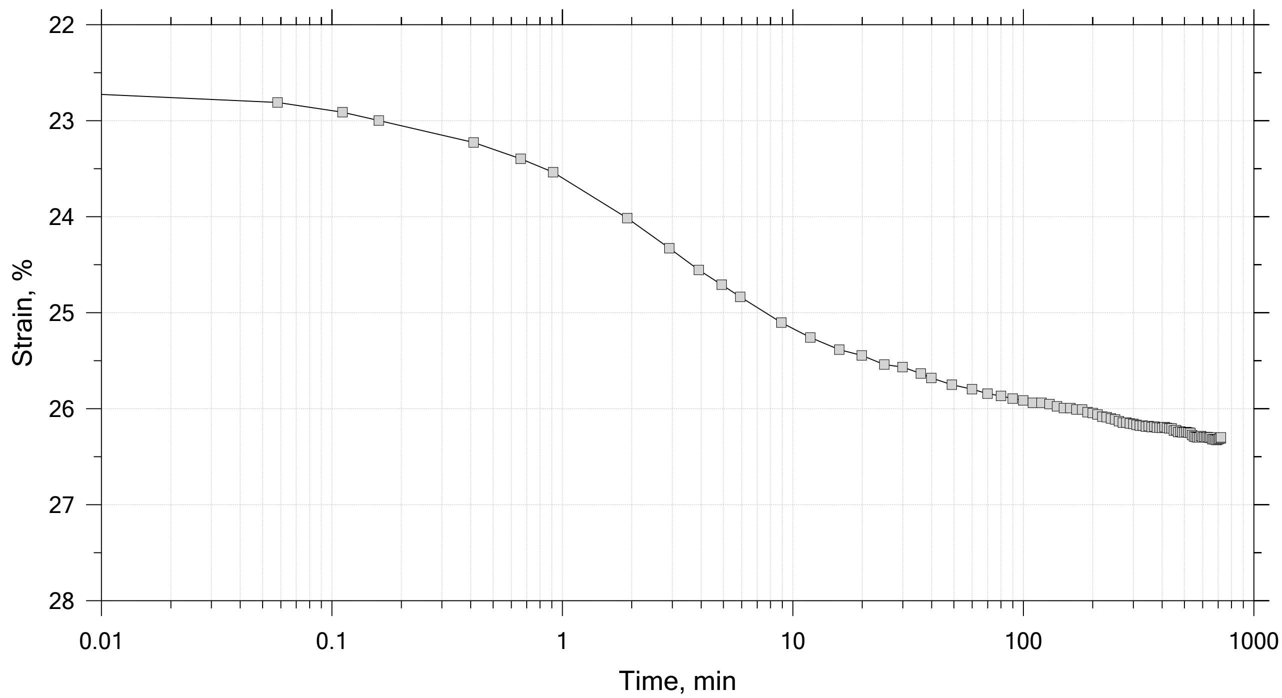
GeoTesting EXPRESS			
	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

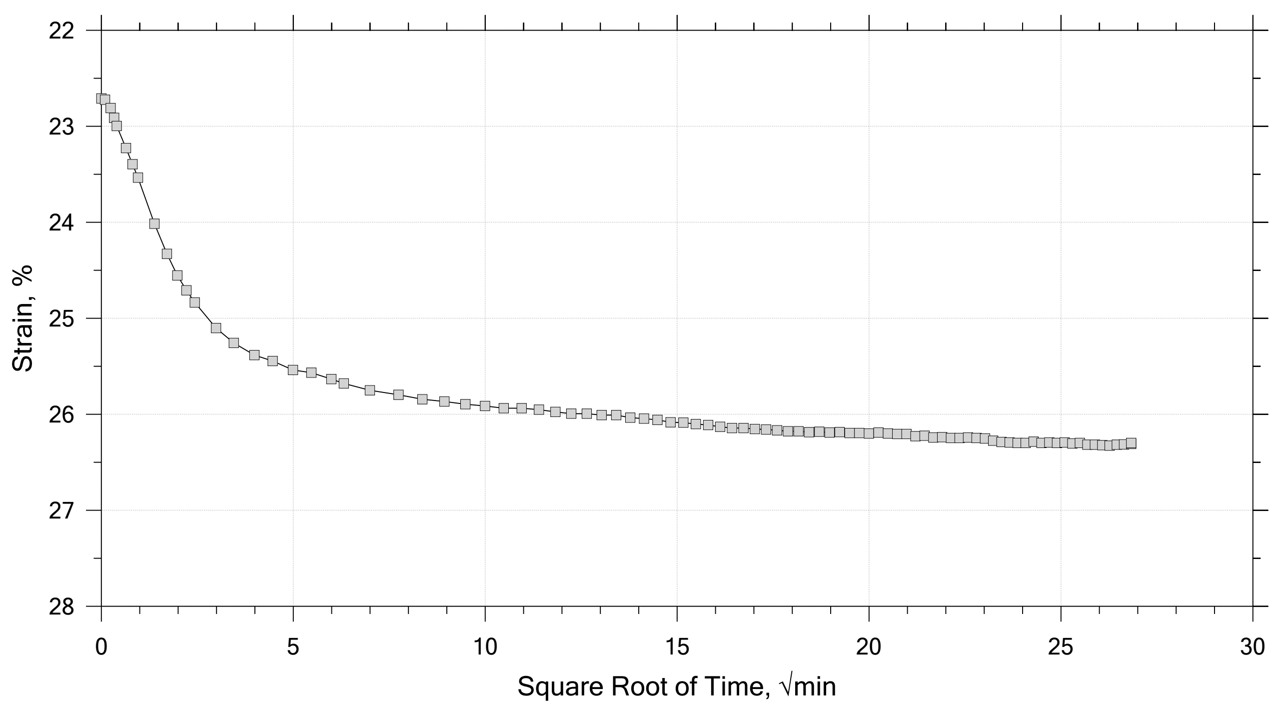
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 17

Constant Load Step

Stress: 16 tsf





GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.76	Liquid Limit: 46
Initial Height: 1.00 in	Initial Void Ratio: 1.13	Plastic Limit: 21
Final Height: 0.79 in	Final Void Ratio: 0.681	Plasticity Index: 25

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	B-2404	RING		C-2245
Mass Container, gm	9.09	111.13	111.13	9.02
Mass Container + Wet Soil, gm	155.31	257.79	241.16	138.98
Mass Container + Dry Soil, gm	110.26	215.41	215.41	113.24
Mass Dry Soil, gm	101.17	104.28	104.28	104.22
Water Content, %	44.53	40.65	24.70	24.70
Void Ratio	---	1.13	0.68	---
Degree of Saturation, %	---	99.38	100.00	---
Dry Unit Weight, pcf	---	80.927	102.44	---

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: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		

One-Dimensional Consolidation by ASTM D2435 - Method B

Log of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/s	Mv 1/tsf	k ft/day	Ca %
1	0.125	0.01526	1.10	1.53	0.000	0.00e+00	1.22e-01	0.00e+00	0.00e+00
2	0.250	0.02480	1.08	2.48	1.601	3.42e-06	7.63e-02	7.04e-04	0.00e+00
3	0.500	0.03639	1.05	3.64	0.000	0.00e+00	4.63e-02	0.00e+00	0.00e+00
4	0.750	0.04390	1.03	4.39	0.000	0.00e+00	3.01e-02	0.00e+00	0.00e+00
5	1.00	0.05208	1.02	5.21	0.000	0.00e+00	3.27e-02	0.00e+00	0.00e+00
6	1.50	0.07797	0.962	7.80	13.783	3.62e-07	5.18e-02	5.05e-05	0.00e+00
7	2.00	0.1086	0.897	10.9	25.772	1.82e-07	6.12e-02	3.00e-05	0.00e+00
8	2.50	0.1280	0.856	12.8	27.501	1.61e-07	3.89e-02	1.69e-05	0.00e+00
9	3.00	0.1433	0.823	14.3	0.000	0.00e+00	3.05e-02	0.00e+00	0.00e+00
10	4.00	0.1670	0.773	16.7	12.228	3.33e-07	2.38e-02	2.13e-05	0.00e+00
11	8.00	0.2155	0.670	21.5	2.937	1.27e-06	1.21e-02	4.15e-05	0.00e+00
12	1.00	0.2044	0.693	20.4	0.000	0.00e+00	1.59e-03	0.00e+00	0.00e+00
13	2.00	0.2069	0.688	20.7	1.681	2.14e-06	2.55e-03	1.47e-05	0.00e+00
14	4.00	0.2120	0.677	21.2	1.243	2.87e-06	2.56e-03	1.98e-05	0.00e+00
15	7.50	0.2231	0.653	22.3	0.000	0.00e+00	3.17e-03	0.00e+00	0.00e+00
16	8.00	0.2257	0.648	22.6	206.926	1.66e-08	5.13e-03	2.29e-07	0.00e+00
17	16.0	0.2630	0.569	26.3	3.292	9.89e-07	4.66e-03	1.24e-05	0.00e+00

Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, dark gray clay		
Remarks: System KK		
Displacement at End of Increment		

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 ft²/s	Mv 1/tsf	k ft/day
1	0.125	0.01526	1.10	1.53	15.225	1.59e-06	1.22e-01	5.23e-04
2	0.250	0.02480	1.08	2.48	5.871	4.01e-06	7.63e-02	8.26e-04
3	0.500	0.03639	1.05	3.64	10.608	2.17e-06	4.63e-02	2.72e-04
4	0.750	0.04390	1.03	4.39	25.277	8.94e-07	3.01e-02	7.25e-05
5	1.00	0.05208	1.02	5.21	54.862	4.05e-07	3.27e-02	3.58e-05
6	1.50	0.07797	0.962	7.80	50.652	4.23e-07	5.18e-02	5.91e-05
7	2.00	0.1086	0.897	10.9	94.915	2.13e-07	6.12e-02	3.51e-05
8	2.50	0.1280	0.856	12.8	117.084	1.63e-07	3.89e-02	1.71e-05
9	3.00	0.1433	0.823	14.3	128.607	1.43e-07	3.05e-02	1.17e-05
10	4.00	0.1670	0.773	16.7	49.874	3.51e-07	2.38e-02	2.25e-05
11	8.00	0.2155	0.670	21.5	16.253	9.87e-07	1.21e-02	3.22e-05
12	1.00	0.2044	0.693	20.4	7.629	2.01e-06	1.59e-03	8.59e-06
13	2.00	0.2069	0.688	20.7	8.412	1.84e-06	2.55e-03	1.27e-05
14	4.00	0.2120	0.677	21.2	4.884	3.14e-06	2.56e-03	2.17e-05
15	7.50	0.2231	0.653	22.3	16.416	9.15e-07	3.17e-03	7.83e-06
16	8.00	0.2257	0.648	22.6	633.049	2.33e-08	5.13e-03	3.23e-07
17	16.0	0.2630	0.569	26.3	18.163	7.71e-07	4.66e-03	9.70e-06

GeoTesting EXPRESS	Project: MEDOT Replace Johnson Rd Bridge	Location: Falmouth, ME	Project No.: GTX-310146
	Boring No.: BB-FJR-105	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 06/28/19	Depth: 28-30 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System KK		
	Displacement at End of Increment		

WARRANTY and LIABILITY

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

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

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

Commonly Used Symbols

A	pore pressure parameter for $\Delta\sigma_1 - \Delta\sigma_3$
B	pore pressure parameter for $\Delta\sigma_3$
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

Soil Metrics LLC Geotechnical Engineering
12 Farms Edge Road
Cape Elizabeth, Maine 04107
Ph. 207. 767.2192
Cell. 207.415.5835

MEMORANDUM

TO: **ANDREW BLAISDELL**

FROM: **STEVE RABASCA**

DATE: **JANUARY 7, 2020**

SUBJECT: **JOHNSON ROAD LAB TEST RESULTS – DSS TESTING**

Attached please find the Final Direct Simple Shear lab test results for the Johnson Road Bridge Project. The following test results are included:

Direct Simple Shear Tests:

DSS-105: BB-FJR-104 2U
DSS-106: BB-FJR-103 1U
DSS-107: BB-FJR-104 1U
DSS-108: BB-FJR-102A 1U

Please call me if you have any questions.

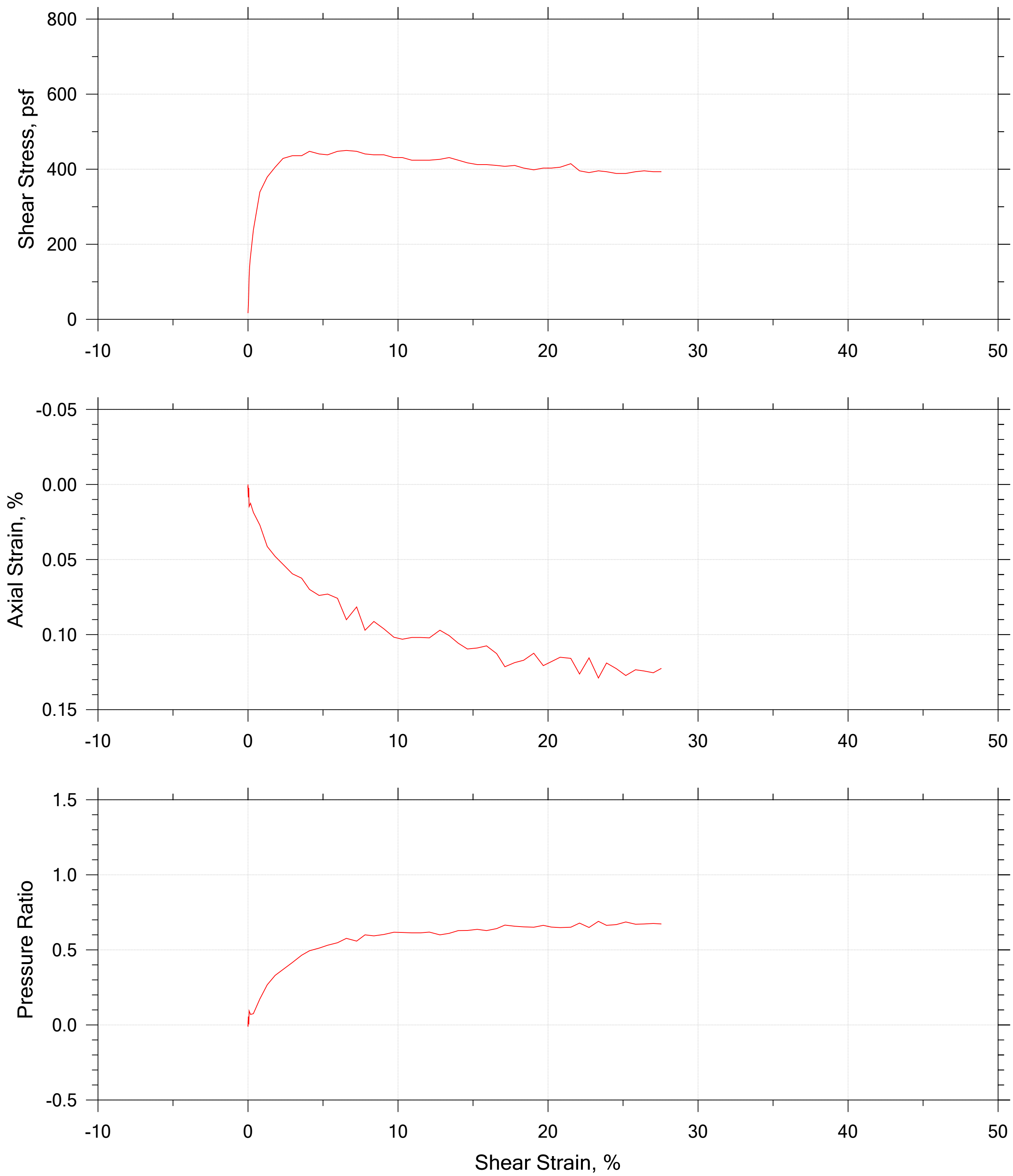
Regards - Steve

DIRECT SIMPLE SHEAR

DSS 105

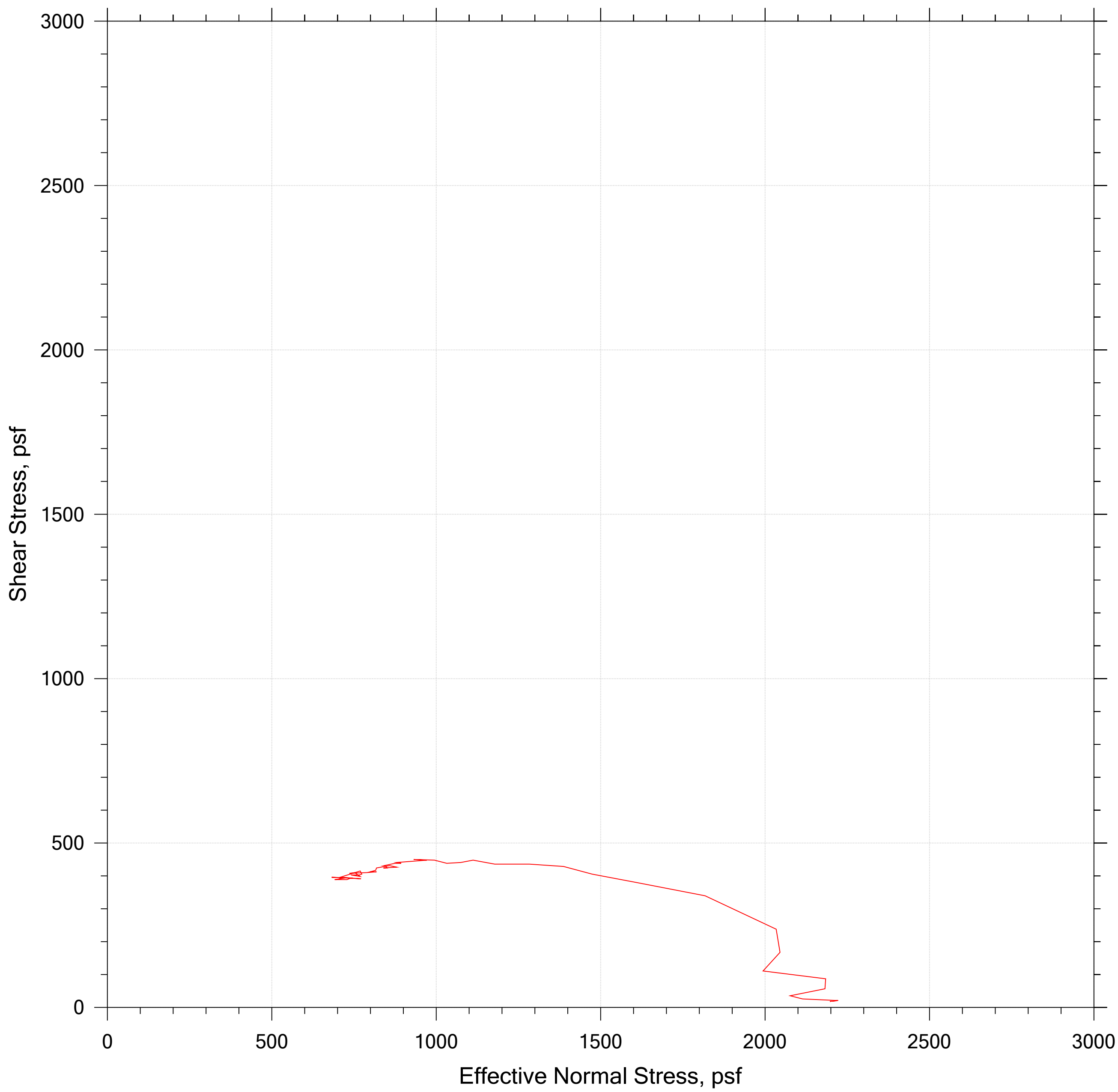
BB-FJR-104 2U

Direct Simple Shear Test by ASTM D6528



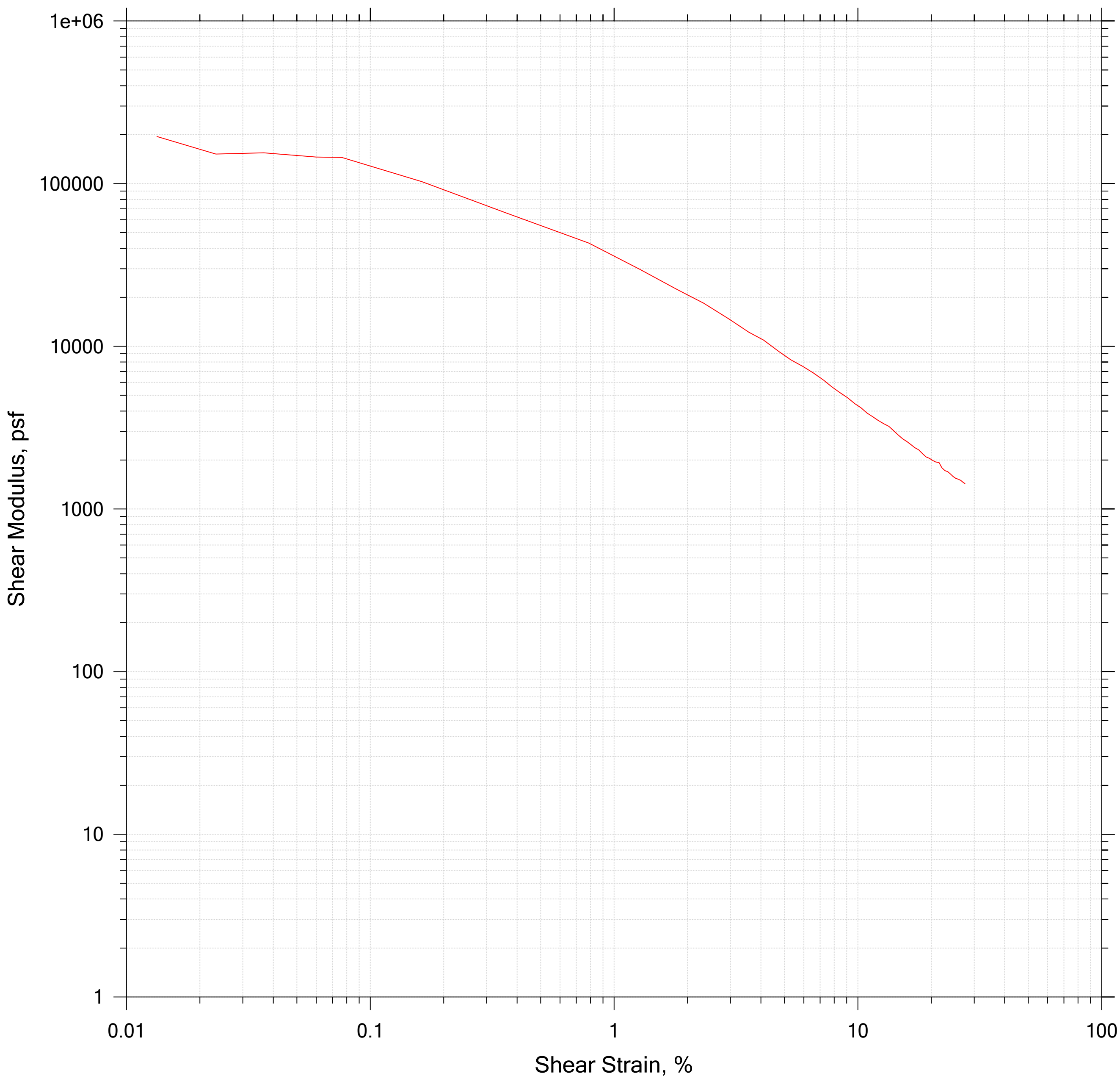
Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Specimen Dimension, in: 2.50	Specific Gravity: 2.80 (Estimated)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 0.785	Plastic Limit: 0
Final Height, in: 0.96	Final Void Ratio: 0.718	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	217	---		408
Mass Container, gm	36.89	0	0	88.51
Mass Container + Wet Soil, gm	151.8	160.53	158.33	245.3
Mass Container + Dry Soil, gm	122.33	126.38	126.38	213.66
Mass Dry Soil, gm	85.44	126.38	126.38	125.15
Water Content, %	34.49	27.02	25.28	25.28
Void Ratio	---	0.79	0.72	---
Degree of Saturation, %	---	96.38	98.58	---
Dry Unit Weight, pcf	---	97.924	101.74	---

Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
0.00000	0.00000	16.496	0.00000	0.00000	2198.5	0.00000
0.00018333	0.00000	16.496	0.00000	0.00000	2198.5	0.00000
0.016767	-0.0033247	18.852	0.00000	0.0017341	2198.5	0.00000
0.034183	-0.0033247	18.852	0.00000	0.0024657	2207.9	-0.0042781
0.084033	-0.0033247	21.209	0.00000	0.00096183	2222.0	-0.010695
0.10068	0.00000	21.209	0.00000	0.0016710	2212.6	-0.0064171
0.16737	-0.0033247	23.565	0.00000	0.0040144	2158.5	0.018182
0.25098	0.013299	25.922	1.9492e+05	0.0073829	2113.8	0.038503
0.50072	0.023273	35.348	1.5189e+05	0.0084853	2076.2	0.055615
1.0007	0.036571	56.557	1.5465e+05	0.0022416	2182.0	0.0074866
2.0009	0.059844	87.192	1.4570e+05	0.0055326	2184.4	0.0064171
4.0009	0.076467	110.76	1.4484e+05	0.014621	1993.9	0.093048
8.0007	0.16291	167.31	1.0271e+05	0.012524	2045.7	0.069519
15.001	0.35906	238.01	66287.	0.018613	2033.9	0.074866
30.001	0.78794	339.34	43067.	0.027066	1817.6	0.17326
45.001	1.2800	379.40	29641.	0.041368	1610.7	0.26738
60.001	1.8086	405.33	22411.	0.047917	1474.3	0.32941
75.001	2.3306	428.89	18403.	0.053201	1387.3	0.36898
90.000	2.9556	435.96	14750.	0.059485	1283.8	0.41604
105.00	3.5773	435.96	12187.	0.062443	1178.0	0.46417
120.00	4.0993	447.74	10922.	0.069910	1112.2	0.49412
135.00	4.7443	440.67	9288.6	0.073929	1074.6	0.51123
150.00	5.3095	438.32	8255.4	0.073031	1032.2	0.53048
165.00	5.9677	447.74	7502.7	0.075829	994.61	0.54759
180.00	6.5562	450.10	6865.3	0.090068	931.12	0.57647
195.00	7.2378	447.74	6186.2	0.081552	971.10	0.55829
210.00	7.7996	440.67	5649.9	0.097111	879.40	0.60000
225.00	8.3914	438.32	5223.4	0.091249	893.50	0.59358
240.00	9.0597	438.32	4838.1	0.096174	874.69	0.60214
255.00	9.7213	431.25	4436.1	0.10176	841.77	0.61711
270.00	10.293	431.25	4189.7	0.10309	844.13	0.61604
285.00	10.918	424.18	3885.1	0.10190	851.18	0.61283
300.00	11.507	424.18	3686.4	0.10190	851.18	0.61283
315.00	12.095	424.18	3507.0	0.10216	839.42	0.61818
330.00	12.790	426.54	3334.9	0.097111	879.40	0.60000
345.00	13.408	431.25	3216.3	0.10070	858.23	0.60963
360.00	14.013	424.18	3027.0	0.10575	818.26	0.62781
375.00	14.628	417.11	2851.4	0.10962	815.91	0.62888
390.00	15.283	412.40	2698.3	0.10894	799.45	0.63636
405.00	15.902	412.40	2593.4	0.10748	818.26	0.62781
420.00	16.573	410.04	2474.1	0.11267	787.69	0.64171
435.00	17.125	407.68	2380.6	0.12145	735.96	0.66524
450.00	17.774	410.04	2307.0	0.11865	752.42	0.65775
465.00	18.379	402.97	2192.6	0.11706	761.83	0.65348
480.00	19.047	398.26	2090.9	0.11239	768.88	0.65027
495.00	19.682	402.97	2047.4	0.12065	740.67	0.66310
510.00	20.224	402.97	1992.5	0.11799	766.53	0.65134
525.00	20.812	405.33	1947.5	0.11506	773.59	0.64813
540.00	21.514	414.75	1927.8	0.11586	768.88	0.65027
555.00	22.102	395.90	1791.2	0.12623	707.75	0.67807

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

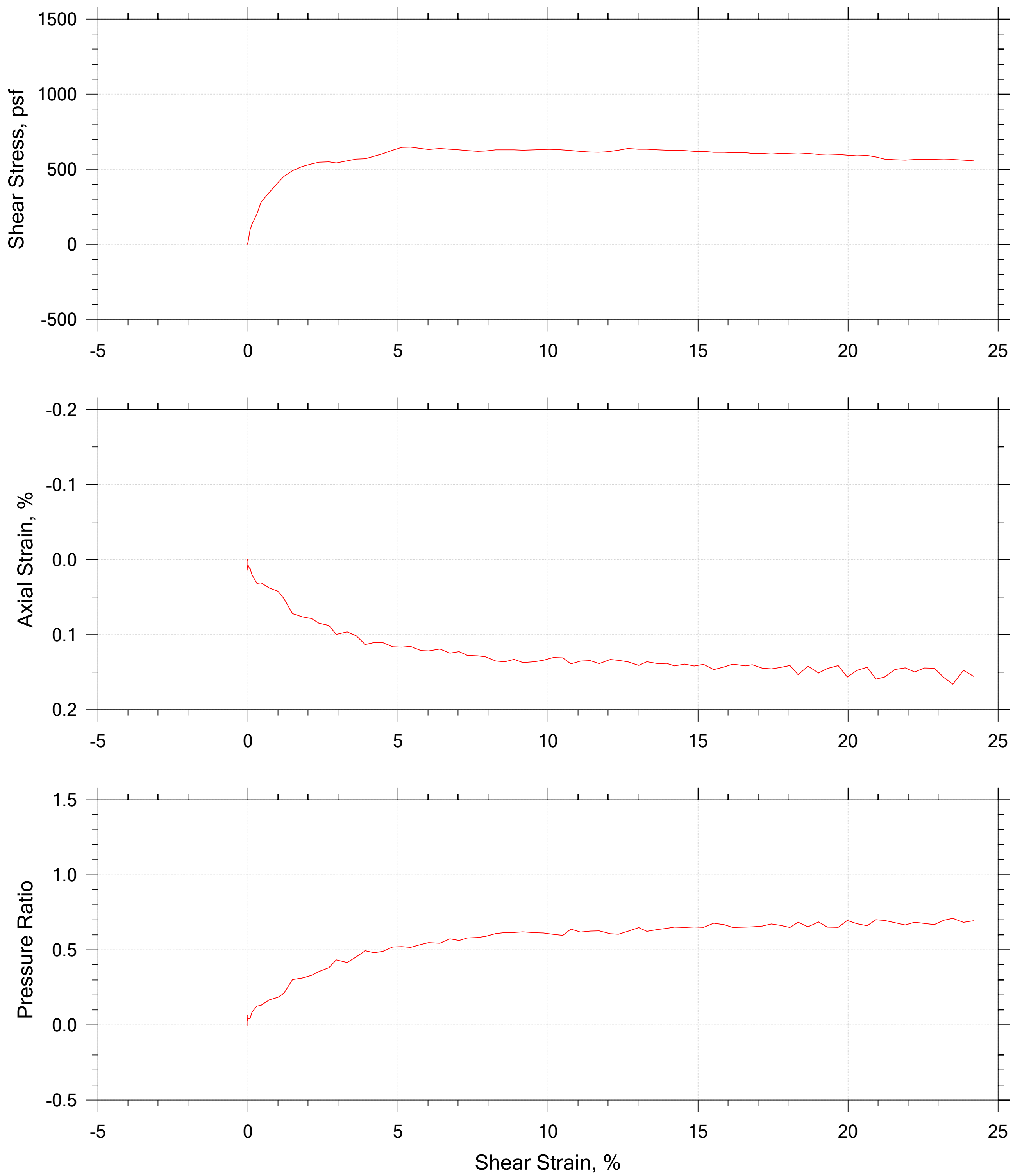
Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
570.00	22.737	391.19	1720.5	0.11546	771.23	0.64920
585.00	23.359	395.90	1694.9	0.12889	681.88	0.68984
600.00	23.901	393.54	1646.6	0.11891	740.67	0.66310
615.00	24.553	388.83	1583.7	0.12264	728.91	0.66845
630.00	25.184	388.83	1543.9	0.12729	691.29	0.68556
645.00	25.833	393.54	1523.4	0.12344	724.21	0.67059
660.00	26.411	395.90	1499.0	0.12424	719.51	0.67273
675.00	27.019	393.54	1456.5	0.12544	712.45	0.67594
687.48	27.558	393.54	1428.1	0.12250	719.51	0.67273

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 11/18/19	Depth: 40.55
	Test Number: DSS 105	Preparation: wet	Elevation: -5.55
	Description: Gray Silty Clay		
	Remarks: Sample was a little gritty when cutting (possible sand). Sample Consolidated to 2,200 psf, held for 2 hours then sheared.		

DSS 106

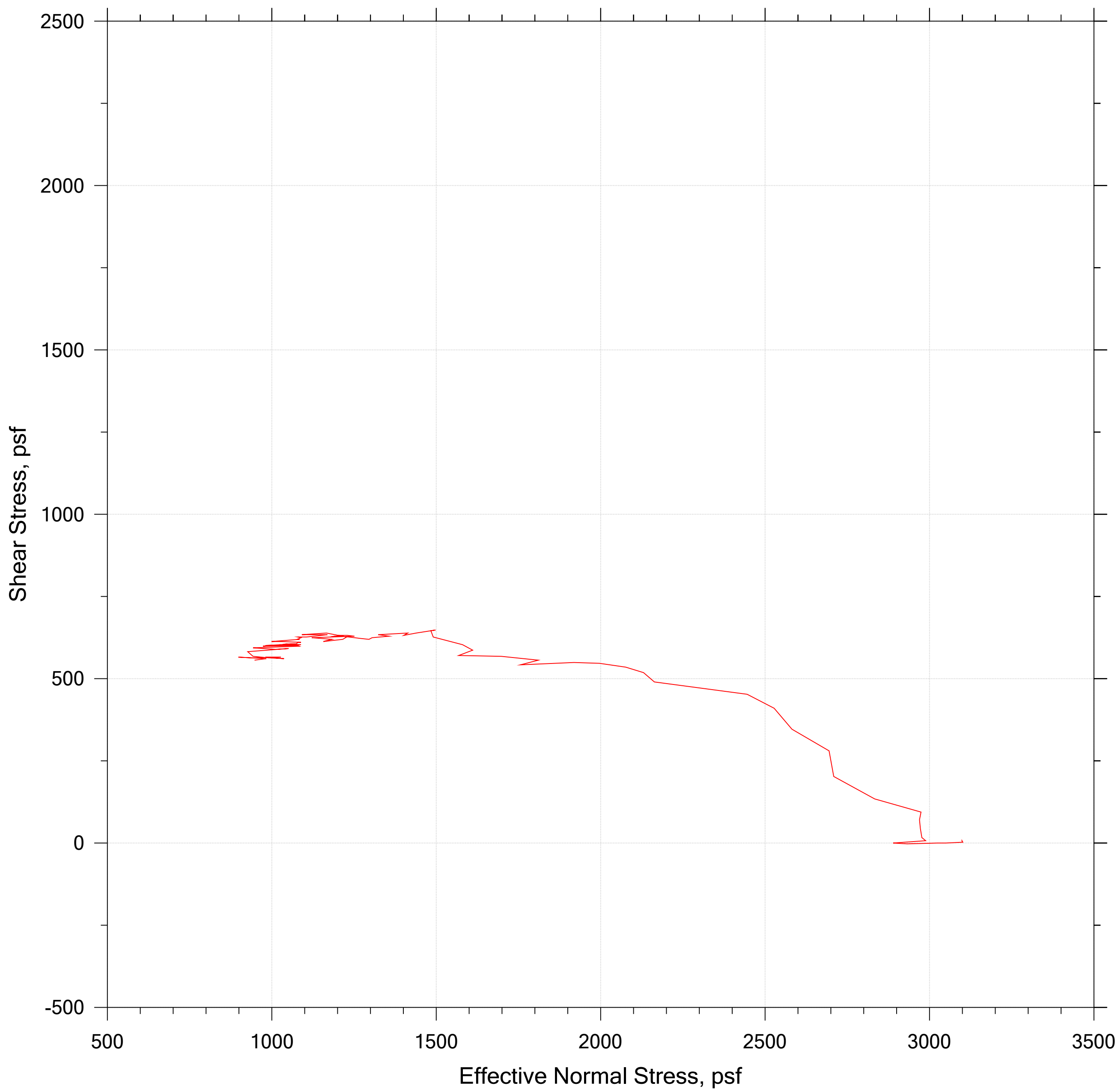
BB-FJR-103 1U

Direct Simple Shear Test by ASTM D6528



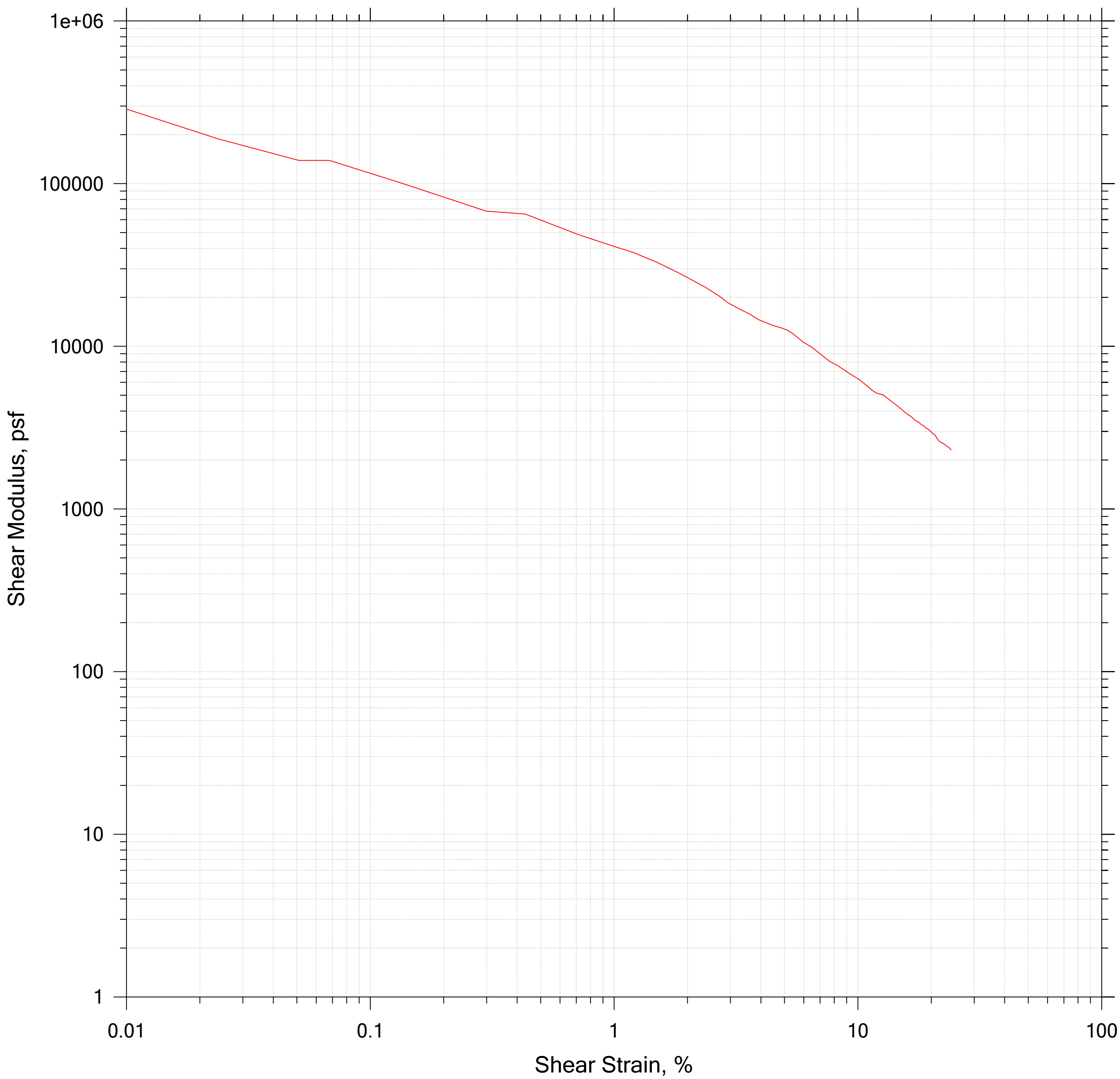
Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Specimen Dimension, in: 2.50	Specific Gravity: 2.80 (Estimated)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 1.14	Plastic Limit: 0
Final Height, in: 0.94	Final Void Ratio: 1.01	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	207	---		405
Mass Container, gm	36.81	0	0	88.05
Mass Container + Wet Soil, gm	137.07	148.63	144	231.68
Mass Container + Dry Soil, gm	108.91	105.62	105.62	193.4
Mass Dry Soil, gm	72.1	105.62	105.62	105.35
Water Content, %	39.06	40.72	36.34	36.34
Void Ratio	---	1.14	1.01	---
Degree of Saturation, %	---	100.38	100.51	---
Dry Unit Weight, pcf	---	81.84	86.869	---

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
0.00000	0.00000	4.7131	0.00000	0.00000	3099.0	0.00000
0.00021667	0.00000	7.0696	0.00000	0.0017711	3099.0	0.00000
0.017100	-0.0067910	2.3565	0.00000	-0.00025599	3101.4	-0.00075873
0.033767	-0.0067910	2.3565	0.00000	-0.00025599	3101.4	-0.00075873
0.083767	-0.0033955	0.00000	0.00000	0.0072696	3049.7	0.015933
0.10020	-0.0033955	0.00000	0.00000	0.0074902	3023.8	0.024279
0.16687	0.00000	-2.3565	0.00000	0.014552	2932.1	0.053869
0.25030	-0.0067910	0.00000	0.00000	0.014269	2889.8	0.067527
0.50058	-0.010186	7.0696	0.00000	0.0084351	2988.5	0.035660
1.0002	0.0033955	16.496	4.8582e+05	0.0075693	2976.8	0.039454
2.0004	0.023768	44.774	1.8838e+05	0.0097025	2972.1	0.040971
4.0002	0.050932	70.696	1.3881e+05	0.011655	2969.7	0.041730
8.0004	0.067910	94.262	1.3881e+05	0.011293	2974.4	0.040212
15.000	0.13242	134.32	1.0143e+05	0.020385	2833.3	0.085736
30.000	0.29880	202.66	67825.	0.031753	2708.7	0.12595
45.000	0.43123	280.43	65031.	0.031068	2694.6	0.13050
60.000	0.71305	346.41	48582.	0.037988	2581.8	0.16692
75.000	0.99488	410.04	41215.	0.042152	2527.7	0.18437
90.000	1.1986	452.46	37749.	0.052032	2445.4	0.21093
105.00	1.4838	490.16	33034.	0.071988	2163.2	0.30197
120.00	1.8098	518.44	28646.	0.076294	2130.3	0.31259
135.00	2.1154	534.94	25288.	0.078688	2076.2	0.33005
150.00	2.3666	546.72	23101.	0.084809	1996.3	0.35584
165.00	2.6960	549.08	20366.	0.087851	1918.7	0.38088
180.00	2.9405	542.01	18433.	0.099686	1756.4	0.43323
195.00	3.3038	556.15	16834.	0.096331	1810.5	0.41578
210.00	3.6026	567.93	15764.	0.10156	1697.7	0.45220
225.00	3.9082	570.28	14592.	0.11313	1568.3	0.49393
240.00	4.2036	586.78	13959.	0.11050	1610.7	0.48027
255.00	4.4956	603.28	13419.	0.11062	1580.1	0.49014
270.00	4.8216	626.84	13001.	0.11617	1490.7	0.51897
285.00	5.1238	645.69	12602.	0.11660	1483.7	0.52124
300.00	5.4158	648.05	11966.	0.11573	1497.8	0.51669
315.00	5.7621	638.62	11083.	0.12115	1439.0	0.53566
330.00	6.0202	631.56	10491.	0.12171	1401.4	0.54780
345.00	6.3937	638.62	9988.4	0.11921	1413.1	0.54401
360.00	6.7298	633.91	9419.4	0.12475	1323.8	0.57284
375.00	7.0388	629.20	8939.0	0.12271	1356.7	0.56222
390.00	7.3071	624.49	8546.3	0.12769	1305.0	0.57891
405.00	7.6636	619.77	8087.2	0.12827	1295.6	0.58194
420.00	7.9217	622.13	7853.5	0.12973	1272.1	0.58953
435.00	8.2544	629.20	7622.6	0.13515	1213.3	0.60850
450.00	8.5600	629.20	7350.4	0.13632	1194.5	0.61457
465.00	8.8656	629.20	7097.1	0.13307	1189.8	0.61608
480.00	9.1644	626.84	6840.0	0.13734	1178.0	0.61988
495.00	9.5549	629.20	6585.1	0.13617	1196.8	0.61381
510.00	9.8469	631.56	6413.8	0.13411	1201.5	0.61229
525.00	10.183	631.56	6202.0	0.13059	1229.7	0.60319
540.00	10.492	629.20	5996.9	0.13104	1250.9	0.59636
555.00	10.764	624.49	5801.8	0.13907	1121.6	0.63809

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

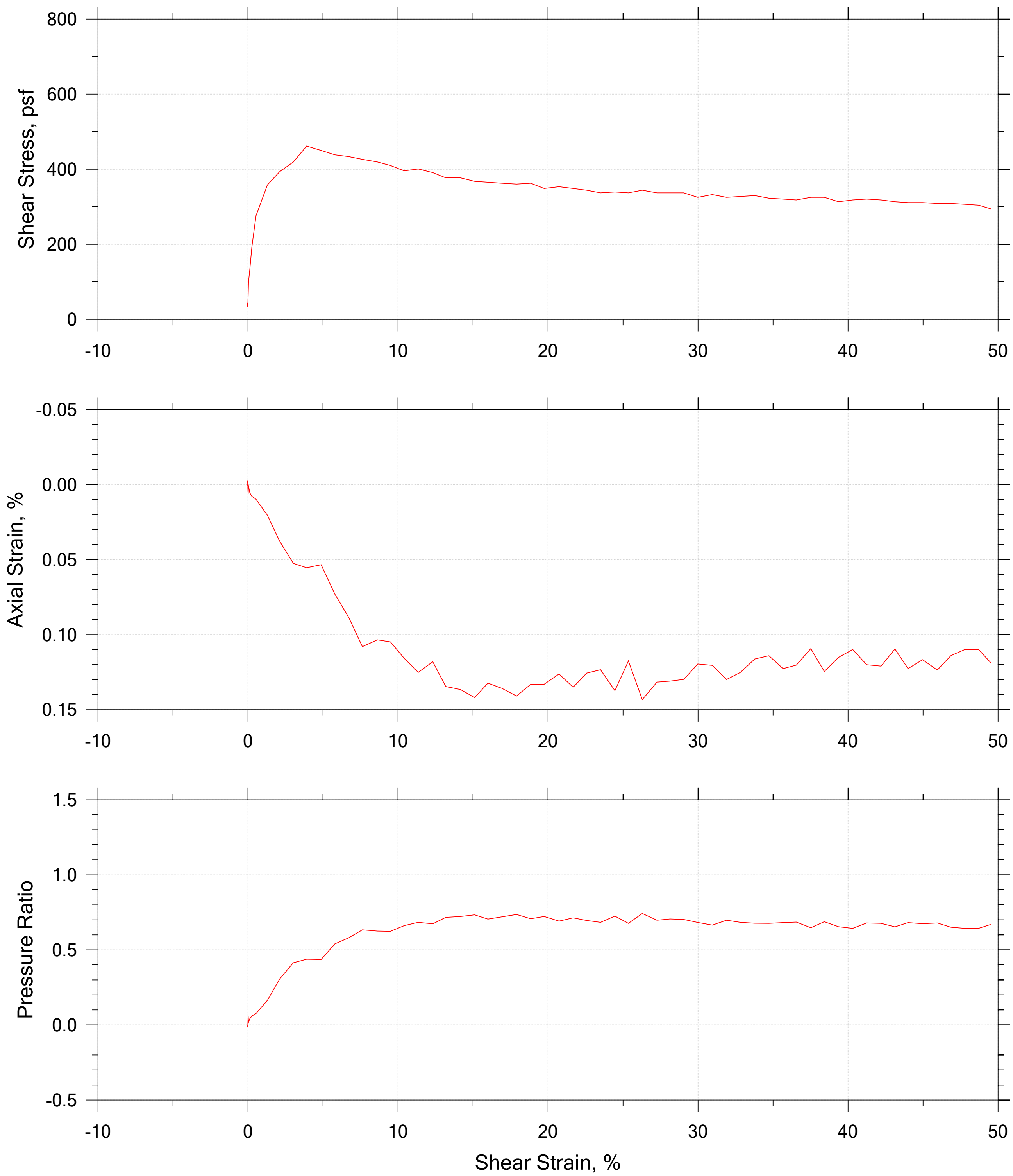
Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
570.00	11.083	619.77	5592.2	0.13527	1182.7	0.61836
585.00	11.405	615.06	5392.7	0.13467	1163.9	0.62443
600.00	11.704	612.70	5234.9	0.13865	1156.9	0.62671
615.00	12.078	619.77	5131.5	0.13323	1215.6	0.60774
630.00	12.346	626.84	5077.3	0.13427	1227.4	0.60395
645.00	12.669	638.62	5041.0	0.13630	1166.3	0.62367
660.00	13.025	633.91	4866.9	0.14096	1091.0	0.64795
675.00	13.293	633.91	4768.7	0.13615	1168.6	0.62291
690.00	13.663	629.20	4605.0	0.13863	1128.6	0.63581
705.00	13.959	626.84	4490.7	0.13832	1105.1	0.64340
720.00	14.213	626.84	4410.2	0.14169	1079.3	0.65175
735.00	14.560	624.49	4289.1	0.13934	1088.7	0.64871
750.00	14.872	619.77	4167.3	0.14184	1076.9	0.65250
765.00	15.185	619.77	4081.6	0.13963	1084.0	0.65023
780.00	15.528	612.70	3945.9	0.14665	999.31	0.67754
795.00	15.874	612.70	3859.8	0.14298	1029.9	0.66768
810.00	16.159	610.35	3777.1	0.13934	1088.7	0.64871
825.00	16.580	610.35	3681.2	0.14169	1079.3	0.65175
840.00	16.811	605.63	3602.6	0.14021	1074.6	0.65326
855.00	17.137	605.63	3534.1	0.14463	1060.4	0.65781
870.00	17.446	600.92	3444.5	0.14563	1015.8	0.67223
885.00	17.738	605.63	3414.3	0.14388	1044.0	0.66313
900.00	18.064	603.28	3339.7	0.14111	1088.7	0.64871
915.00	18.339	600.92	3276.7	0.15345	980.50	0.68361
930.00	18.662	605.63	3245.4	0.14198	1074.6	0.65326
945.00	19.018	598.56	3147.3	0.15113	973.45	0.68589
960.00	19.310	600.92	3111.9	0.14523	1079.3	0.65175
975.00	19.670	598.56	3043.0	0.14140	1084.0	0.65023
990.00	19.976	593.85	2972.9	0.15643	942.88	0.69575
1005.0	20.295	589.14	2902.9	0.14769	1011.1	0.67375
1020.0	20.638	591.49	2866.1	0.14344	1051.0	0.66085
1035.0	20.930	582.07	2781.1	0.15928	926.42	0.70106
1050.0	21.218	567.93	2676.6	0.15643	942.88	0.69575
1065.0	21.565	563.22	2611.7	0.14651	989.91	0.68058
1080.0	21.904	560.86	2560.5	0.14432	1036.9	0.66540
1095.0	22.220	565.57	2545.3	0.14991	980.50	0.68361
1110.0	22.546	565.57	2508.5	0.14444	1006.4	0.67527
1125.0	22.875	565.57	2472.4	0.14490	1027.5	0.66844
1140.0	23.195	563.22	2428.2	0.15724	938.18	0.69727
1155.0	23.493	565.57	2407.4	0.16594	898.21	0.71017
1170.0	23.846	560.86	2352.0	0.14773	982.85	0.68285
1184.3	24.189	556.15	2299.1	0.15561	947.58	0.69423

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-103	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/18/19	Depth: 45.85
	Test Number: DSS 106	Preparation: wet	Elevation: 1.95
	Description: Gray silty clay		
	Remarks: Sample consolidated to 3,100 psf, held for 2 hours, then sheared.		

DSS 107

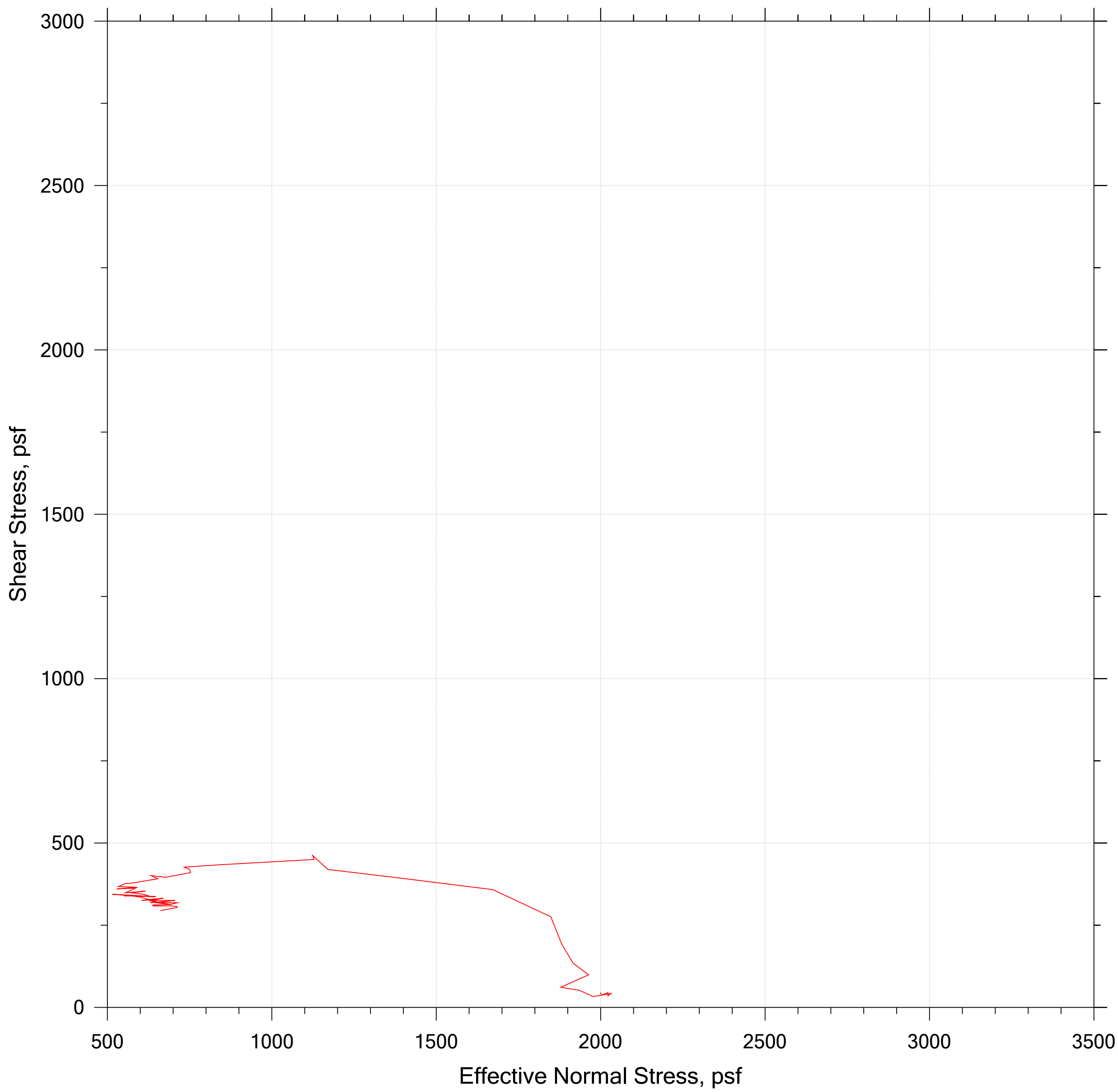
BB-FJR-104 1U

Direct Simple Shear Test by ASTM D6528



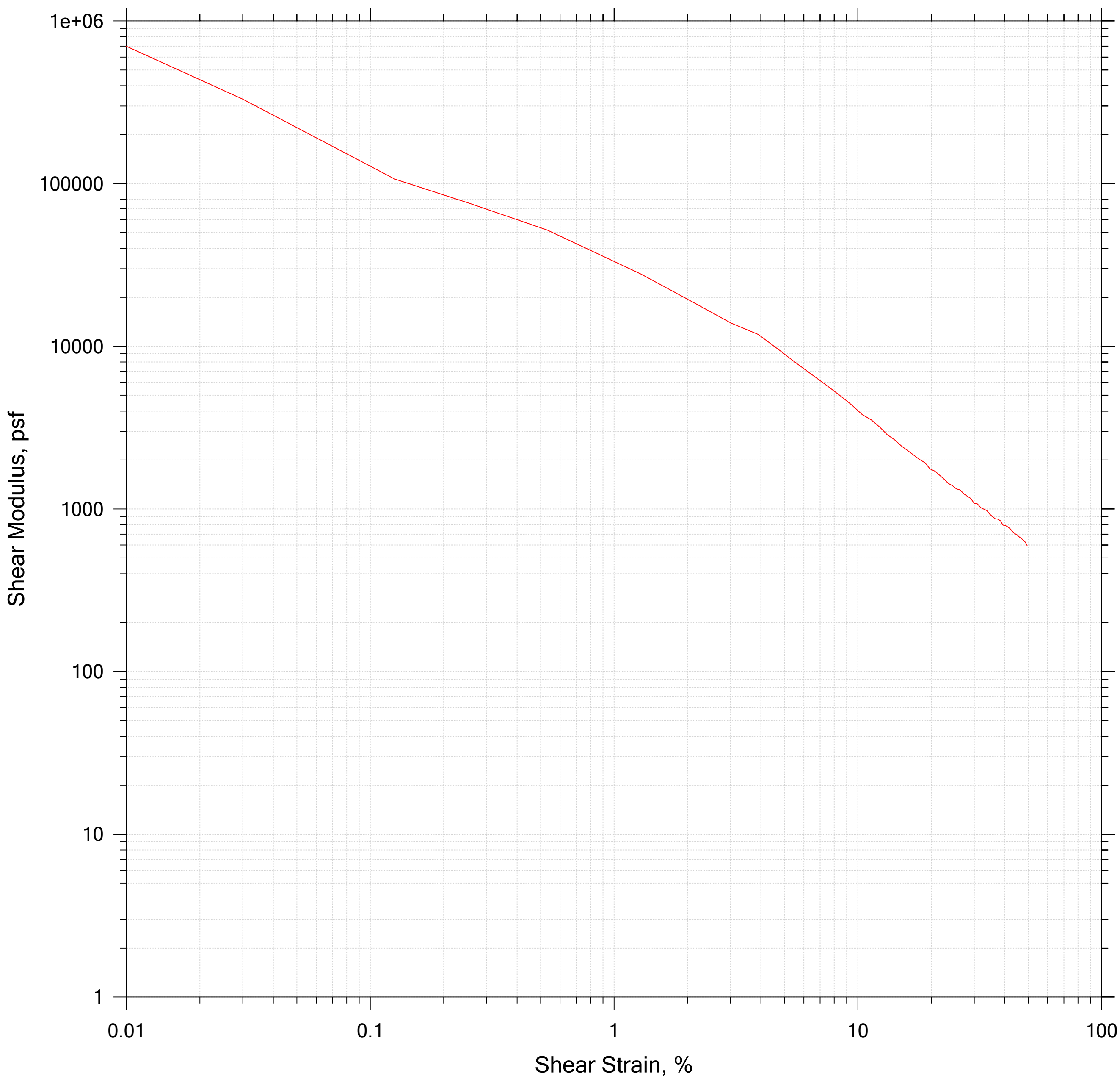
Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Specimen Dimension, in: 2.50	Specific Gravity: 2.80 (Estimated)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 1.16	Plastic Limit: 0
Final Height, in: 0.97	Final Void Ratio: 1.08	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	202	---		321
Mass Container, gm	36.86	0	0	60.4
Mass Container + Wet Soil, gm	159.98	148.59	144.62	204.59
Mass Container + Dry Soil, gm	124.93	104.68	104.68	164.77
Mass Dry Soil, gm	88.07	104.68	104.68	104.37
Water Content, %	39.80	41.95	38.15	38.15
Void Ratio	---	1.16	1.08	---
Degree of Saturation, %	---	101.68	99.12	---
Dry Unit Weight, pcf	---	81.112	84.126	---

Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
0.00000	0.00000	44.774	0.00000	0.00000	1998.6	0.00000
0.00021667	0.00000	44.774	0.00000	0.00000	1998.6	0.00000
0.017533	-0.0033190	37.705	0.00000	0.0012308	2003.3	-0.0023529
0.034217	-0.0066380	40.061	0.00000	-0.0012511	2010.4	-0.0058824
0.083800	-0.0066380	44.774	0.00000	-0.0025022	2022.1	-0.011765
0.10012	-0.013276	35.348	0.00000	-0.0025022	2022.1	-0.011765
0.16743	-0.0066380	42.418	0.00000	0.0016905	2031.5	-0.016471
0.25093	-0.0066380	32.992	0.00000	0.0019890	1977.5	0.010588
0.50072	-0.0033190	51.844	0.00000	0.0045553	1935.1	0.031765
1.0007	0.0066380	61.270	9.2302e+05	0.0062458	1878.7	0.060000
2.0002	0.029871	98.975	3.3134e+05	0.0011133	1963.4	0.017647
4.0007	0.12612	134.32	1.0650e+05	0.0056959	1916.3	0.041176
8.0009	0.25556	193.24	75612.	0.0078344	1881.1	0.058824
15.001	0.53104	275.72	51920.	0.0098304	1848.1	0.075294
30.001	1.2878	358.20	27815.	0.020523	1671.8	0.16353
45.001	2.1076	393.54	18673.	0.037774	1387.3	0.30588
60.001	3.0203	419.47	13888.	0.052622	1171.0	0.41412
75.001	3.9065	461.88	11824.	0.055473	1123.9	0.43765
90.001	4.8756	450.10	9231.6	0.053457	1128.6	0.43529
105.00	5.7884	438.32	7572.4	0.073108	919.37	0.54000
120.00	6.7077	433.61	6464.3	0.088380	839.42	0.58000
135.00	7.6205	426.54	5597.2	0.10803	733.61	0.63294
150.00	8.6228	419.47	4864.6	0.10351	750.07	0.62471
165.00	9.4891	410.04	4321.2	0.10485	752.42	0.62353
180.00	10.418	395.90	3800.0	0.11586	677.18	0.66118
195.00	11.351	400.61	3529.3	0.12516	632.51	0.68353
210.00	12.317	391.19	3176.0	0.11811	653.67	0.67294
225.00	13.180	377.05	2860.8	0.13458	566.67	0.71647
240.00	14.156	377.05	2663.6	0.13657	554.91	0.72235
255.00	15.111	367.62	2432.7	0.14188	533.75	0.73294
270.00	15.988	365.27	2284.7	0.13233	590.18	0.70471
285.00	16.917	362.91	2145.2	0.13577	559.62	0.72000
300.00	17.903	360.55	2013.9	0.14095	529.05	0.73529
315.00	18.852	362.91	1925.0	0.13312	585.48	0.70706
330.00	19.745	348.77	1766.4	0.13311	554.91	0.72235
345.00	20.737	353.48	1704.6	0.12621	616.05	0.69176
360.00	21.673	348.77	1609.2	0.13511	573.72	0.71294
375.00	22.573	344.06	1524.2	0.12568	608.99	0.69529
390.00	23.502	336.99	1433.9	0.12343	632.51	0.68353
405.00	24.458	339.34	1387.5	0.13737	550.21	0.72471
420.00	25.357	336.99	1329.0	0.11757	646.61	0.67647
435.00	26.283	344.06	1309.0	0.14334	514.94	0.74235
450.00	27.262	336.99	1236.1	0.13167	604.29	0.69765
465.00	28.132	336.99	1197.9	0.13099	587.83	0.70588
480.00	29.055	336.99	1159.8	0.12980	594.89	0.70235
495.00	29.987	325.20	1084.5	0.11957	634.86	0.68235
510.00	30.950	332.27	1073.6	0.12052	670.13	0.66471
525.00	31.899	325.20	1019.5	0.12994	604.29	0.69765
540.00	32.825	327.56	997.90	0.12516	632.51	0.68353
555.00	33.788	329.92	976.44	0.11624	644.26	0.67765

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

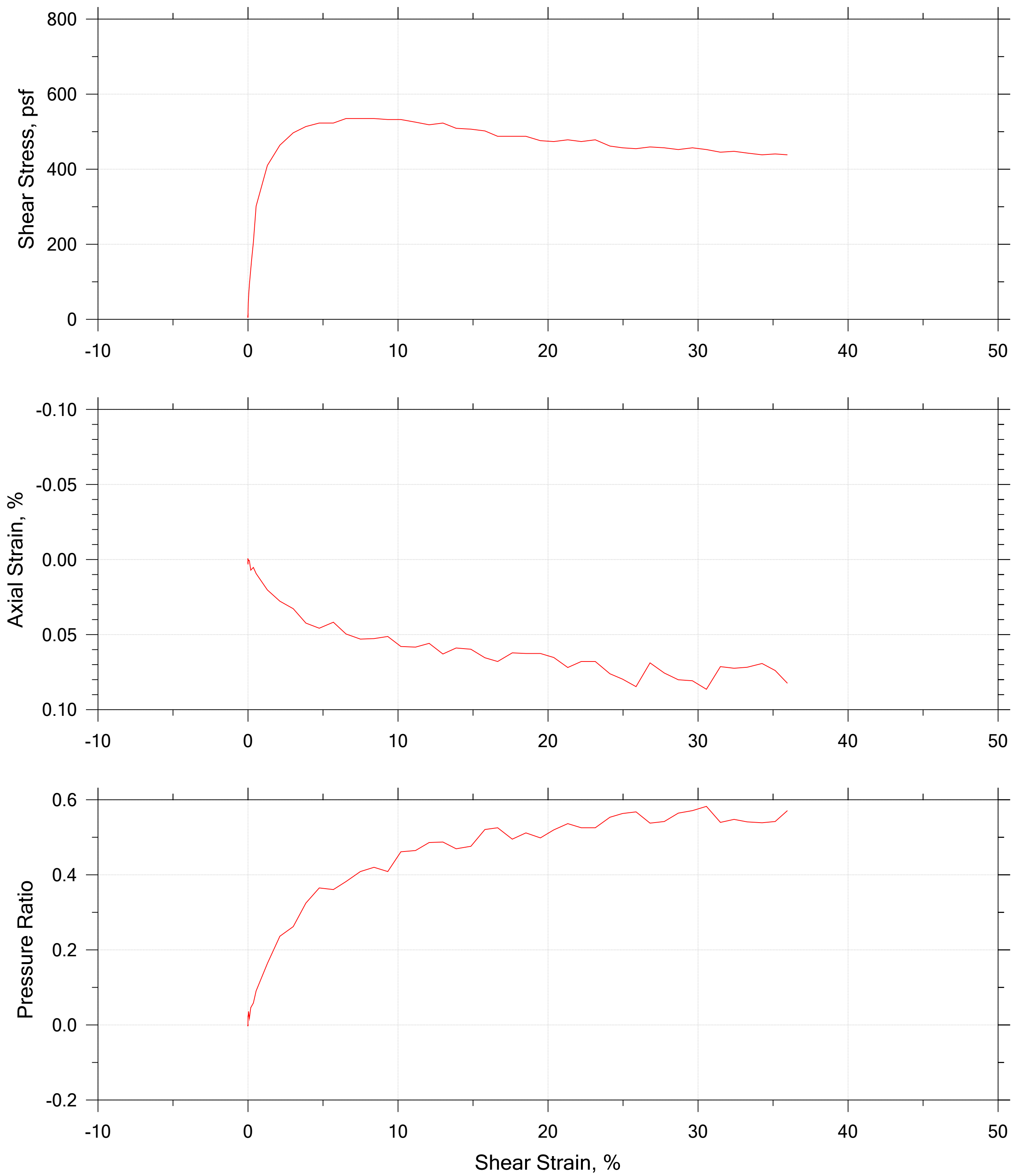
Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
570.00	34.727	322.85	929.67	0.11411	646.61	0.67647
585.00	35.666	320.49	898.58	0.12263	637.21	0.68118
600.00	36.552	318.13	870.35	0.12036	630.16	0.68471
615.00	37.518	325.20	866.79	0.10935	705.40	0.64706
630.00	38.418	325.20	846.50	0.12462	625.45	0.68706
645.00	39.367	313.42	796.15	0.11520	691.29	0.65412
660.00	40.313	318.13	789.16	0.10989	712.45	0.64353
675.00	41.245	320.49	777.03	0.12010	641.91	0.67882
690.00	42.205	318.13	753.79	0.12104	646.61	0.67647
705.00	43.127	313.42	726.73	0.10961	693.64	0.65294
720.00	44.004	311.06	706.91	0.12263	637.21	0.68118
735.00	44.973	311.06	691.67	0.11678	651.32	0.67412
750.00	45.952	308.71	671.81	0.12356	641.91	0.67882
765.00	46.861	308.71	658.77	0.11401	698.34	0.65059
780.00	47.791	306.35	641.03	0.10989	712.45	0.64353
795.00	48.703	303.99	624.18	0.10989	712.45	0.64353
807.43	49.510	294.57	594.97	0.11865	660.72	0.66941

Soil Metrics LLC			
	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/20/19	Depth: 30.83
	Test Number: DSS 107	Preparation: wet	Elevation: 4.17
	Description: Gray silty clay		
	Remarks: Sample consolidated to 2,000 psf, held for 2 hours then sheared.		

DSS 108

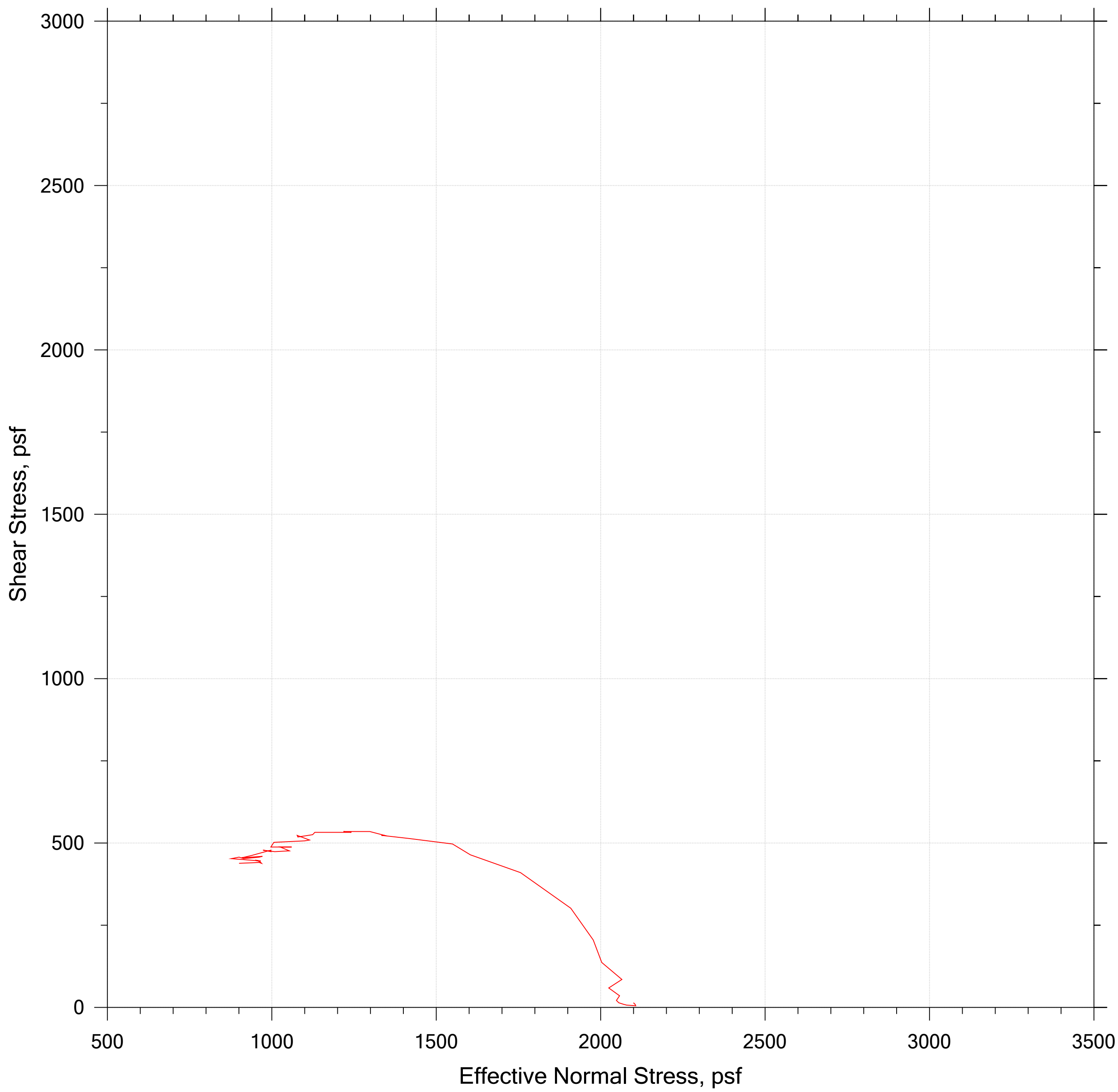
BB-FJR-102A 1U

Direct Simple Shear Test by ASTM D6528



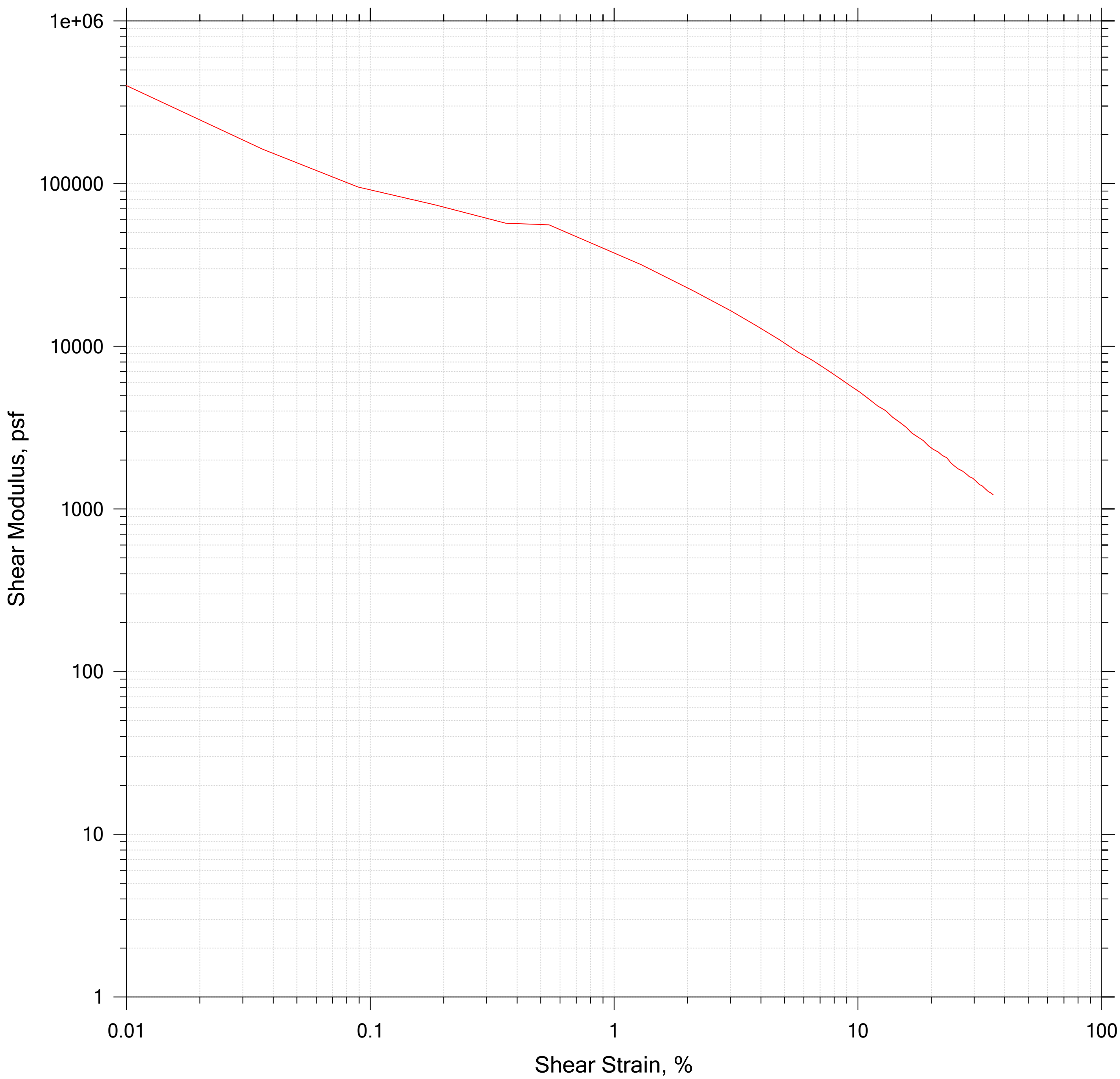
Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528



Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Specimen Dimension, in: 2.50	Specific Gravity: 2.82 (Implied)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 1.17	Plastic Limit: 0
Final Height, in: 0.97	Final Void Ratio: 1.11	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	212	---		301
Mass Container, gm	36.72	0	0	60.57
Mass Container + Wet Soil, gm	146.18	148.08	145.73	206.24
Mass Container + Dry Soil, gm	117.57	104.79	104.79	165.32
Mass Dry Soil, gm	80.85	104.79	104.79	104.75
Water Content, %	35.39	41.31	39.06	39.06
Void Ratio	---	1.17	1.11	---
Degree of Saturation, %	---	99.72	99.55	---
Dry Unit Weight, pcf	---	81.198	83.57	---

Warning: The change in the sample wet weight during the test is not consistent with the change in the moisture content.

Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
0.00000	0.00000	14.139	0.00000	0.00000	2099.7	0.00000
0.00036667	0.0032948	14.139	4.2914e+05	0.00000	2099.7	0.00000
0.017367	-0.0065895	9.4262	0.00000	-0.00042409	2104.4	-0.0022396
0.034033	-0.0098843	4.7131	0.00000	-0.00067248	2106.8	-0.0033595
0.083517	-0.0032948	7.0696	0.00000	0.0031544	2078.6	0.010078
0.10022	0.00000	9.4262	0.00000	0.0021386	2069.2	0.014558
0.16712	0.00000	14.139	0.00000	0.0014742	2055.1	0.021277
0.25052	0.00000	21.209	0.00000	0.0020013	2048.0	0.024636
0.50053	0.0065895	35.348	5.3643e+05	0.0012985	2057.4	0.020157
1.0005	0.036242	58.914	1.6255e+05	0.00032109	2024.5	0.035834
2.0007	0.088959	84.836	95365.	0.00077143	2064.5	0.016797
4.0005	0.18451	136.68	74078.	0.0070580	2003.3	0.045913
8.0005	0.35913	205.02	57088.	0.0052460	1977.5	0.058231
15.000	0.54034	301.64	55824.	0.0093504	1909.3	0.090705
30.000	1.2915	410.04	31748.	0.020268	1756.4	0.16349
45.001	2.1251	464.24	21845.	0.027749	1603.6	0.23628
60.000	3.0147	497.23	16493.	0.032723	1549.5	0.26204
75.000	3.8582	513.73	13315.	0.042367	1417.8	0.32475
90.000	4.7511	523.15	11011.	0.045743	1333.2	0.36506
105.00	5.6934	523.15	9188.8	0.041740	1342.6	0.36058
120.00	6.5368	534.94	8183.4	0.049585	1297.9	0.38186
135.00	7.4890	534.94	7143.0	0.052982	1241.5	0.40873
150.00	8.3984	534.94	6369.5	0.052678	1218.0	0.41993
165.00	9.3176	532.58	5715.8	0.051263	1241.5	0.40873
180.00	10.187	532.58	5227.8	0.057915	1131.0	0.46137
195.00	11.163	525.51	4707.7	0.058339	1123.9	0.46473
210.00	12.075	518.44	4293.4	0.055873	1079.3	0.48600
225.00	12.988	523.15	4028.0	0.062889	1076.9	0.48712
240.00	13.871	509.01	3669.6	0.058906	1114.5	0.46920
255.00	14.856	506.66	3410.4	0.059755	1100.4	0.47592
270.00	15.795	501.94	3177.8	0.065416	1006.4	0.52072
285.00	16.642	487.81	2931.2	0.067954	996.96	0.52520
300.00	17.614	487.81	2769.4	0.062161	1060.4	0.49496
315.00	18.523	487.81	2633.5	0.062565	1025.2	0.51176
330.00	19.495	476.02	2441.7	0.062585	1053.4	0.49832
345.00	20.385	473.67	2323.6	0.065274	1008.7	0.51960
360.00	21.320	478.38	2243.8	0.071908	973.45	0.53639
375.00	22.220	473.67	2131.7	0.067954	996.96	0.52520
390.00	23.149	478.38	2066.5	0.067954	996.96	0.52520
405.00	24.124	461.88	1914.6	0.076120	938.18	0.55319
420.00	24.964	457.17	1831.3	0.079678	917.02	0.56327
435.00	25.864	454.81	1758.5	0.084696	907.61	0.56775
450.00	26.800	459.53	1714.7	0.068866	971.10	0.53751
465.00	27.755	457.17	1647.2	0.075603	961.69	0.54199
480.00	28.681	452.46	1577.6	0.080073	914.67	0.56439
495.00	29.627	457.17	1543.1	0.080727	900.56	0.57111
510.00	30.559	452.46	1480.6	0.086399	877.04	0.58231
525.00	31.491	445.39	1414.3	0.071375	966.39	0.53975
540.00	32.397	447.74	1382.0	0.072424	949.94	0.54759
555.00	33.280	443.03	1331.2	0.071771	964.04	0.54087

Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Direct Simple Shear Test by ASTM D6528

Shear Phase

Elapsed Time min	Shear Strain %	Shear Stress psf	Shear Modulus psf	Normal Strain %	Normal Stress psf	Pressure Ratio
570.00	34.272	438.32	1278.9	0.069261	968.75	0.53863
585.00	35.142	440.67	1254.0	0.073885	961.69	0.54199
597.96	35.962	438.32	1218.8	0.082445	900.56	0.57111

Soil Metrics LLC	Project Name: Johnson Road Bridge	Location: Falmouth,Maine	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: 1U	Test Date: 11/21/19	Depth: 31.1
	Test Number: DSS 108	Preparation: wet	Elevation: 5.8
	Description:		
	Remarks: Sample consolidated to 2,100 psf, held for 2 hours, then sheared.		

Soil Metrics LLC Geotechnical Engineering
12 Farms Edge Road
Cape Elizabeth, Maine 04107
Ph. 207. 767.2192
Cell. 207.415.5835

MEMORANDUM

TO: **ANDREW BLAISDELL**

FROM: **STEVE RABASCA**

DATE: **OCTOBER 21, 2019**

SUBJECT: **JOHNSON ROAD LAB TEST RESULTS**

Attached please find the lab test results for the Johnson Road Bridge Project. The following test results are included:

Incremental Consolidation Tests:
- BB-FJR-102A 1U
- BB-FJR 102A 2U (see time curve for load step 11 for c_{alpha})
- BB-FJR 104 1U
- BB-FJR 104 2U

Constant Rate of Strain Consolidation Tests:
- BB-FJR-102A 3U

Consolidated (K_o) Undrained Triaxial Compression Tests:
- BB FJR 102A 1U
- BB FJR 102A 3U
- BB FJR 104 1U
- BB FJR 104 2U

Atterberg Limits: (Results included on Consolidation test Summary Sheets)
- BB-FJR-102A 1U
- BB-FJR 102A 2U
- BB-FJR 104 1U
- BB-FJR 104 2U

Please call me if you have any questions.

Regards - Steve

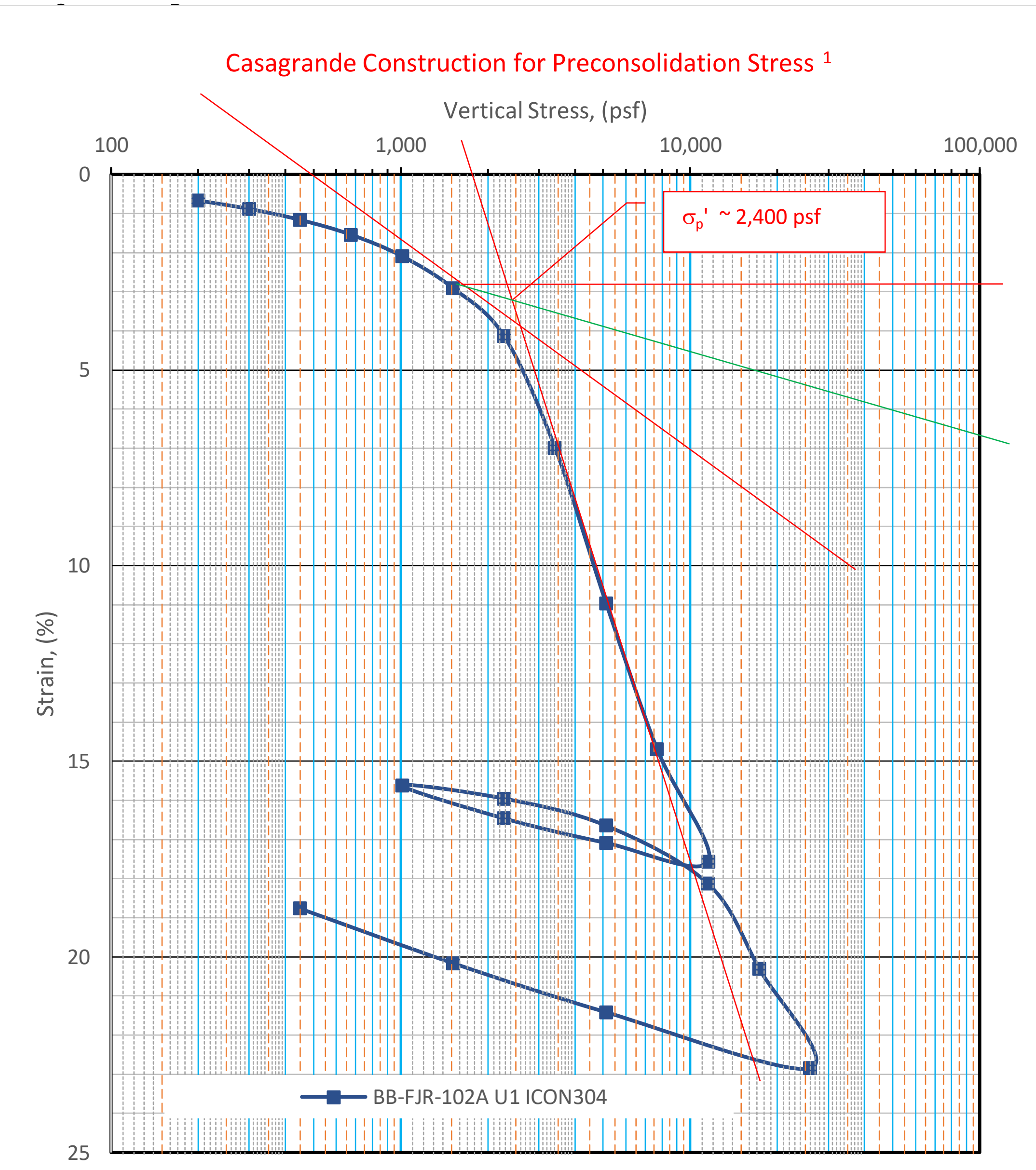
INCREMENTAL CONSOLIDATION

BB-FJR-102A 1U

Consolidation Test Data

Project Name:	Johnson Road Bridge
Project Number:	166-11
Project Location:	Falmouth, Maine
Client:	GZA Proj. No. 09.0026024.00
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	ICON 304			
Boring No.	BB-FJR-102A			
Sample No:	1U			
Boring Elevation (ft).	36.9			
Sample Depth (ft):	30 - 32			
Test Specimen Depth (Ft):	31.38			
Test Specimen Elevation:	5.52			
Water Content (%):	35.49			
Dry Unit Weight (pcf):	81.82			
Wet Unit Weight (pcf):	110.86			
Saturation Before (%):	97.73			
Saturation After (%):	100			
Void Ratio Before:	1.2			
Void Ratio After:	0.78			
Overburden Pressure (psf):				
Max Previous stress (psf):	2,400			
Max Prev. stress (Work) (psf):	2100.00			
OCR:				
Compression Index (C_{CE}):	0.23			
Recompression Index (C_{RE}):	0.021			
Liquid Limit:	39			
Plastic Limit:	21			
Plasticity Index:	18			
Liquidity Index:	0.8			
Specific Gravity (implied)	2.88			
Lab Vane Su at 21 ft. (psf)				
Tested By:	sjr			
Date Tested:	9/22/2019			
Checked By:	sjr			

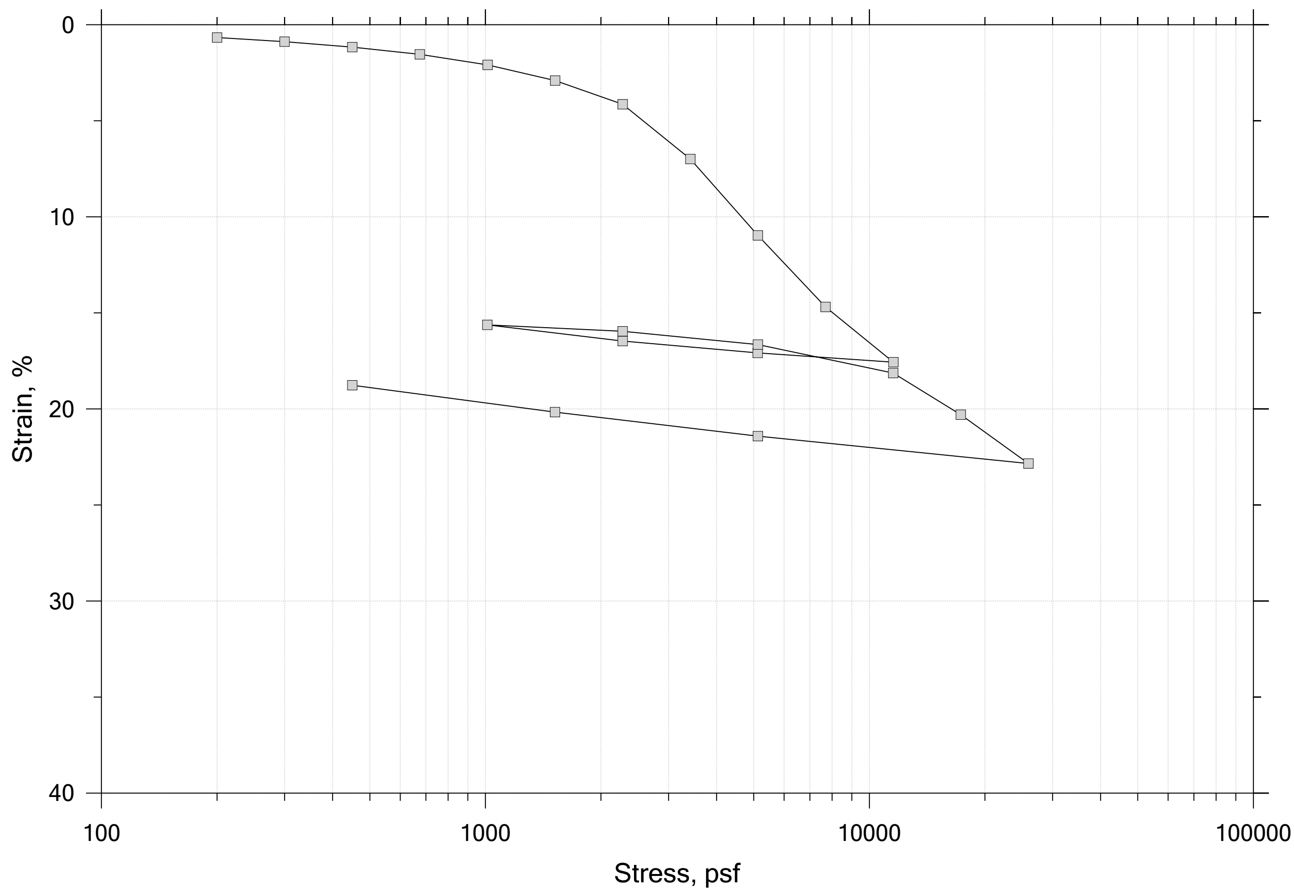


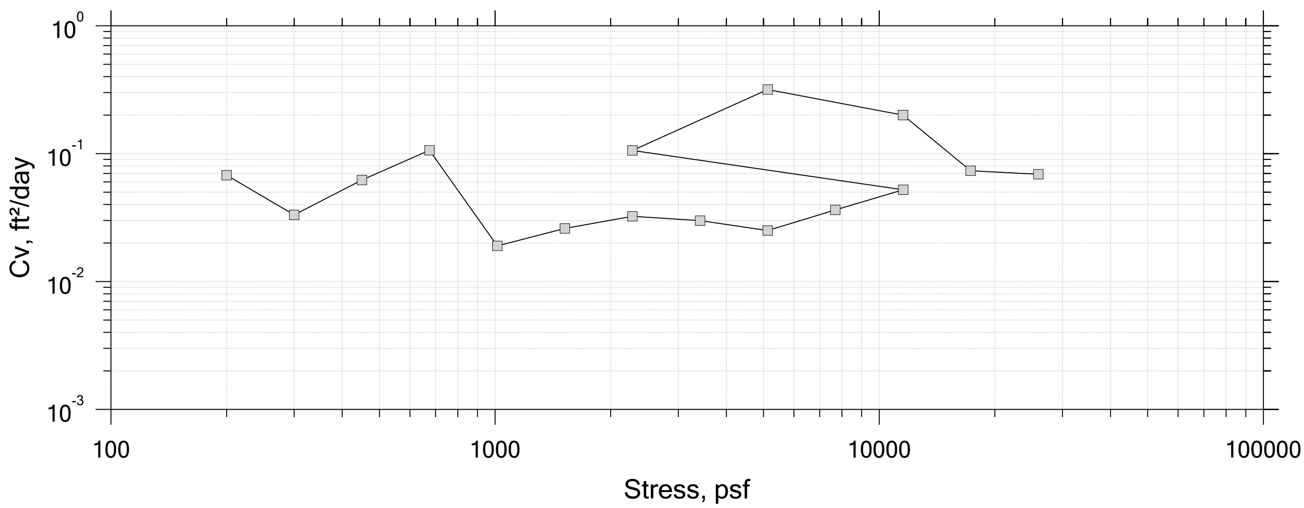
Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convienience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.

Soil Metrics LLC Geotechnical Engineering

One-Dimensional Consolidation by ASTM D2435 - Method B

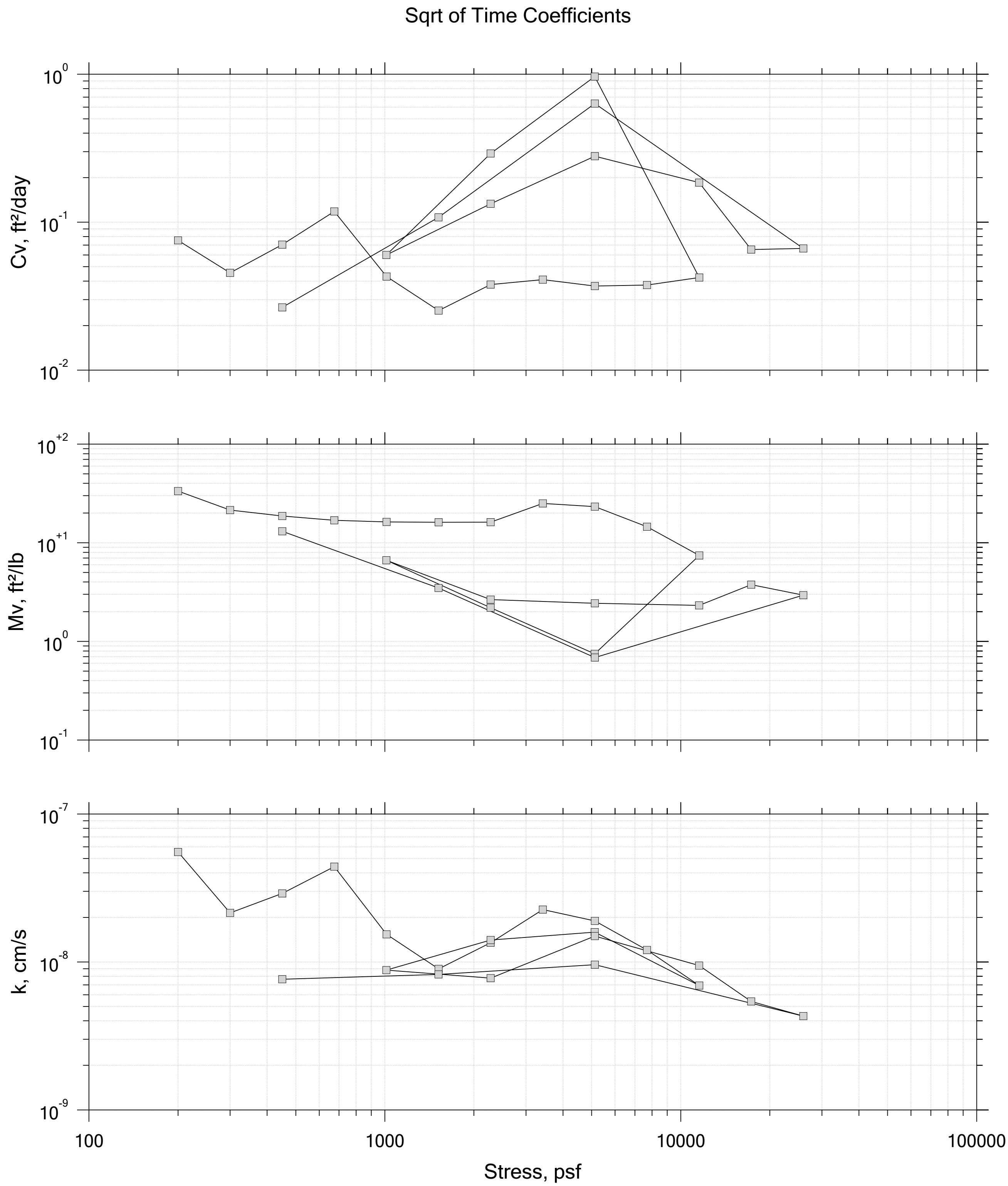
Summary Report





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		
	Displacement at End of Primary		

One-Dimensional Consolidation by ASTM D2435 - Method B



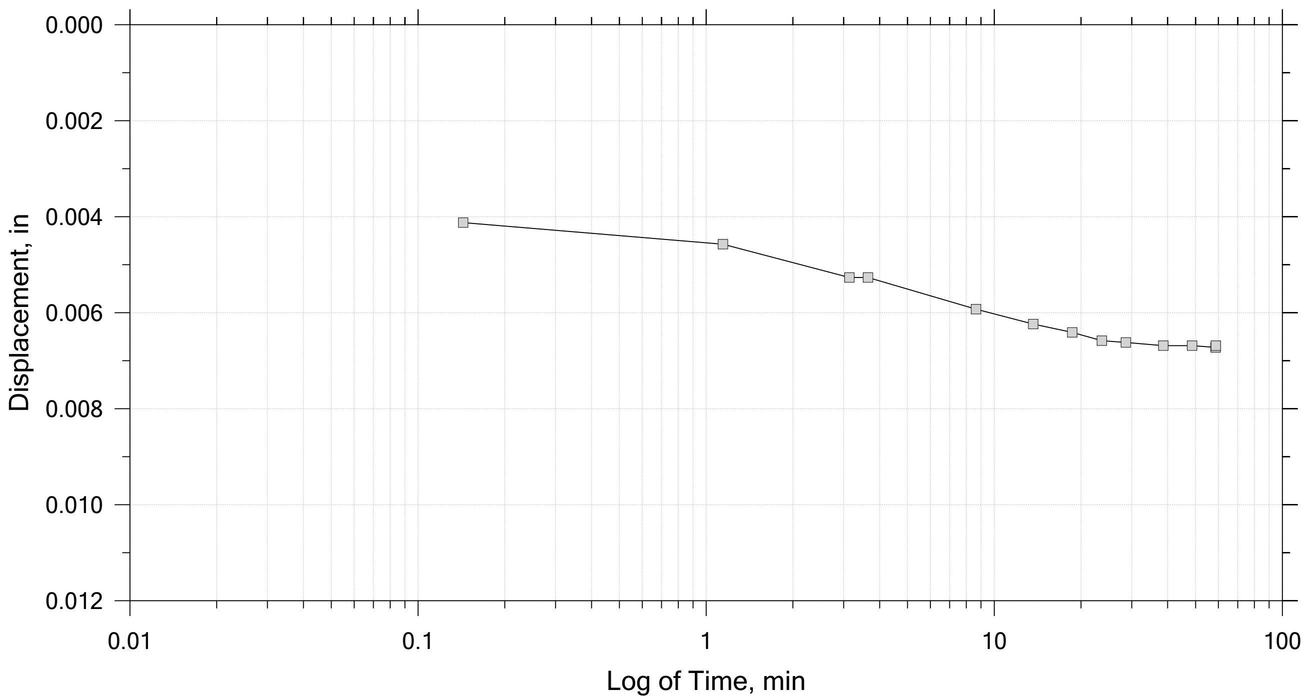
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

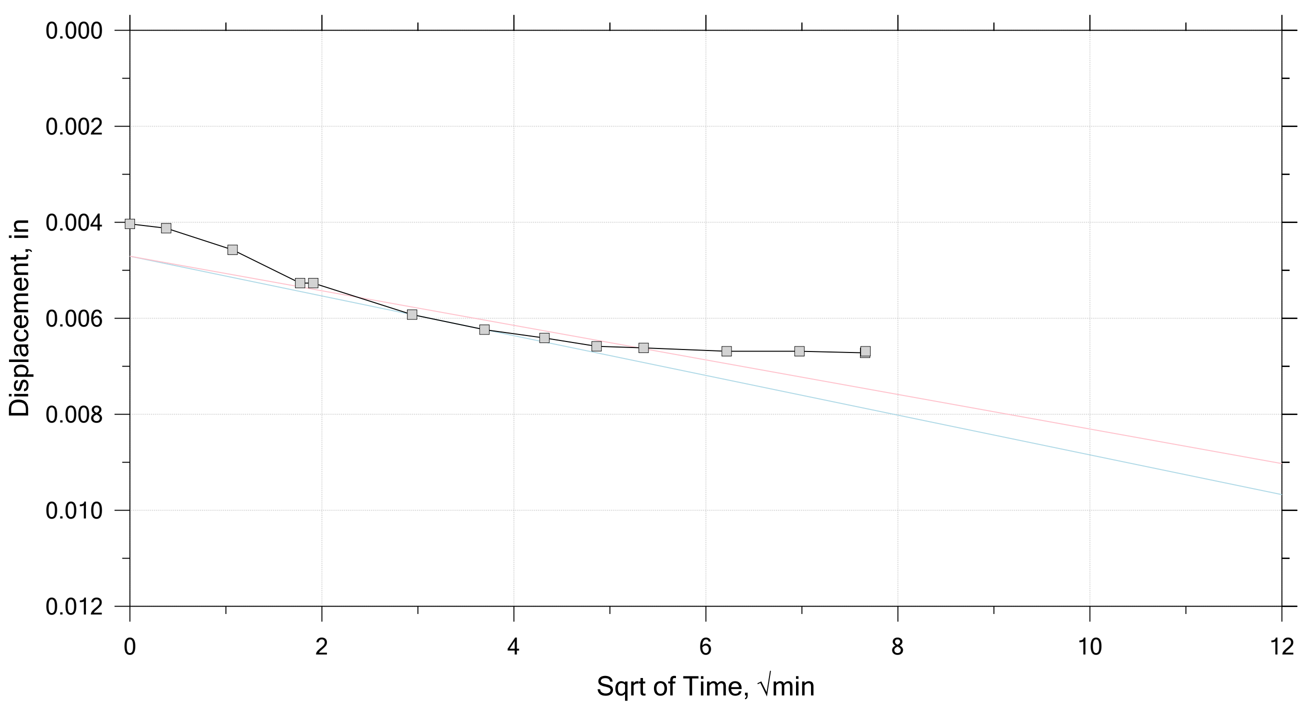
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 22

Constant Load Step

Stress: 200 psf





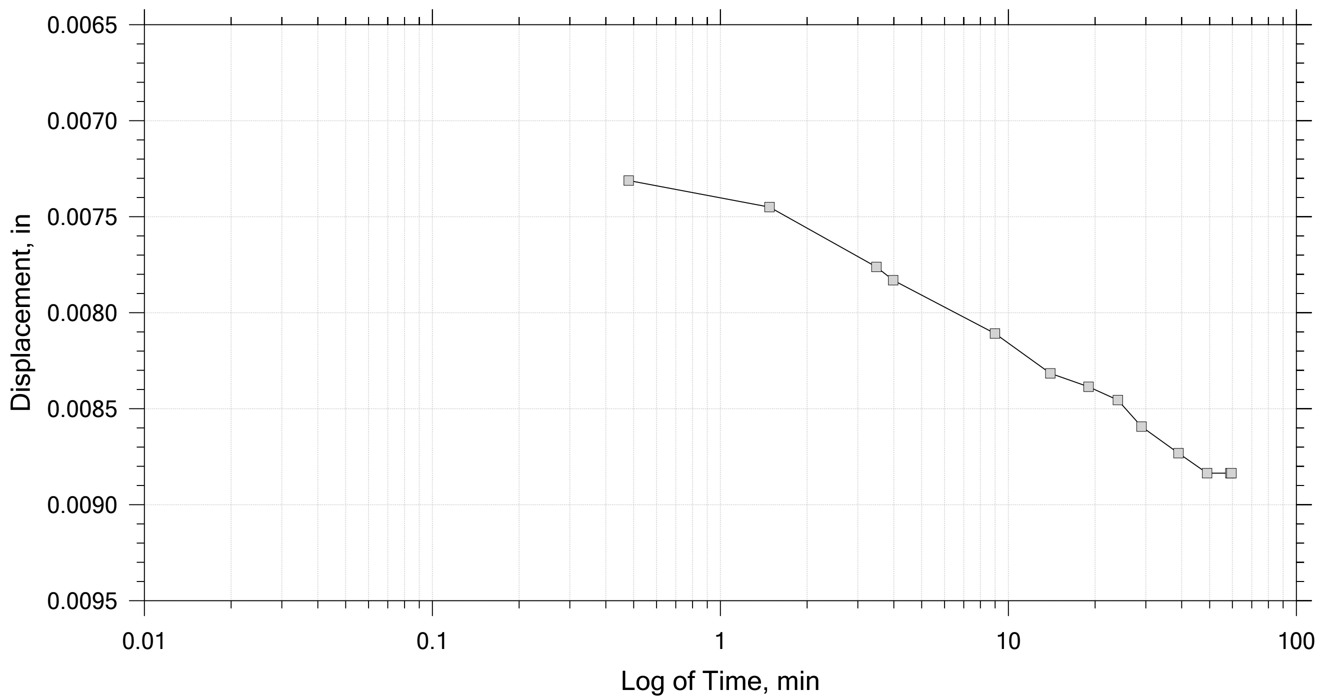
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

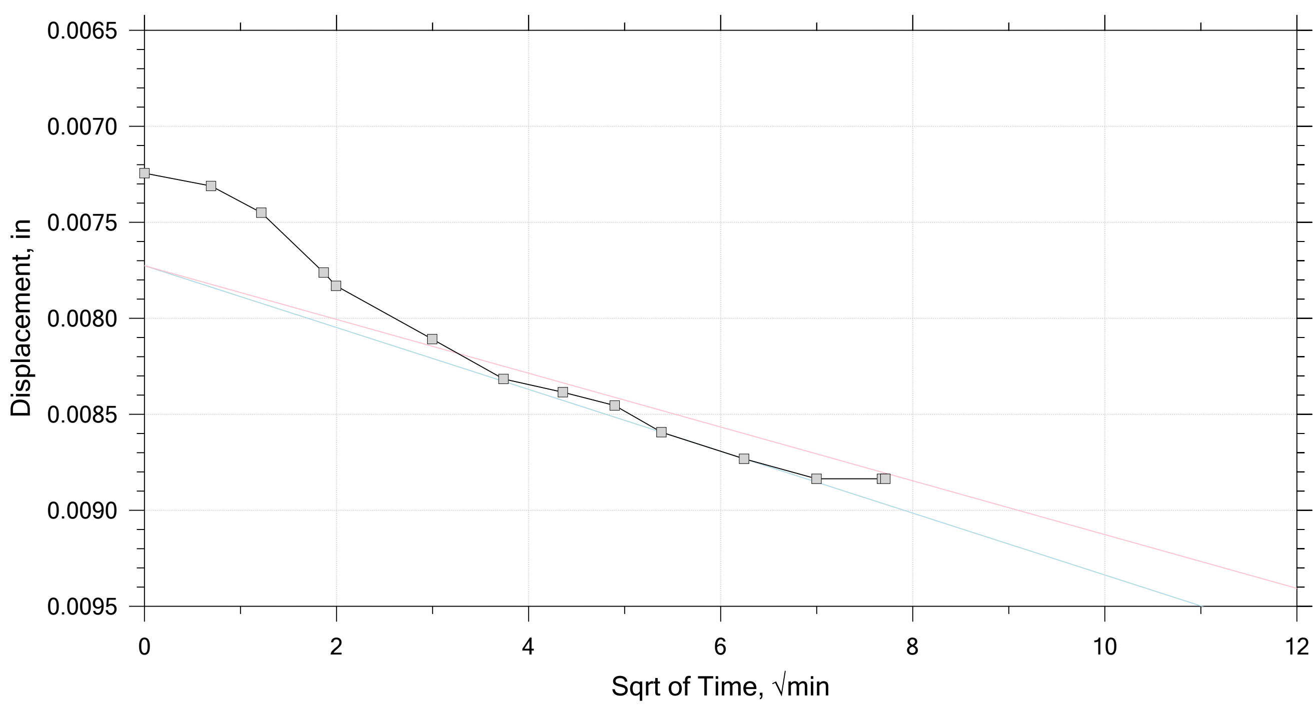
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 22

Constant Load Step

Stress: 300 psf





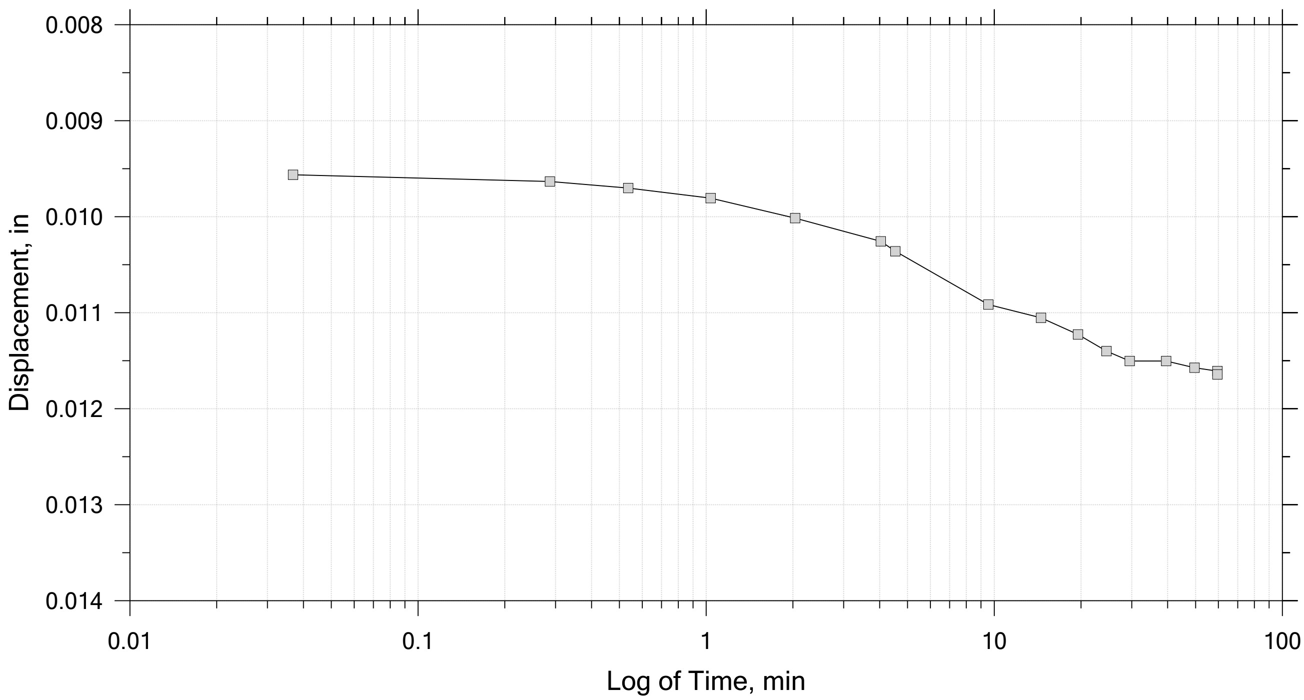
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

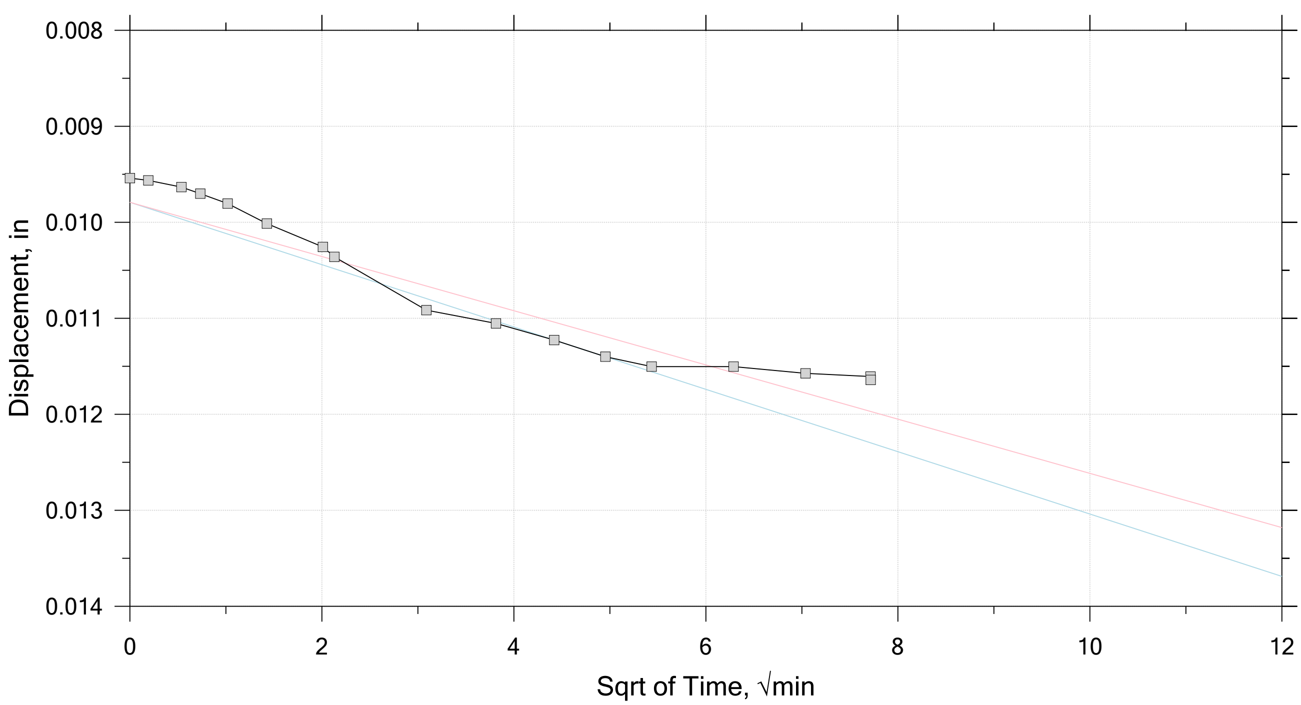
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 22

Constant Load Step

Stress: 450 psf





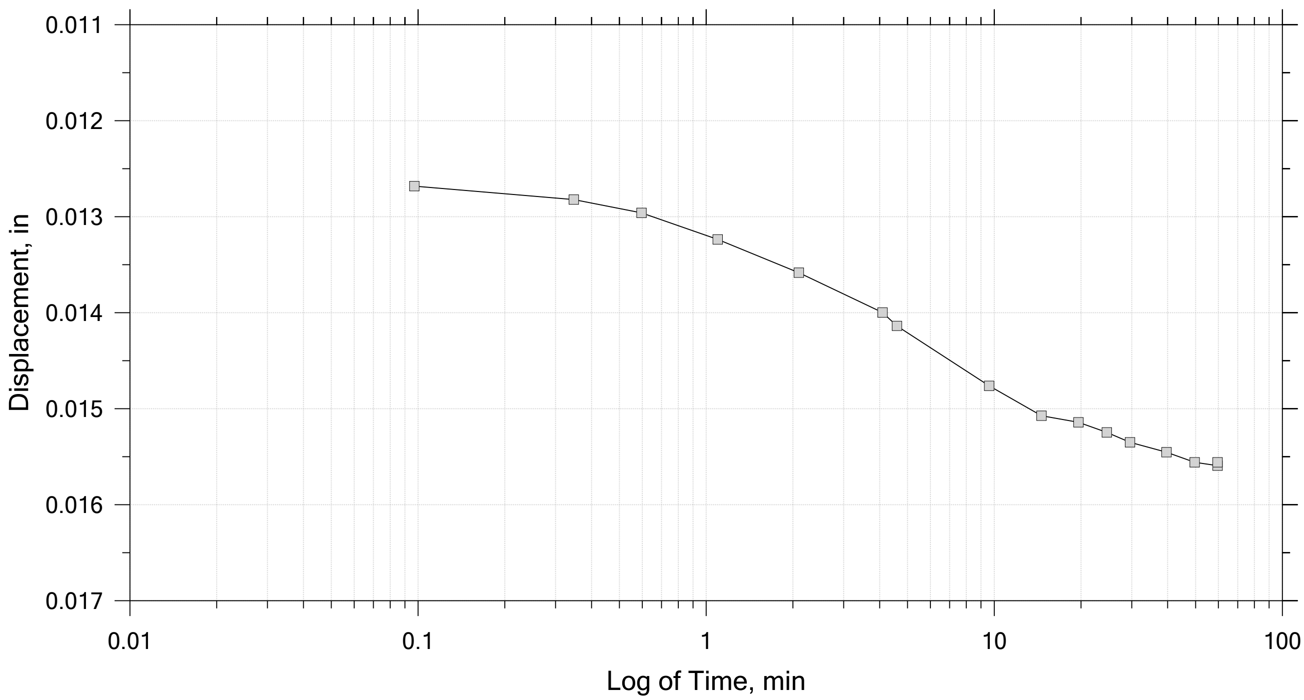
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

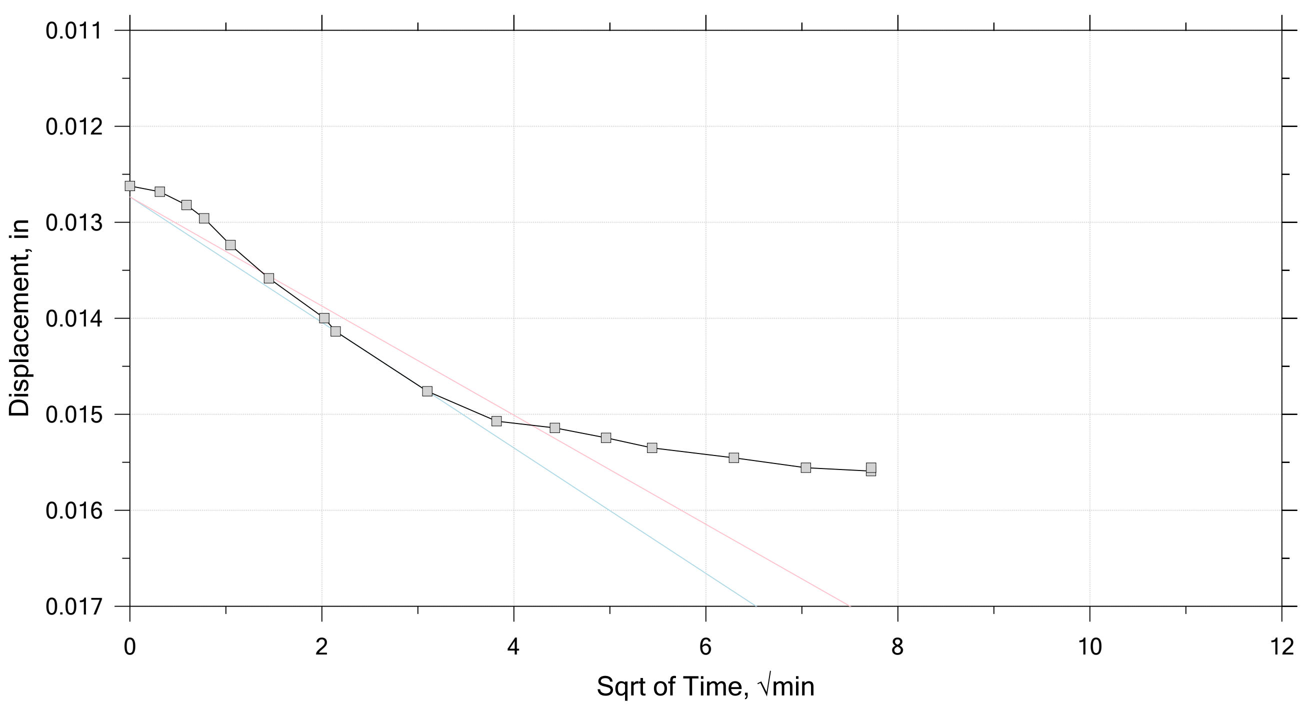
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 22

Constant Load Step

Stress: 675 psf





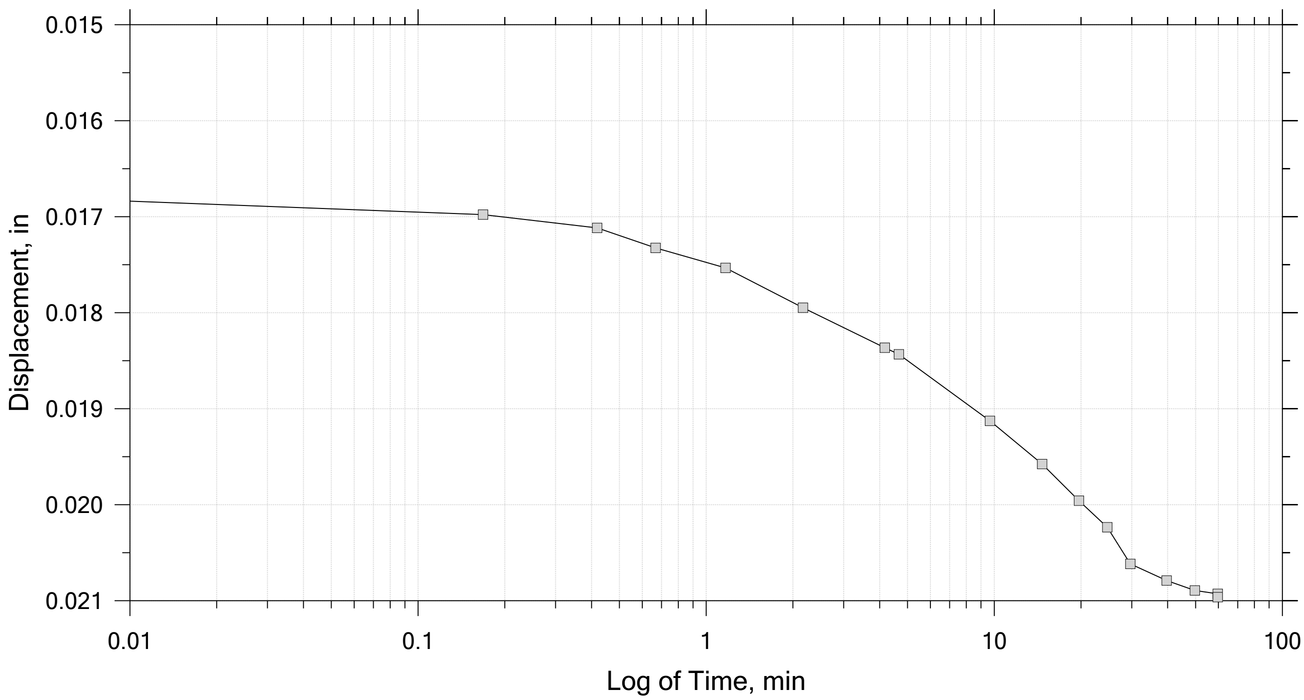
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

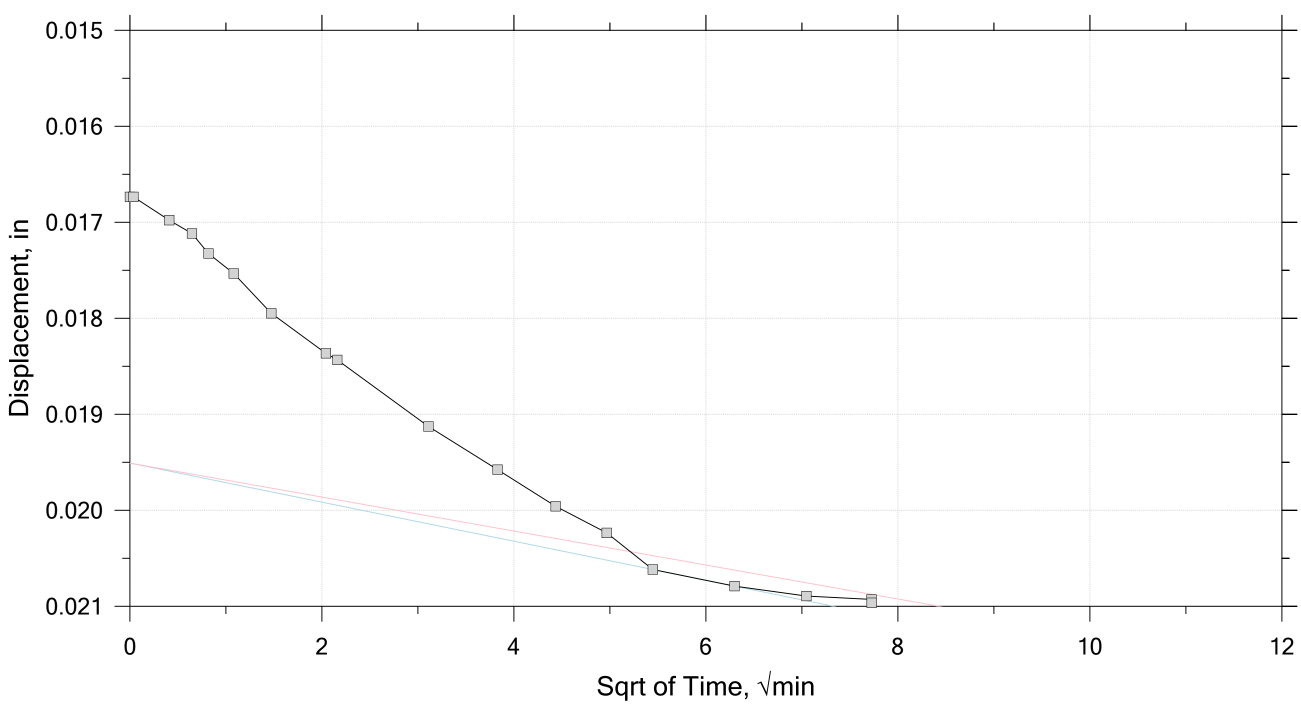
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 22

Constant Load Step

Stress: 1.01e+03 psf





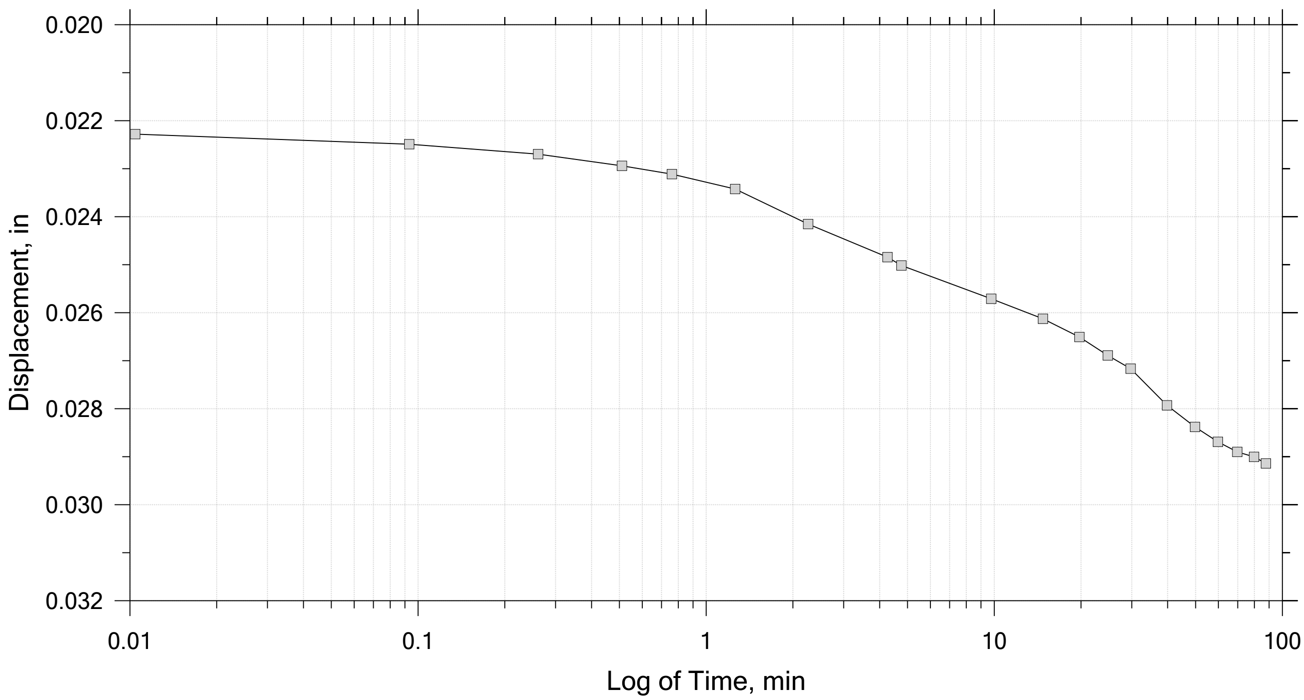
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

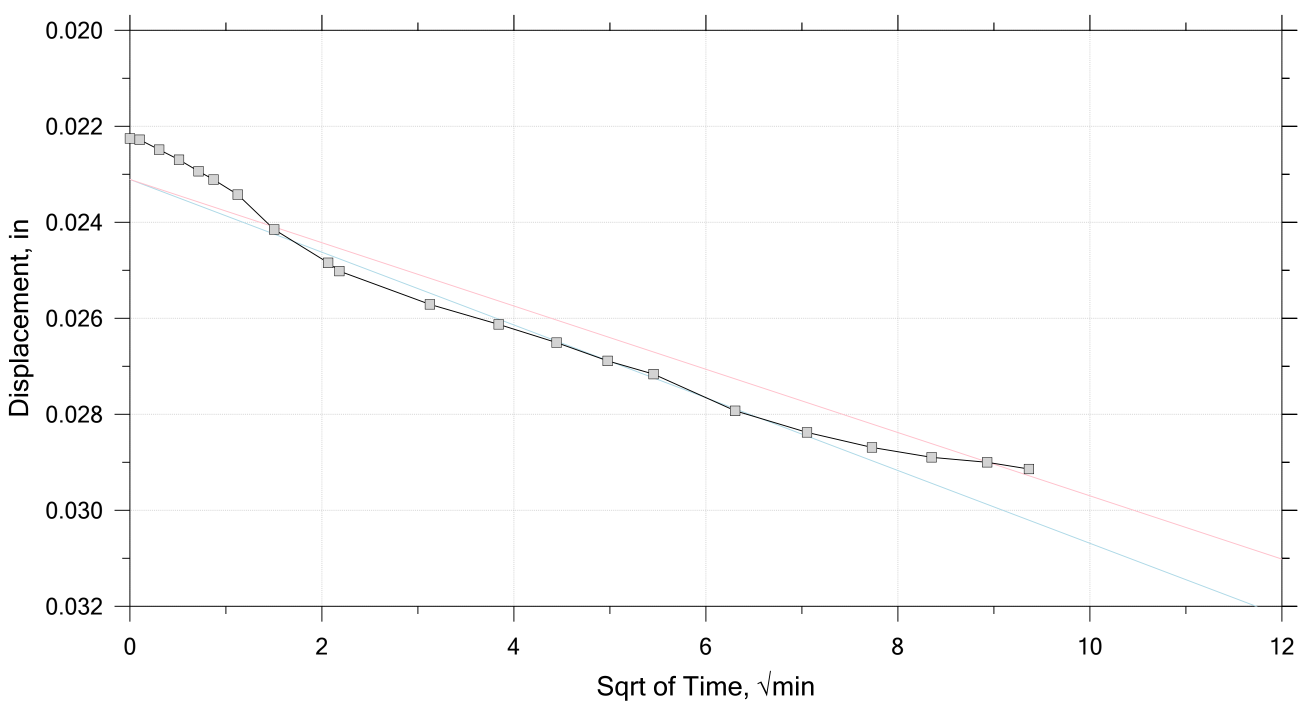
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 22

Constant Load Step

Stress: 1.52e+03 psf





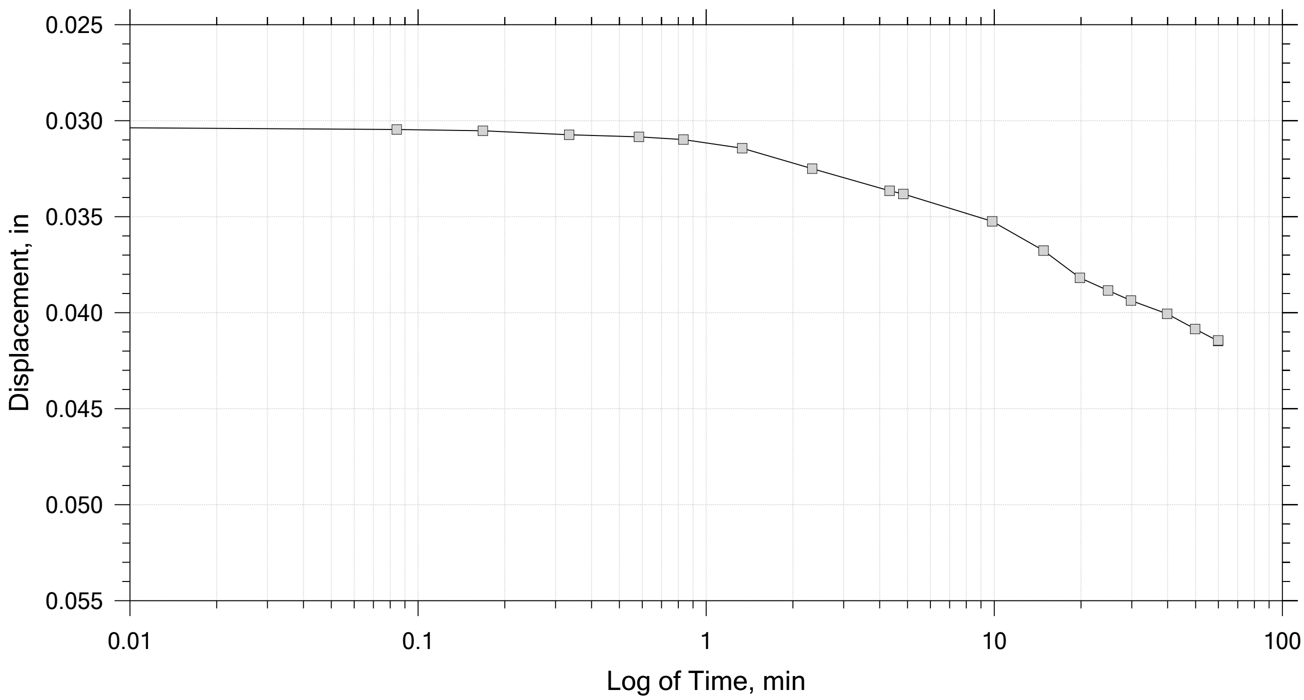
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

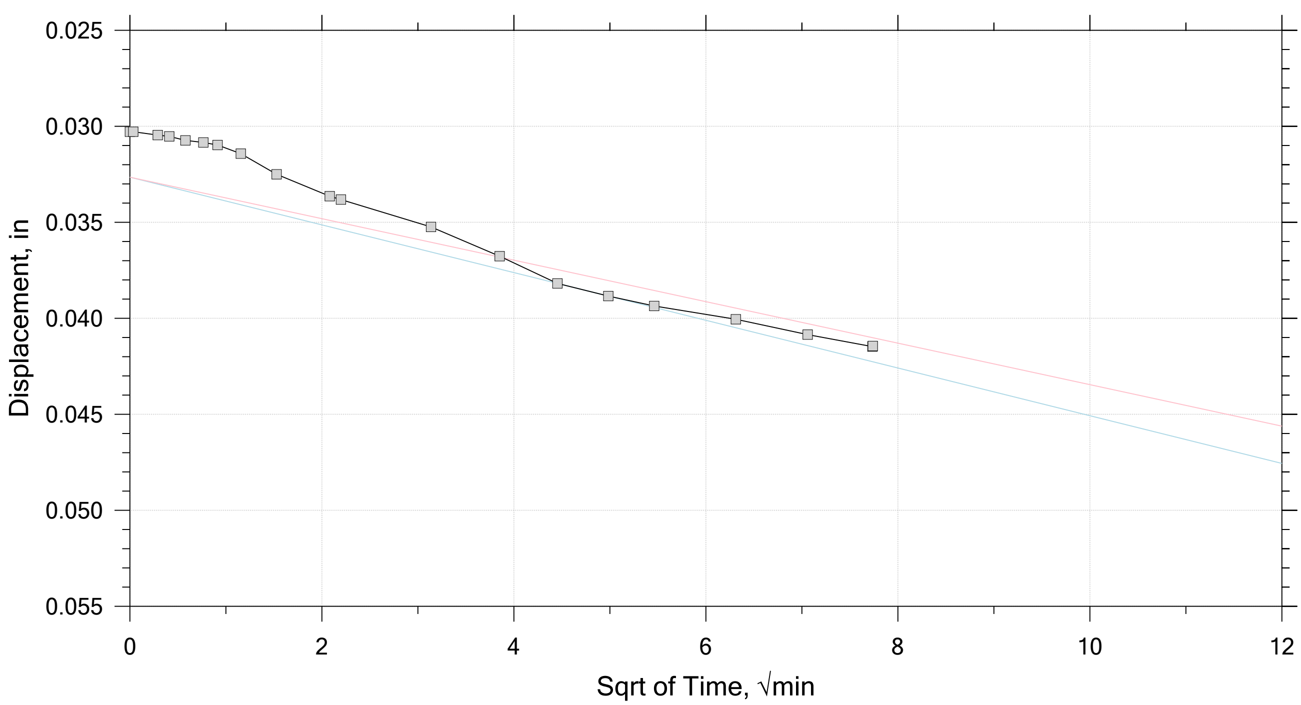
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 22

Constant Load Step

Stress: 2.28e+03 psf





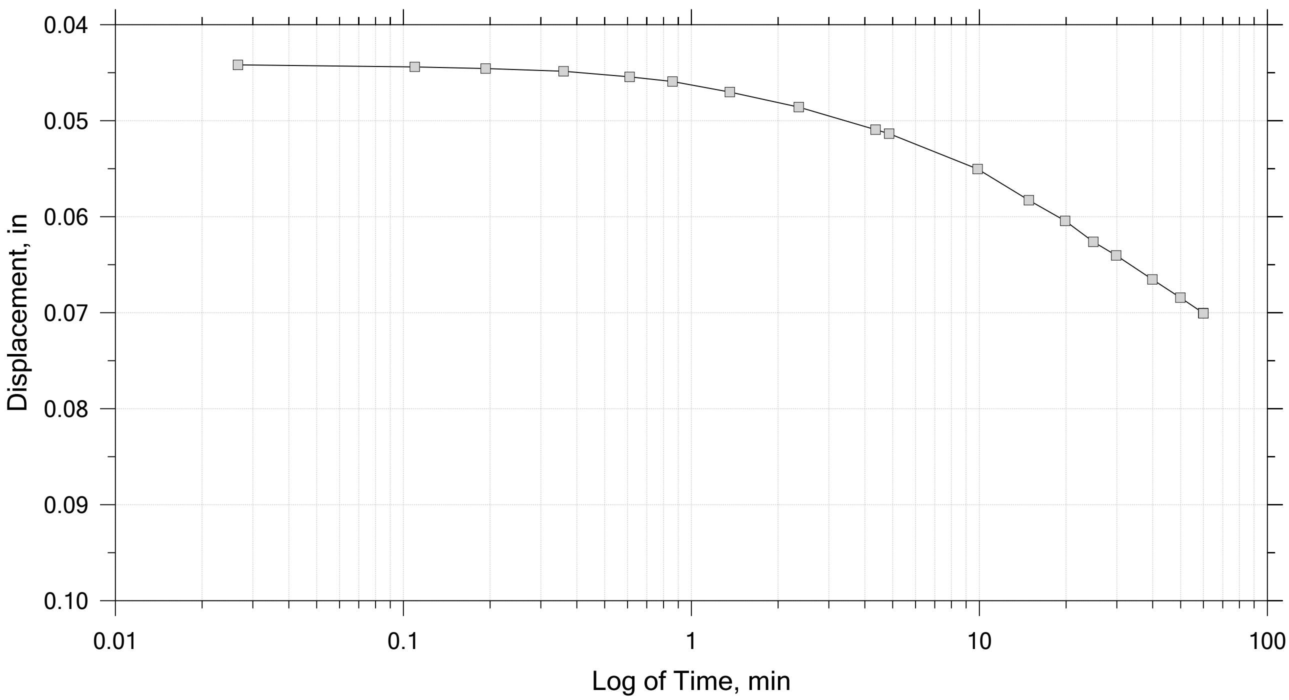
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

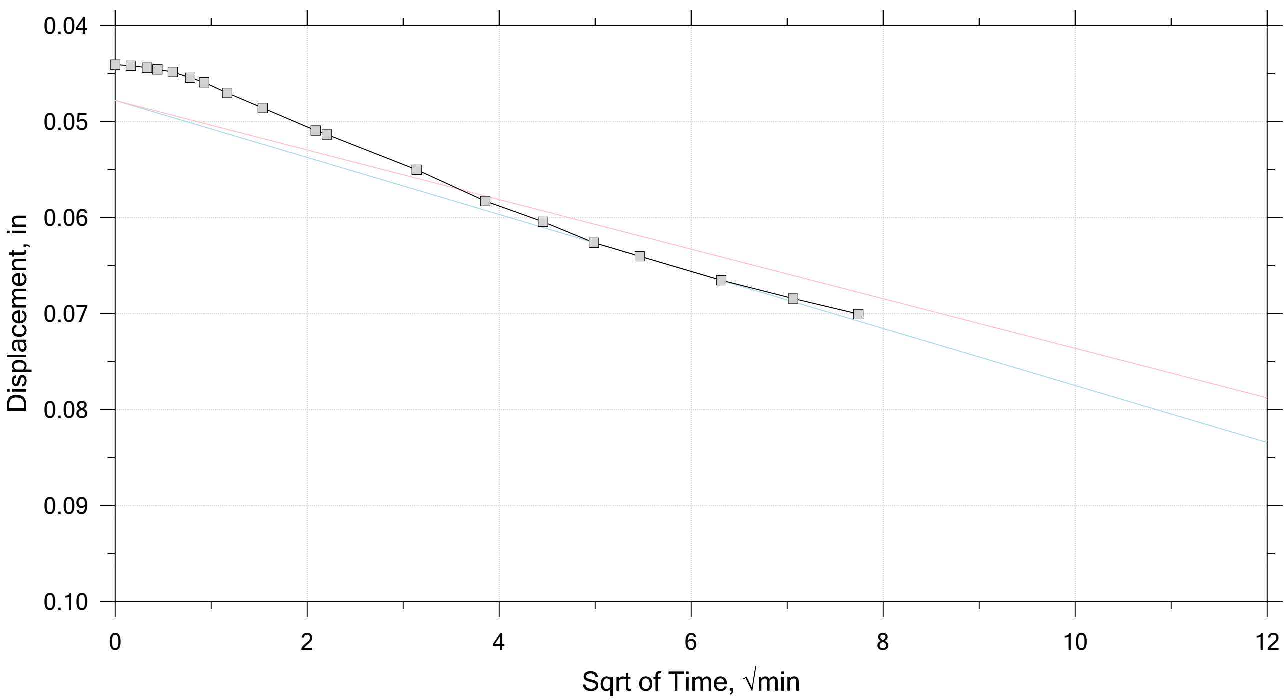
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 22

Constant Load Step

Stress: 3.42e+03 psf





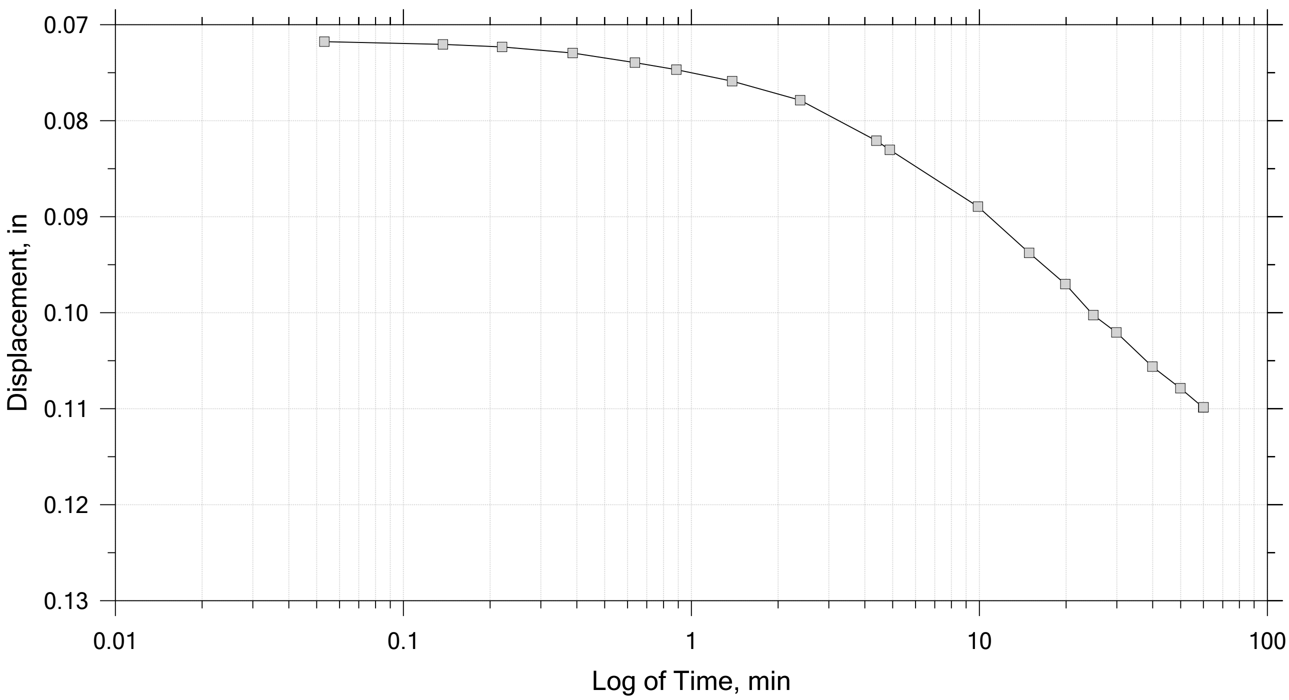
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

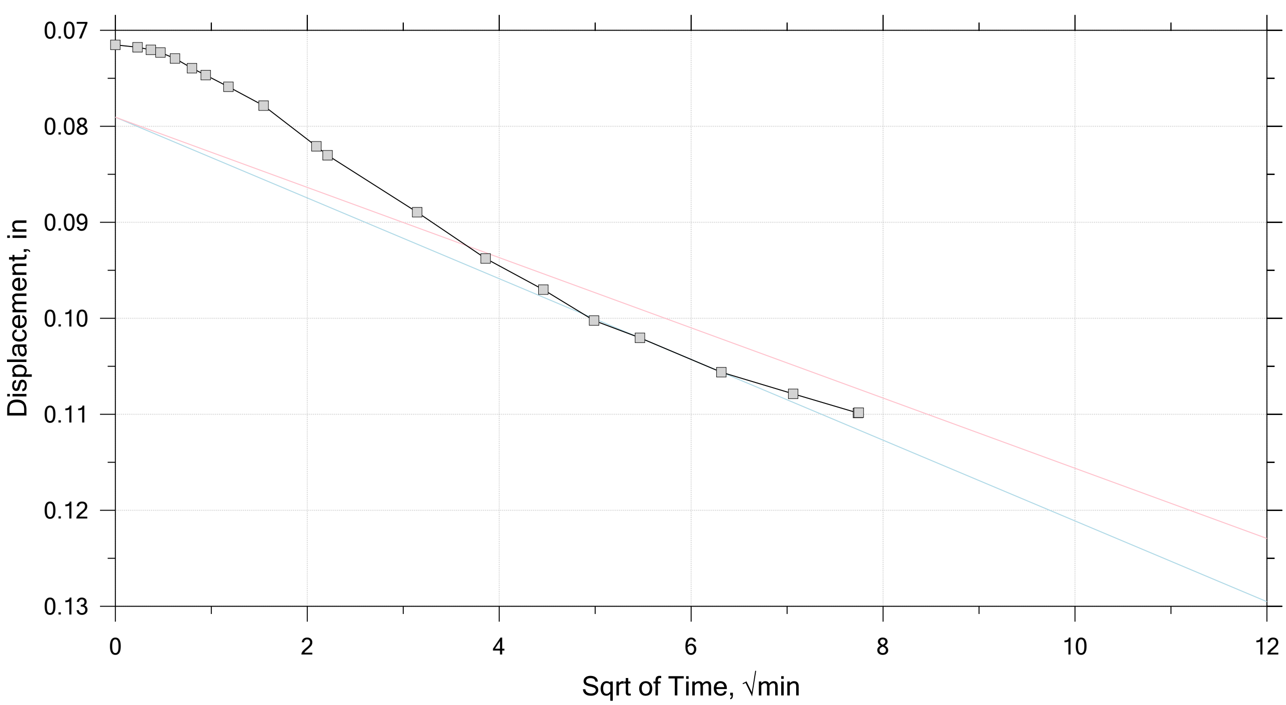
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 22

Constant Load Step

Stress: 5.13e+03 psf





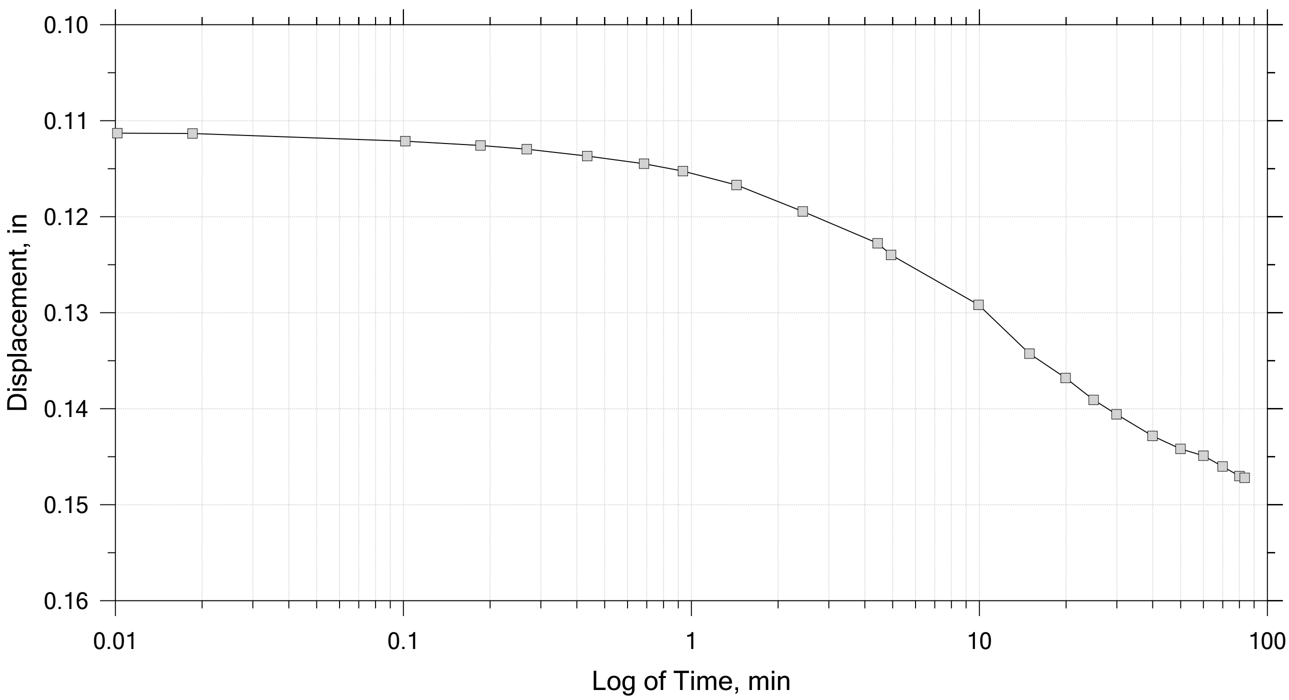
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

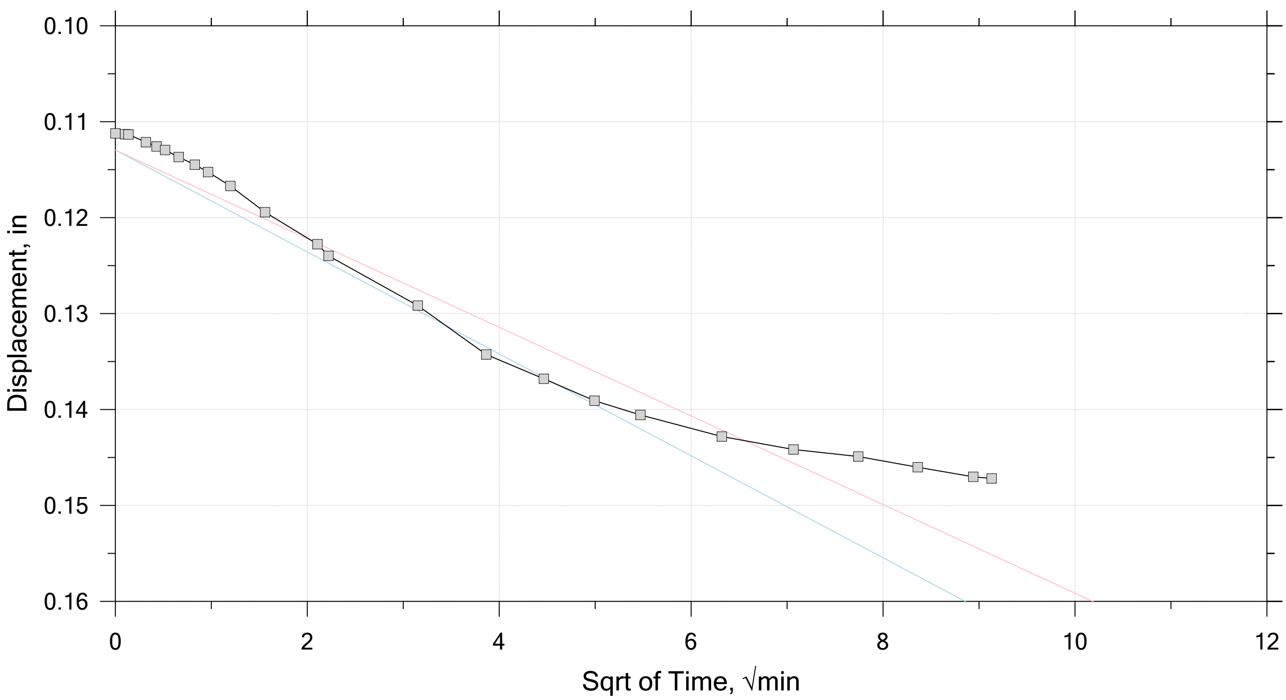
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 22

Constant Load Step

Stress: 7.69e+03 psf





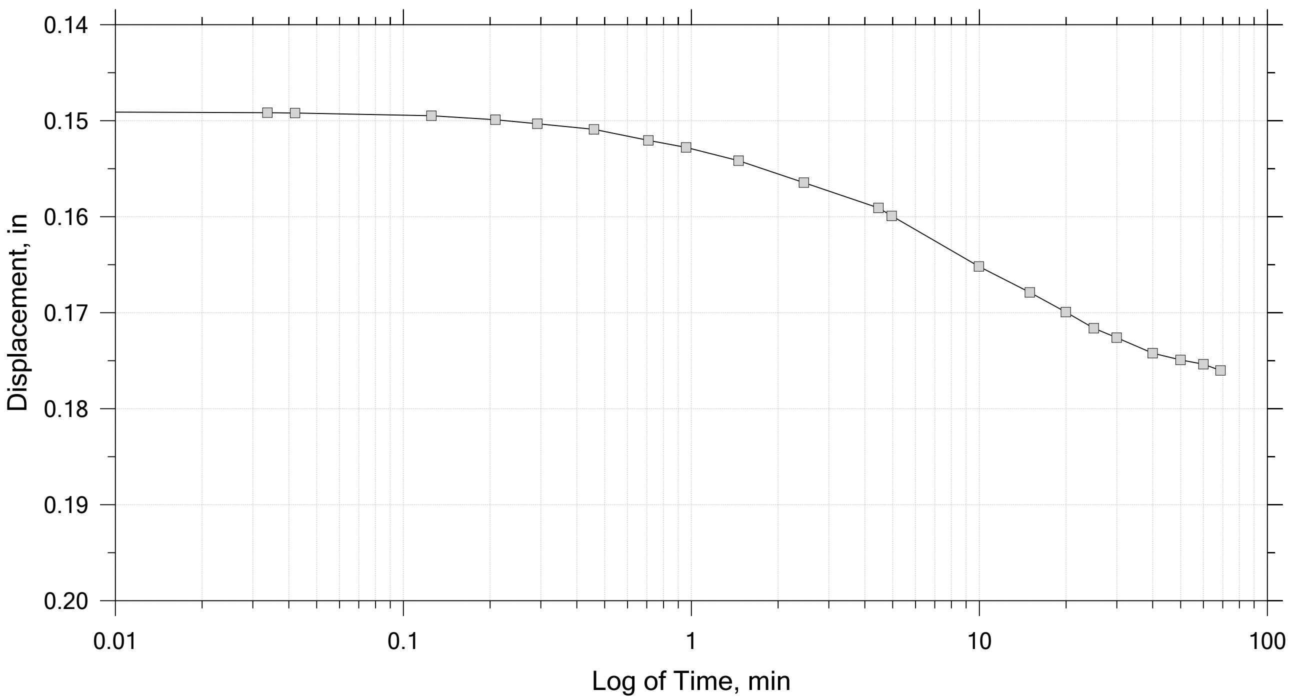
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

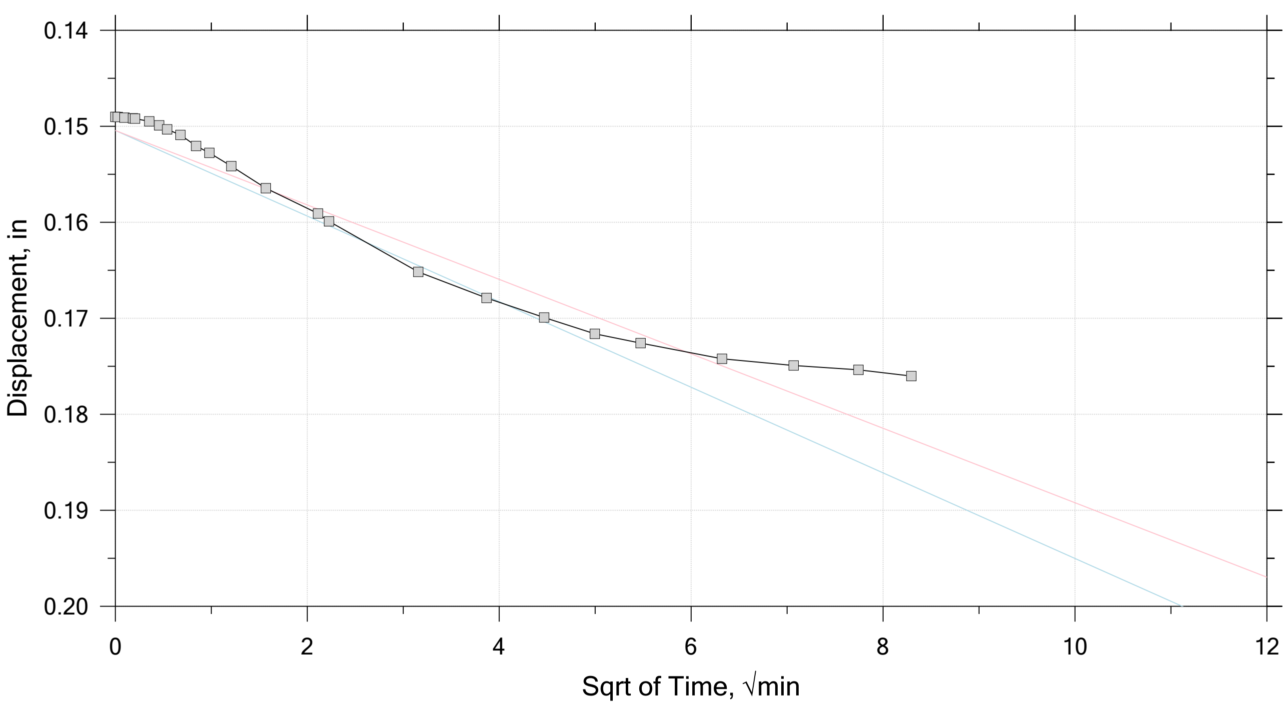
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 22

Constant Load Step

Stress: 1.16e+04 psf





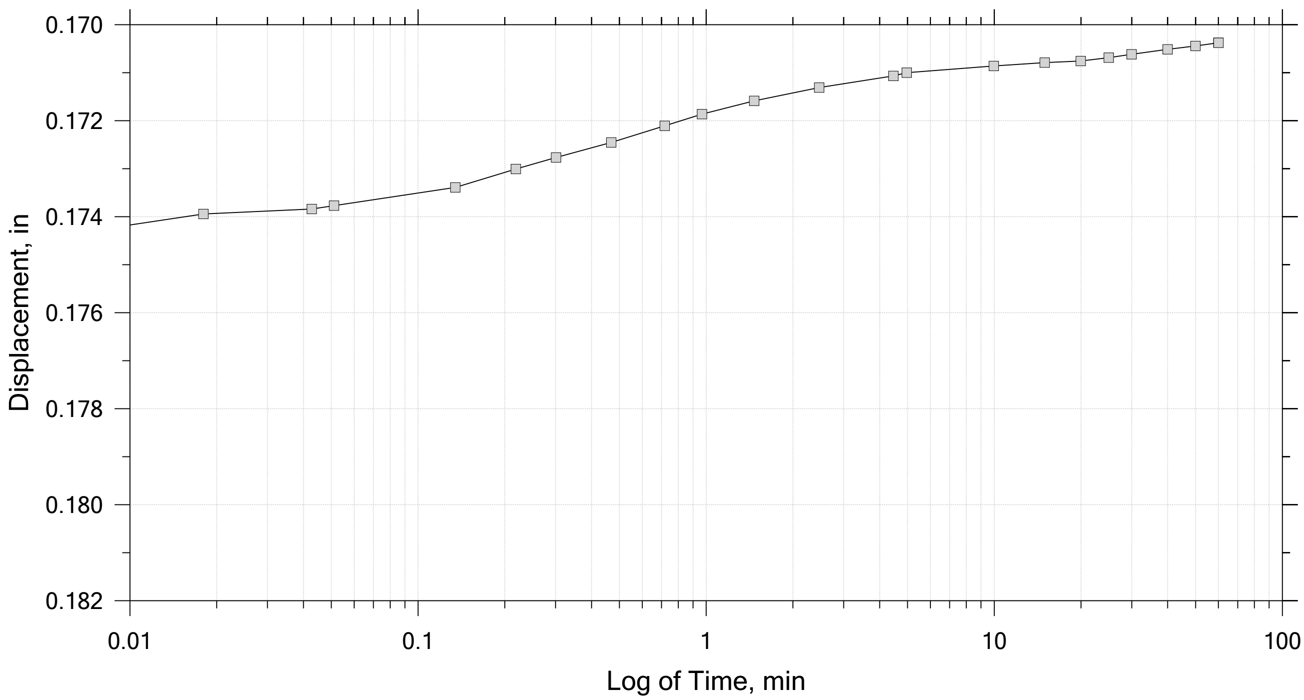
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

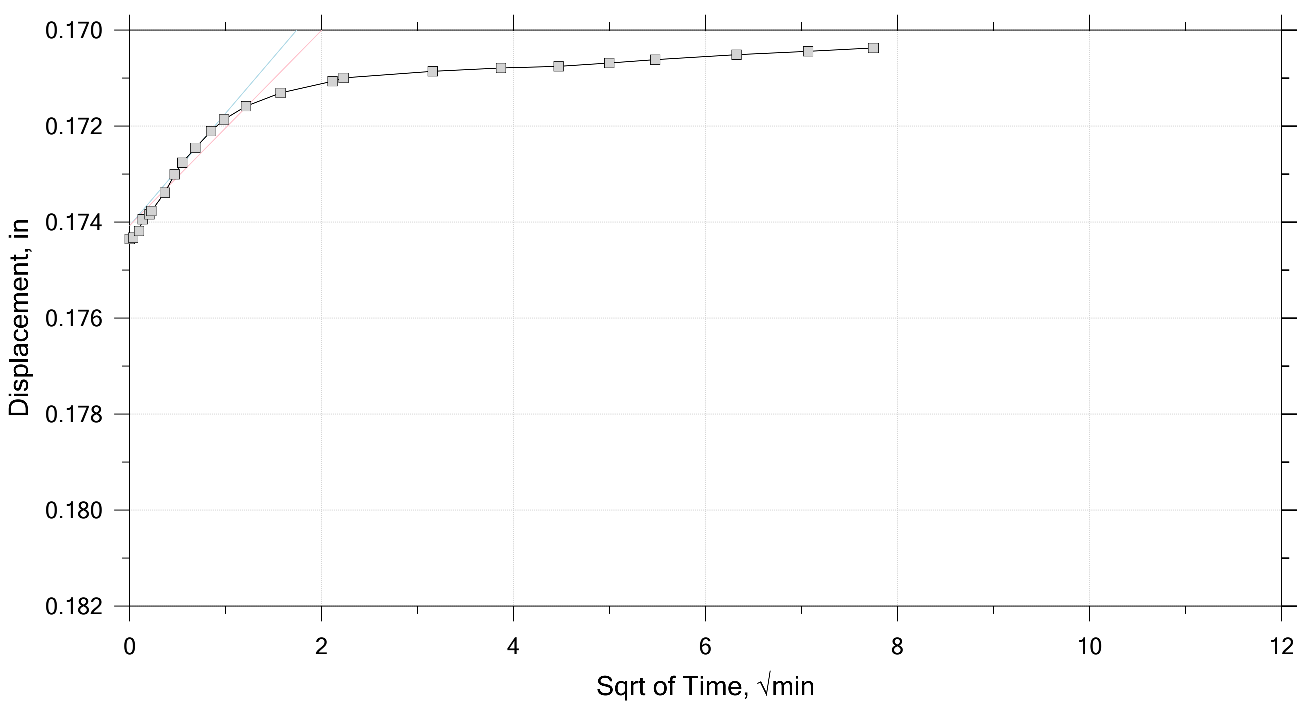
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 22

Constant Load Step

Stress: 5.12e+03 psf

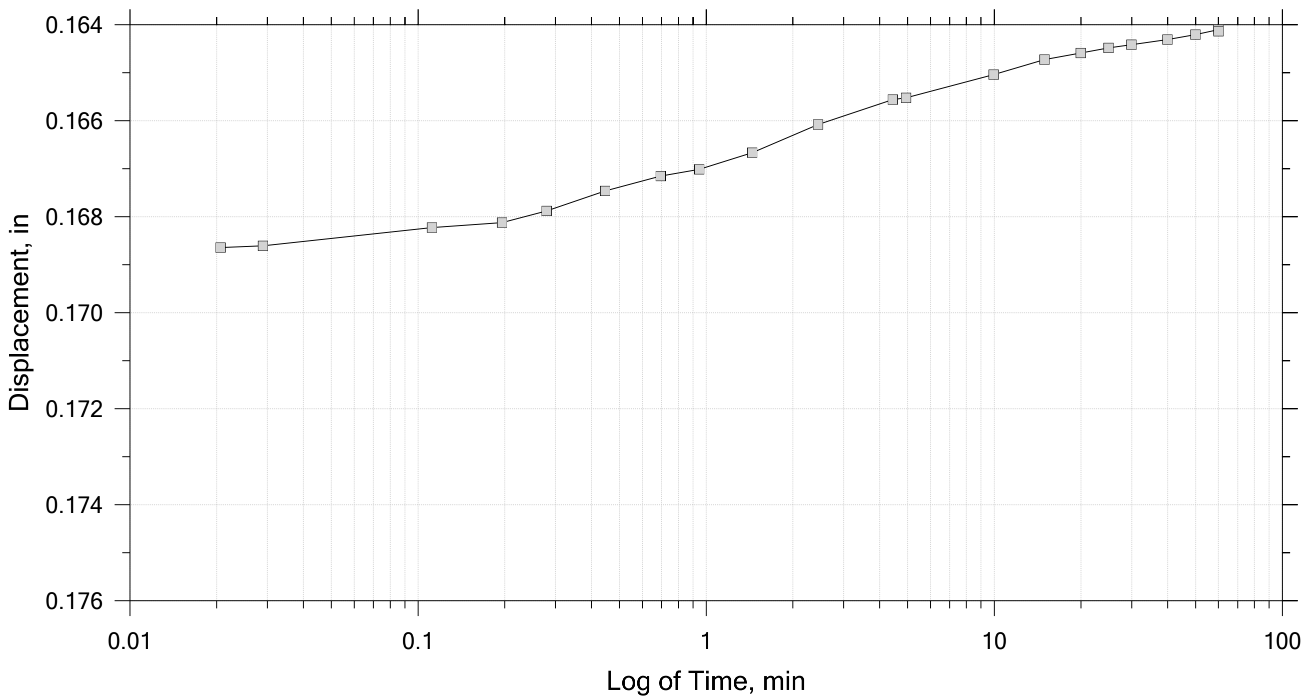


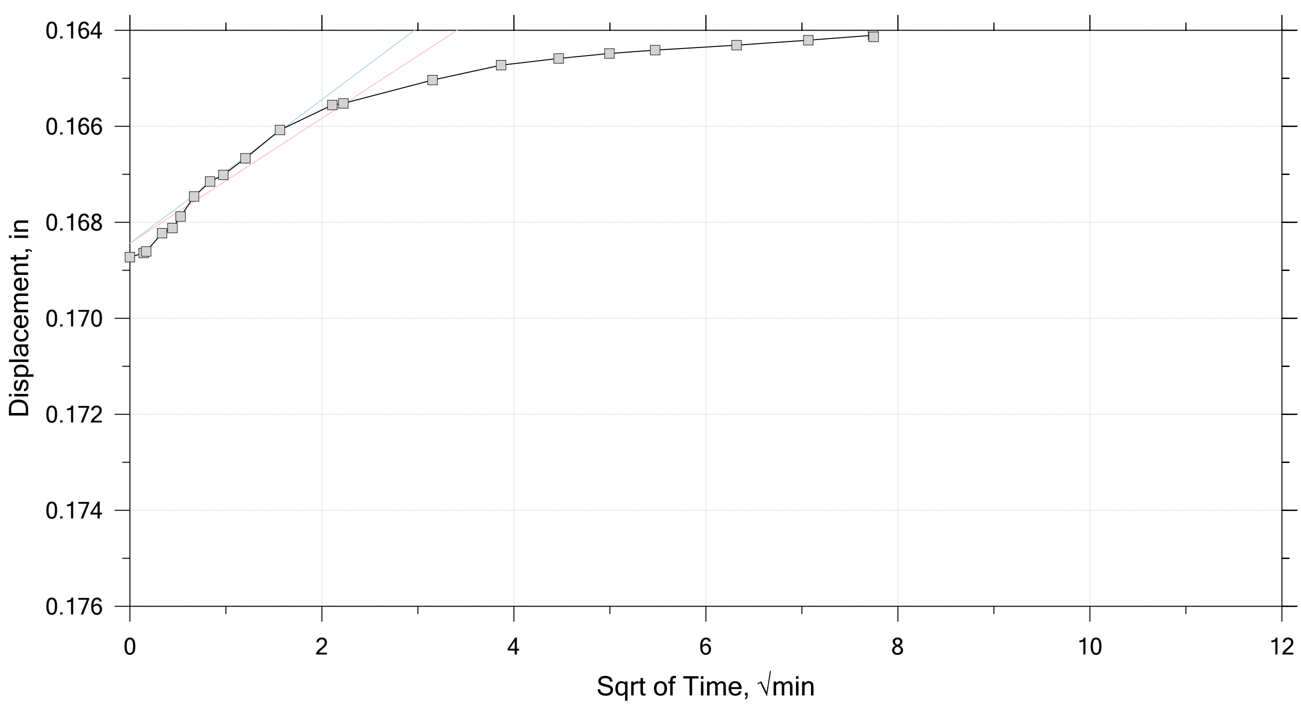


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 22
Constant Load Step
Stress: 2.28e+03 psf

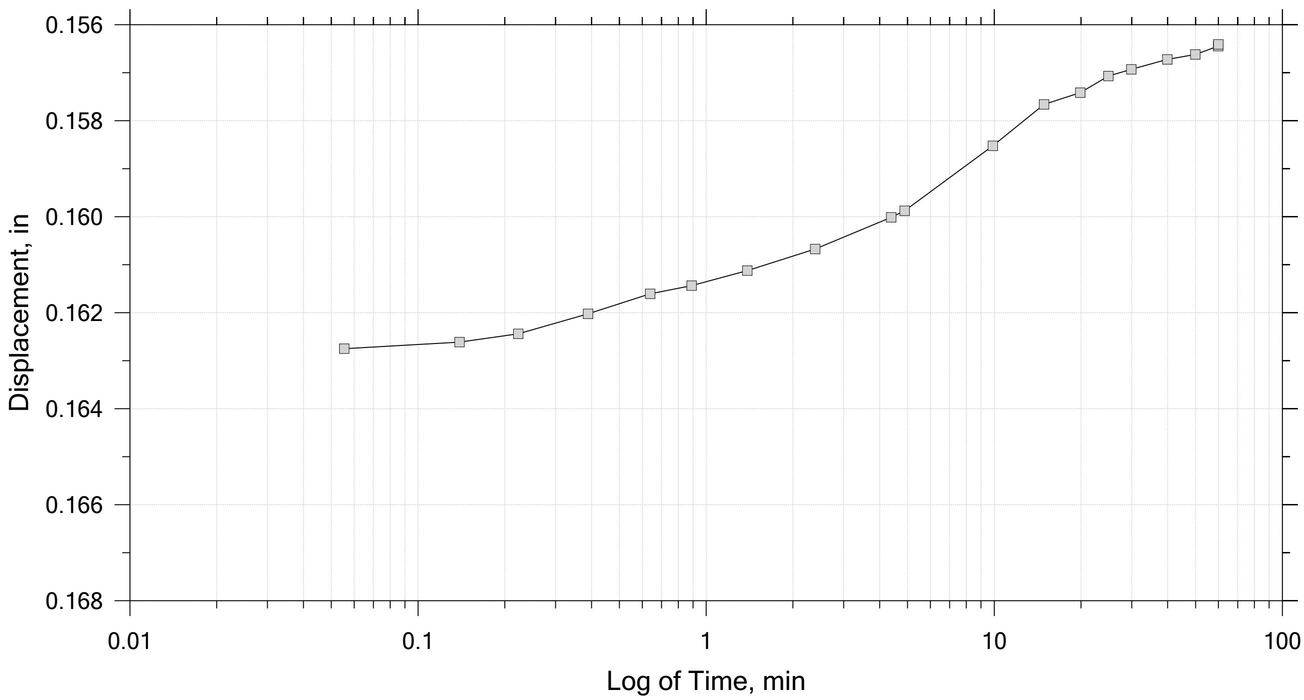


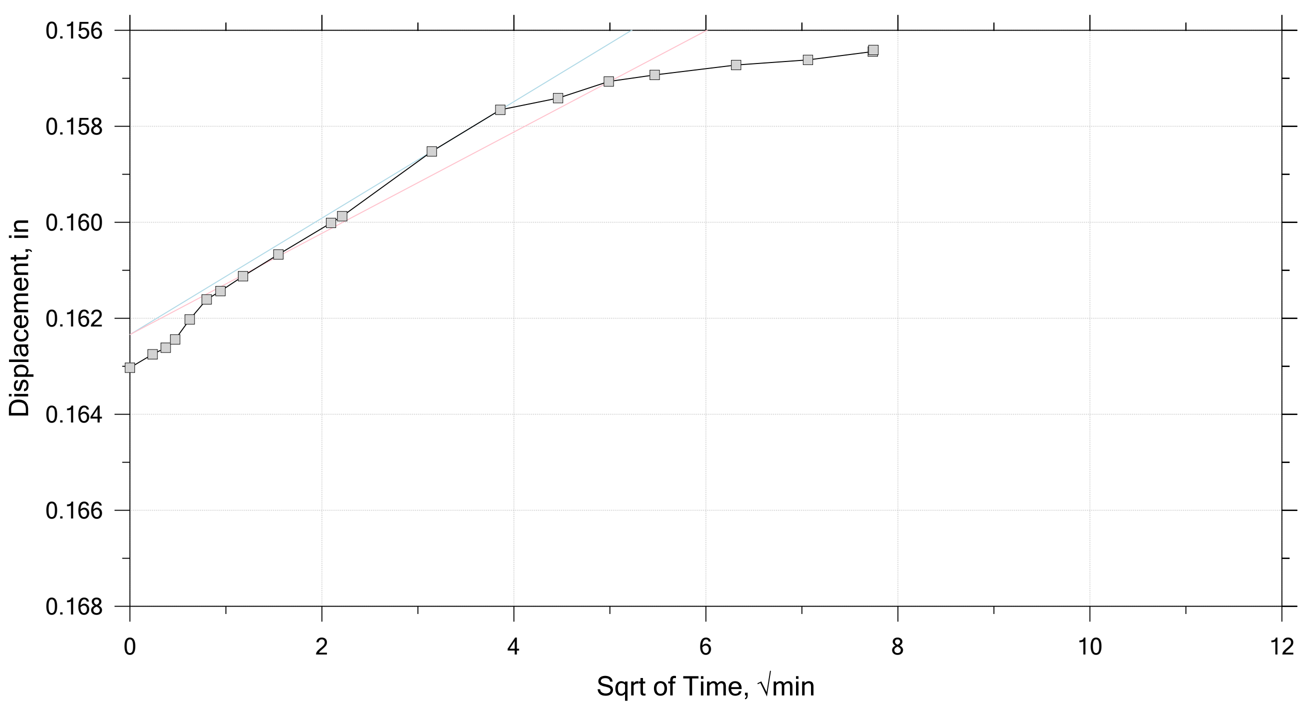


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 22
Constant Load Step
Stress: 1.01e+03 psf





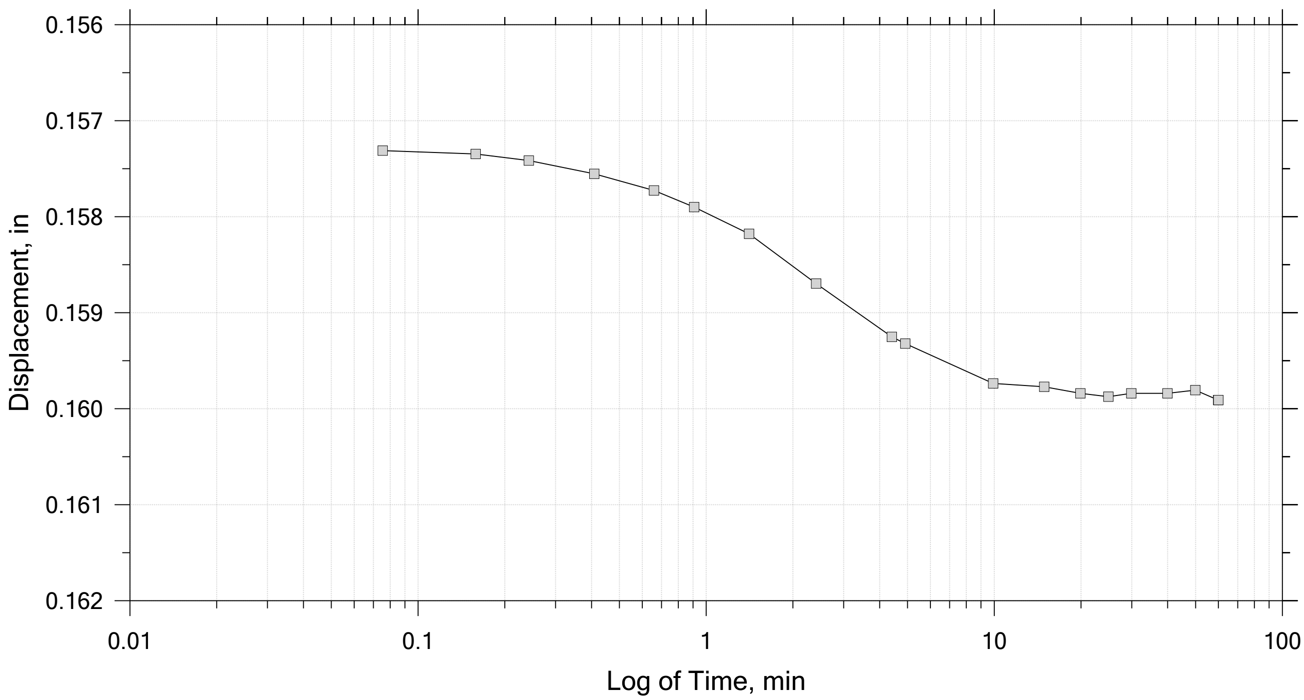
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

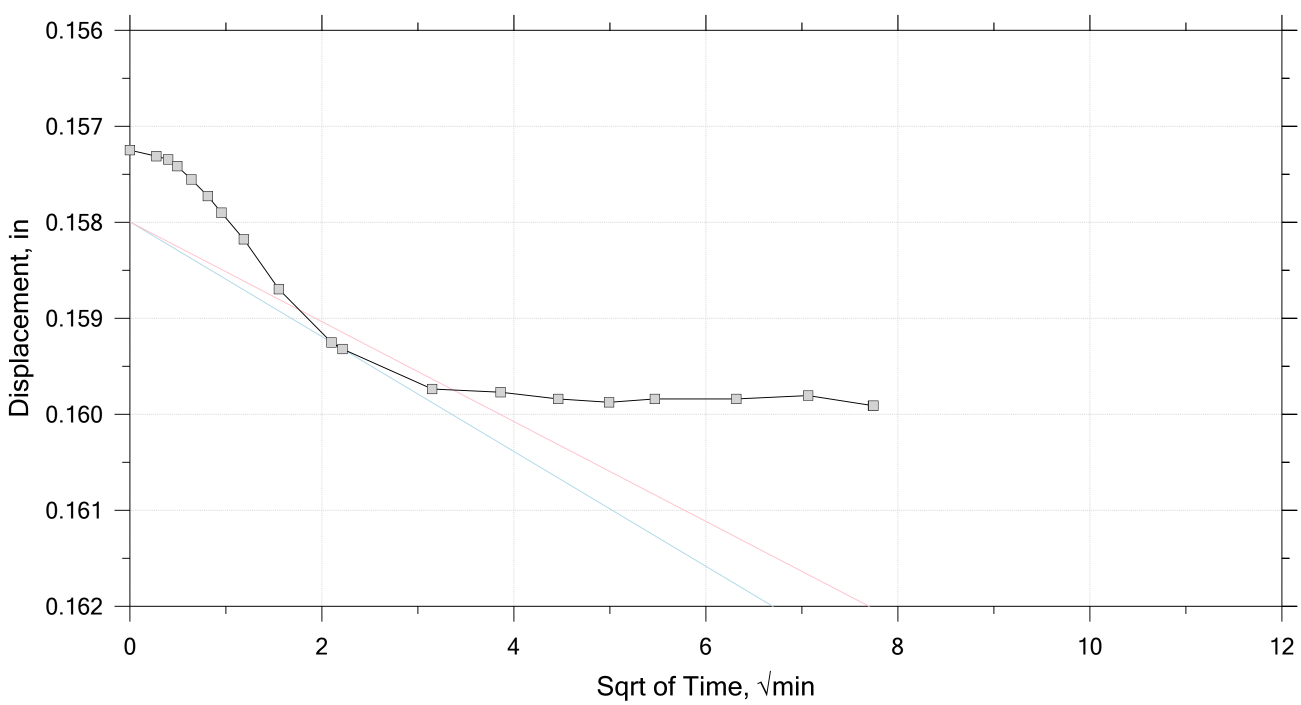
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 22

Constant Load Step

Stress: 2.28e+03 psf

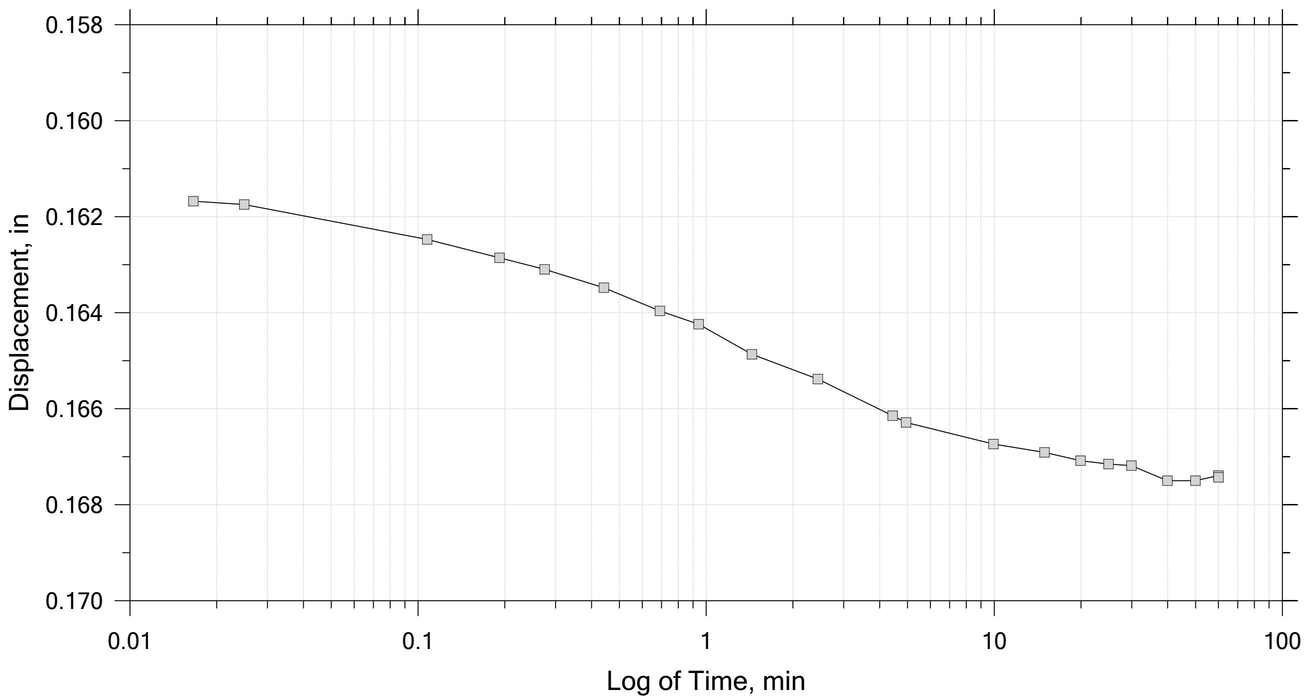


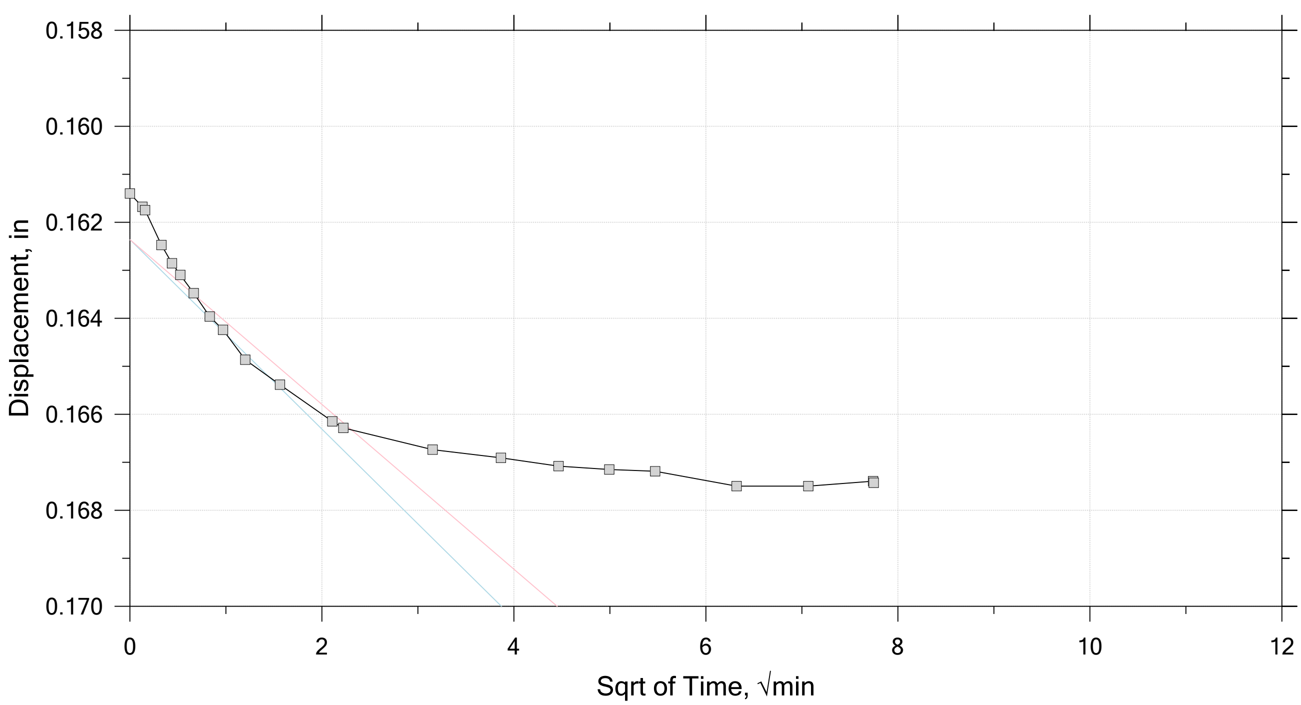


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 22
Constant Load Step
Stress: 5.13e+03 psf

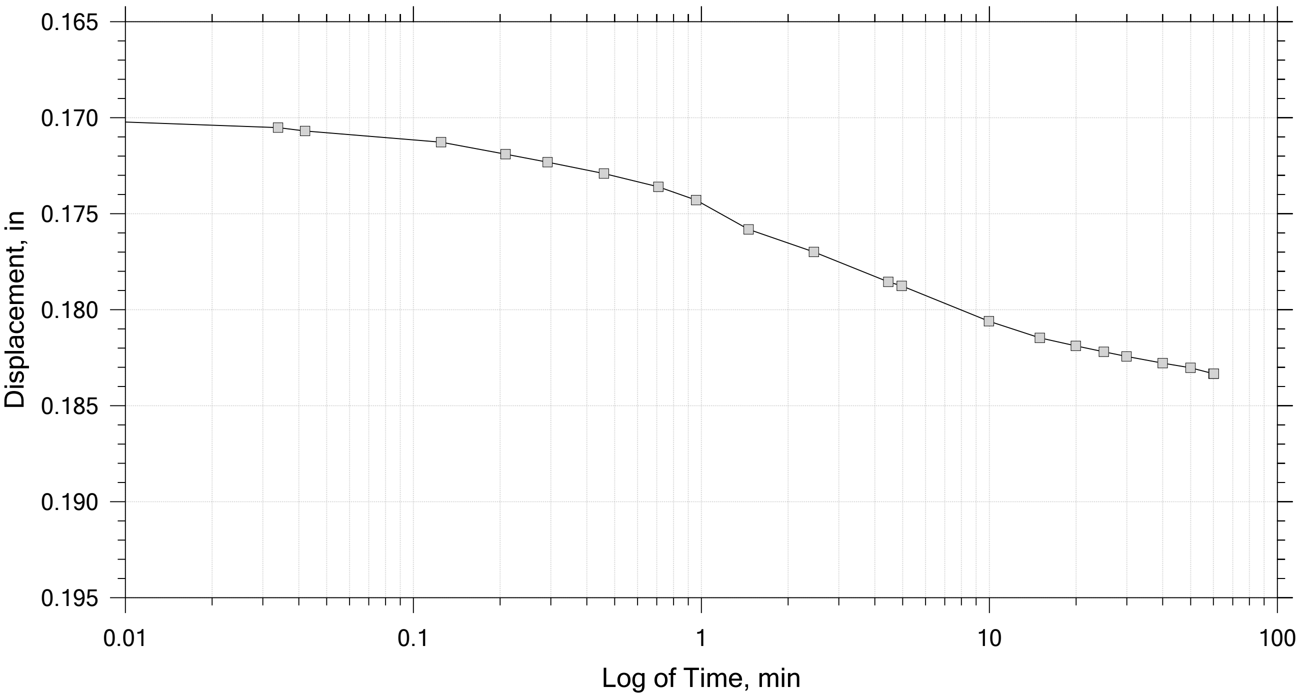


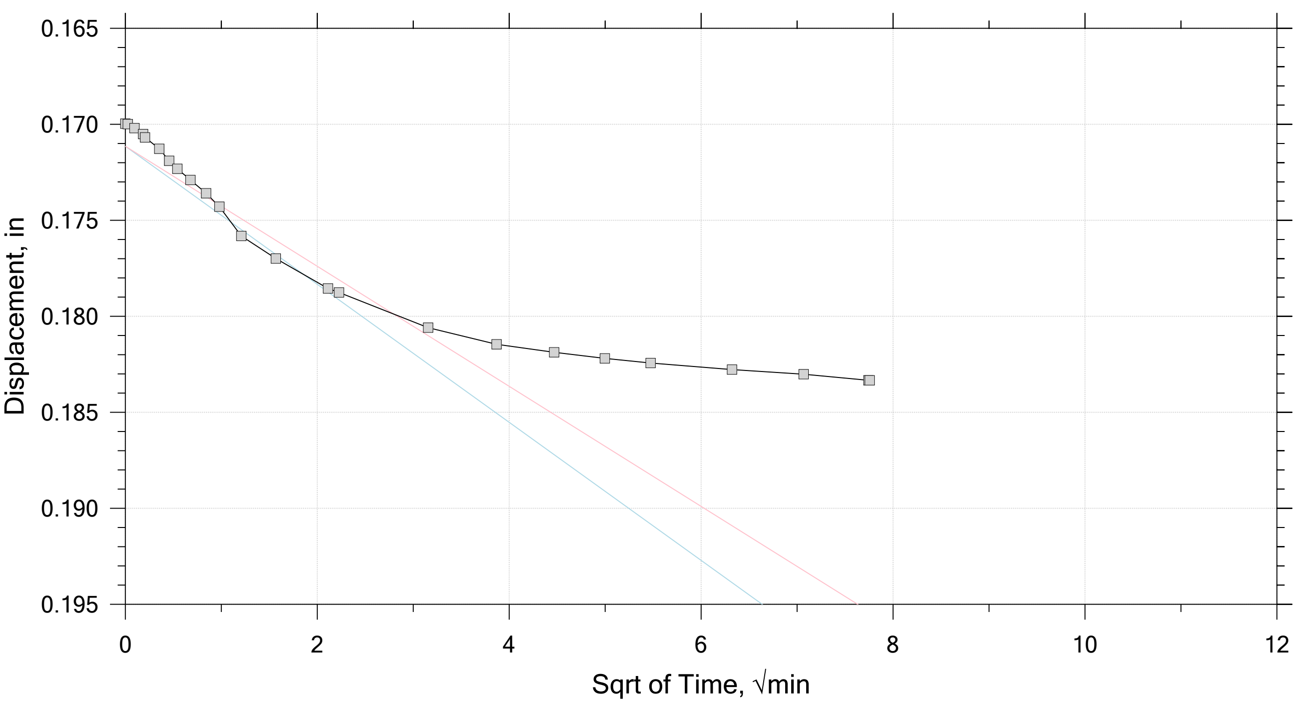


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 22
Constant Load Step
Stress: 1.15e+04 psf

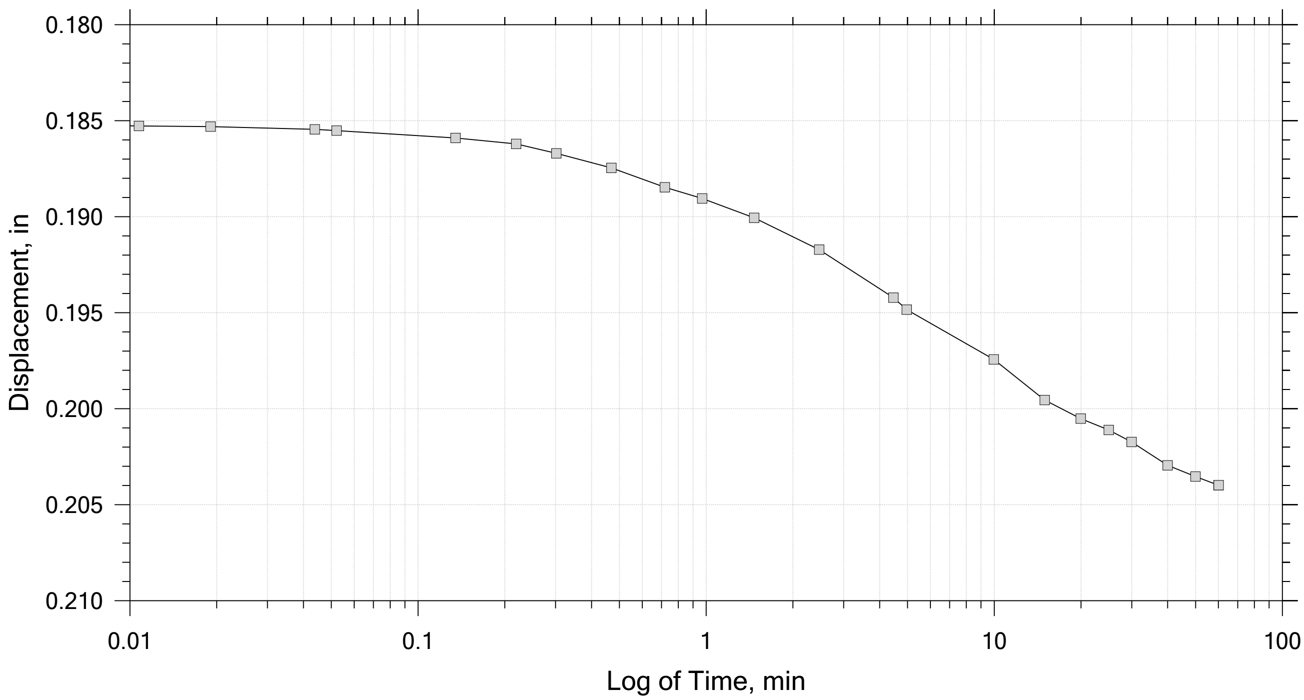


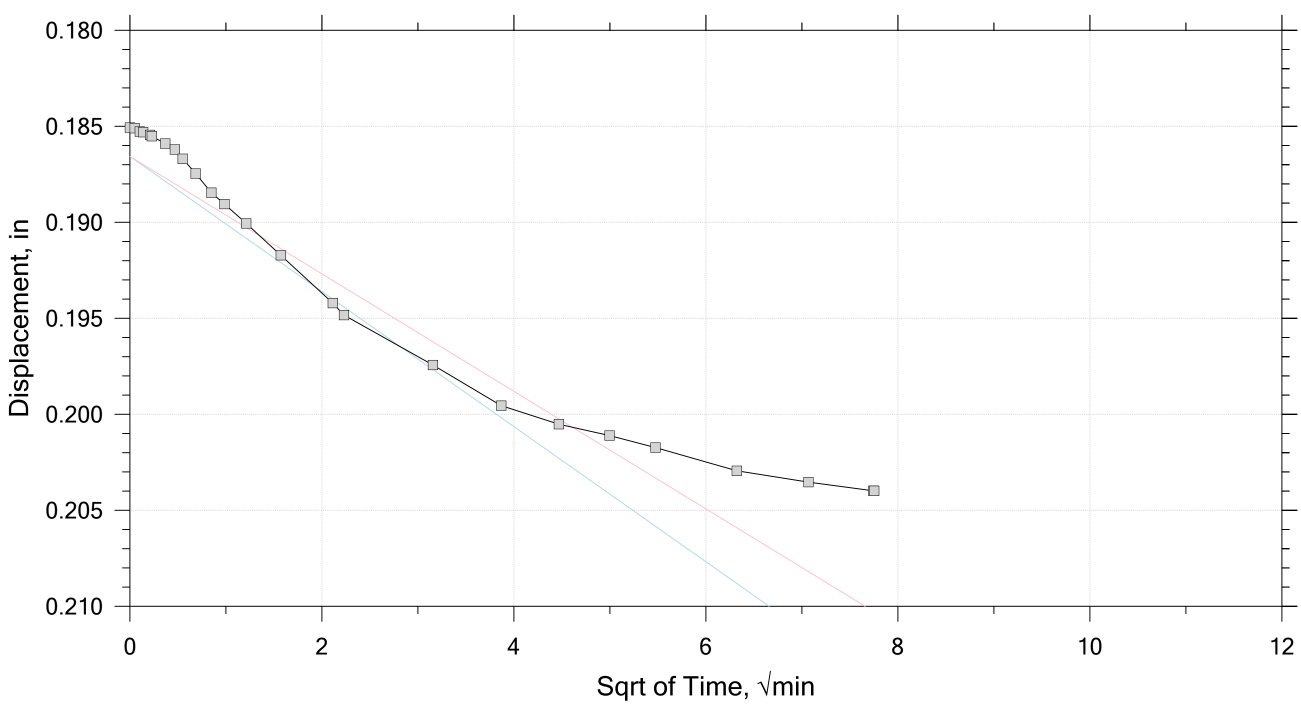


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 18 of 22
Constant Load Step
Stress: 1.73e+04 psf





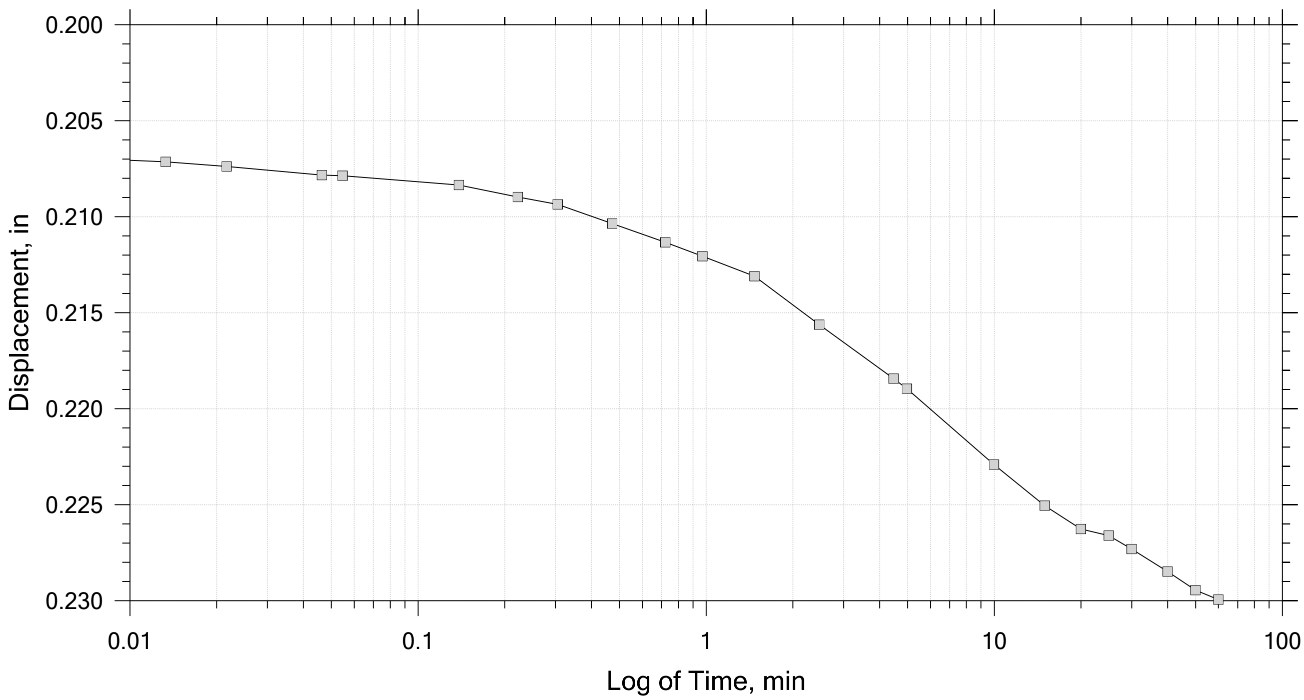
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

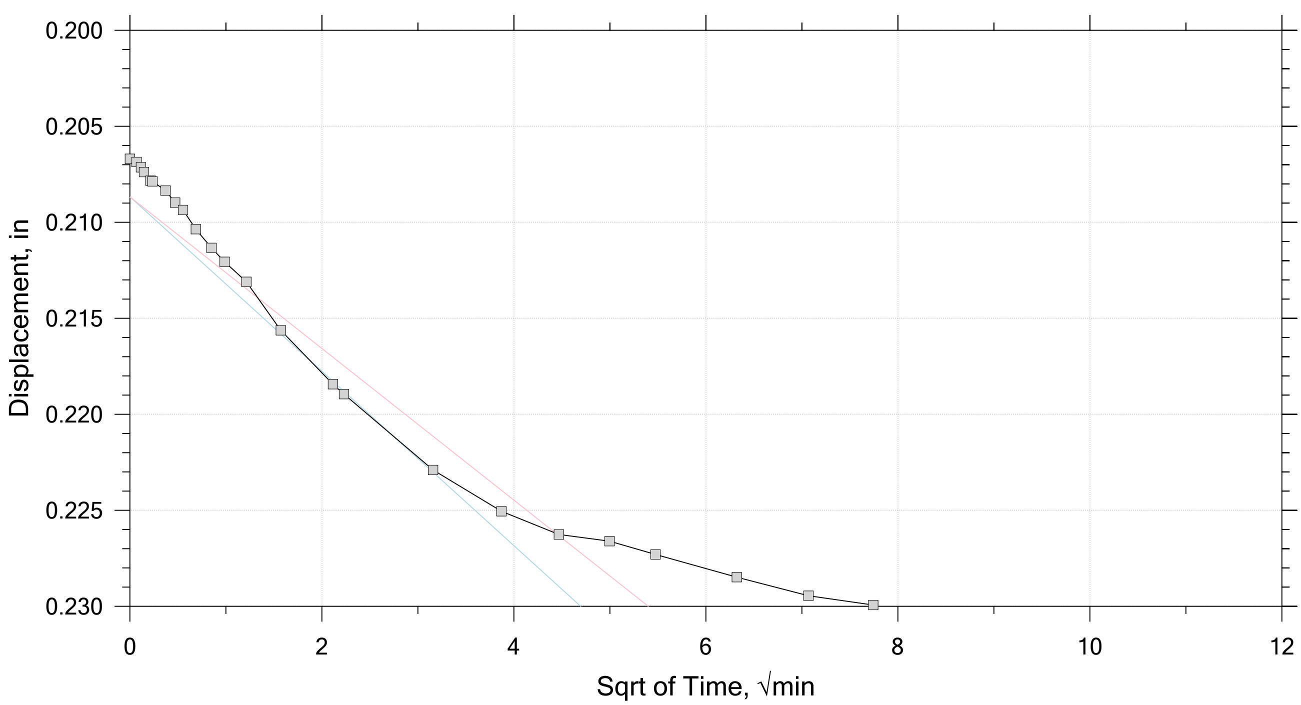
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 19 of 22

Constant Load Step

Stress: 2.6e+04 psf

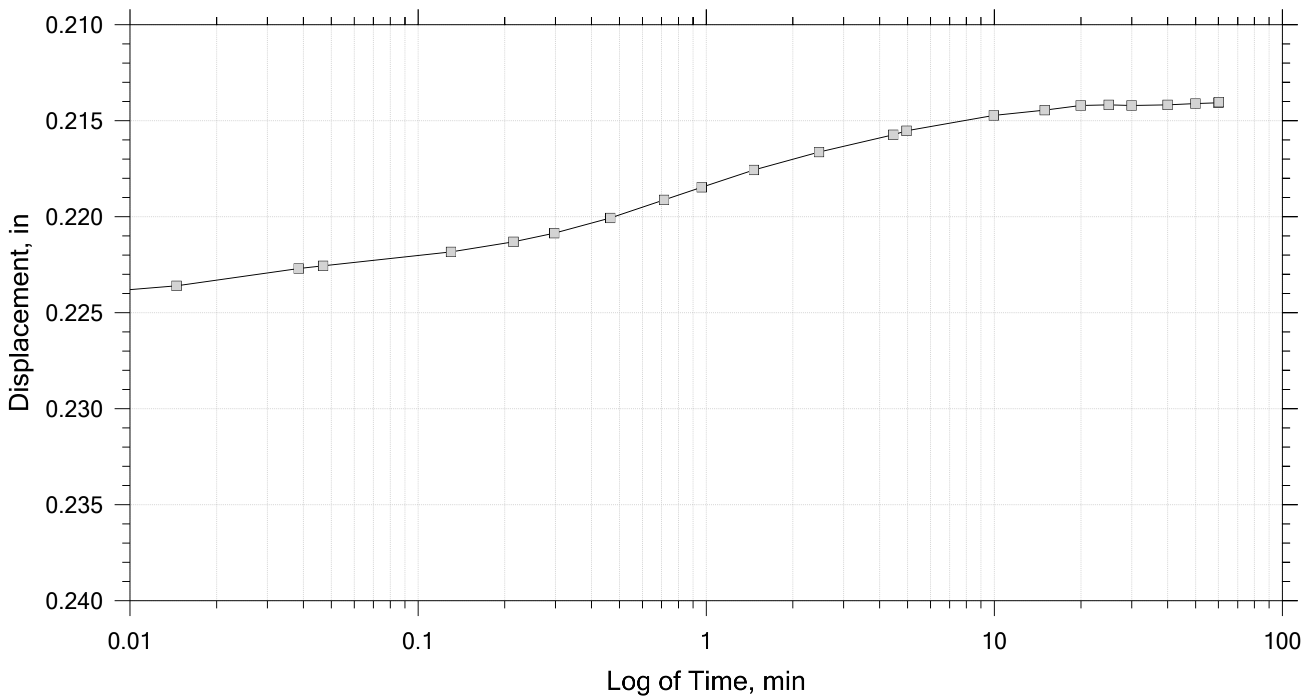


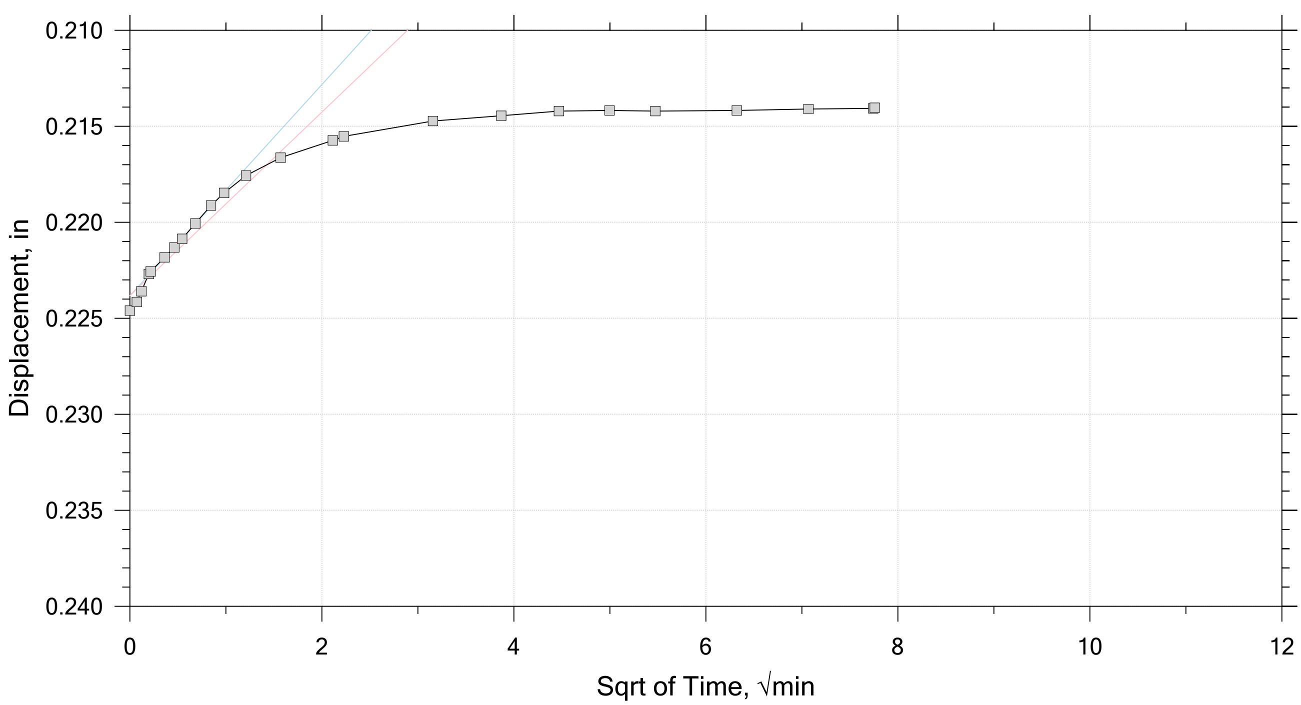


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 20 of 22
Constant Load Step
Stress: 5.13e+03 psf





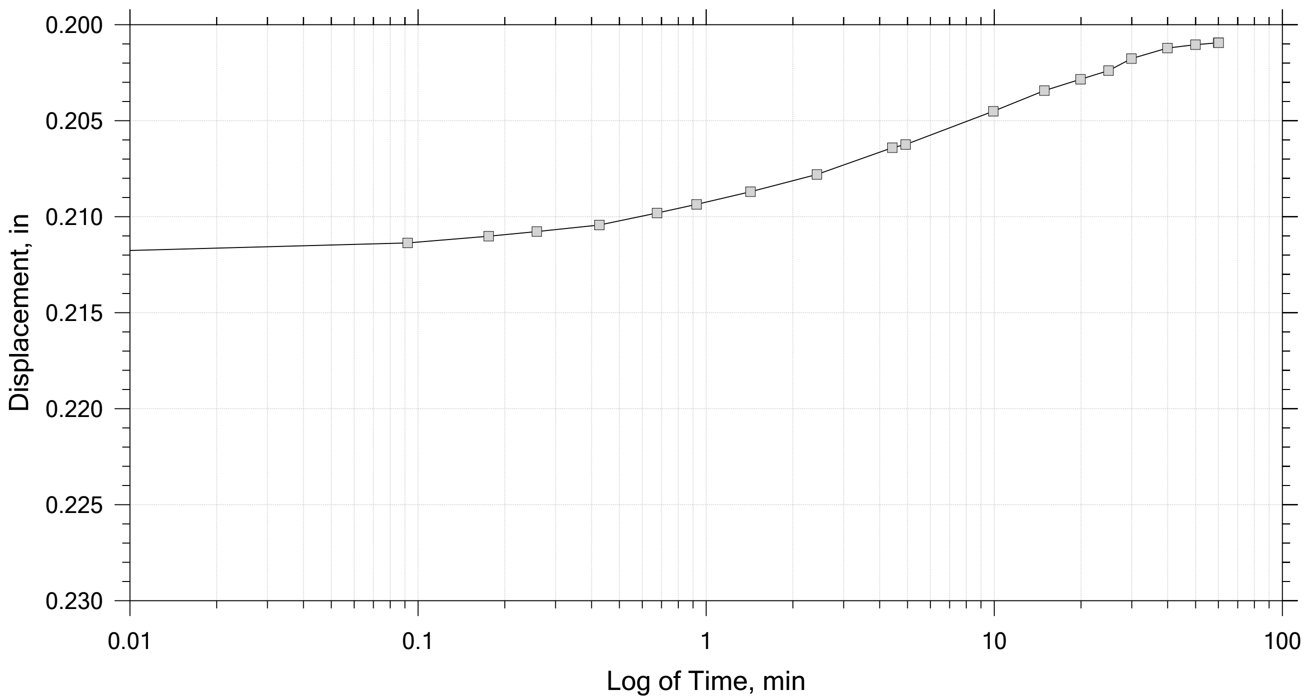
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

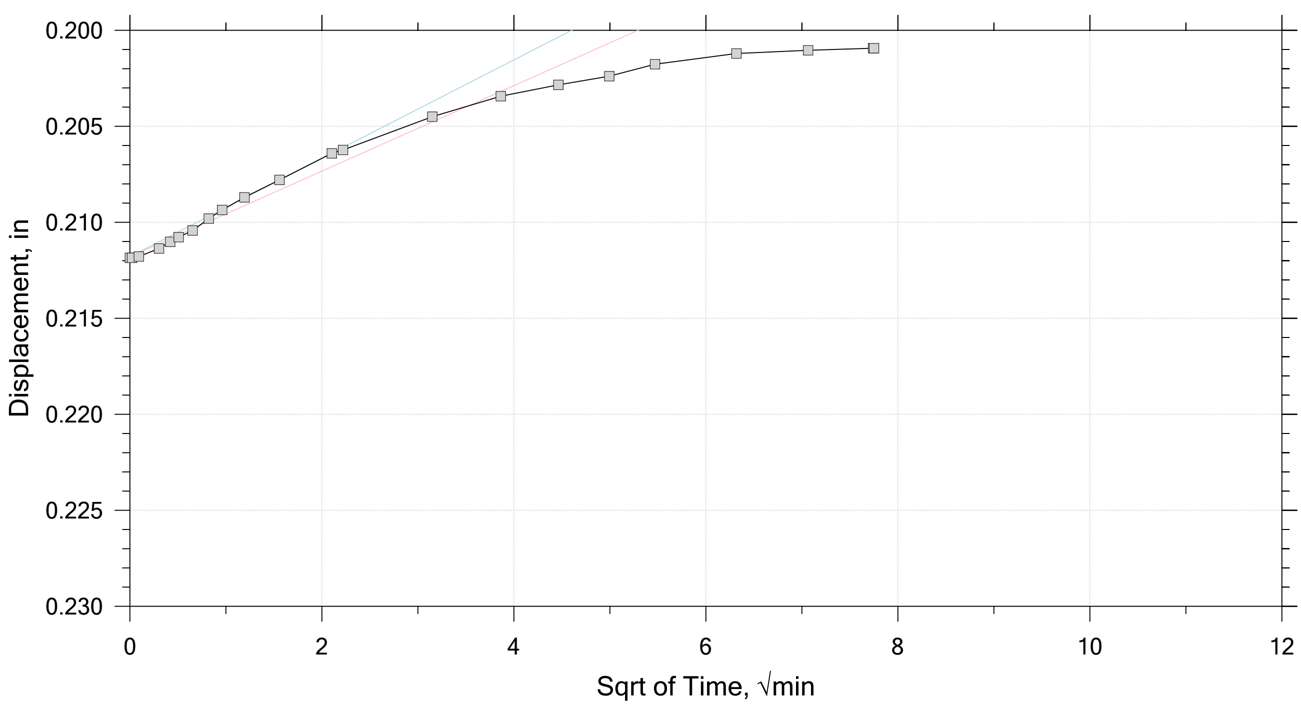
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 21 of 22

Constant Load Step

Stress: 1.52e+03 psf





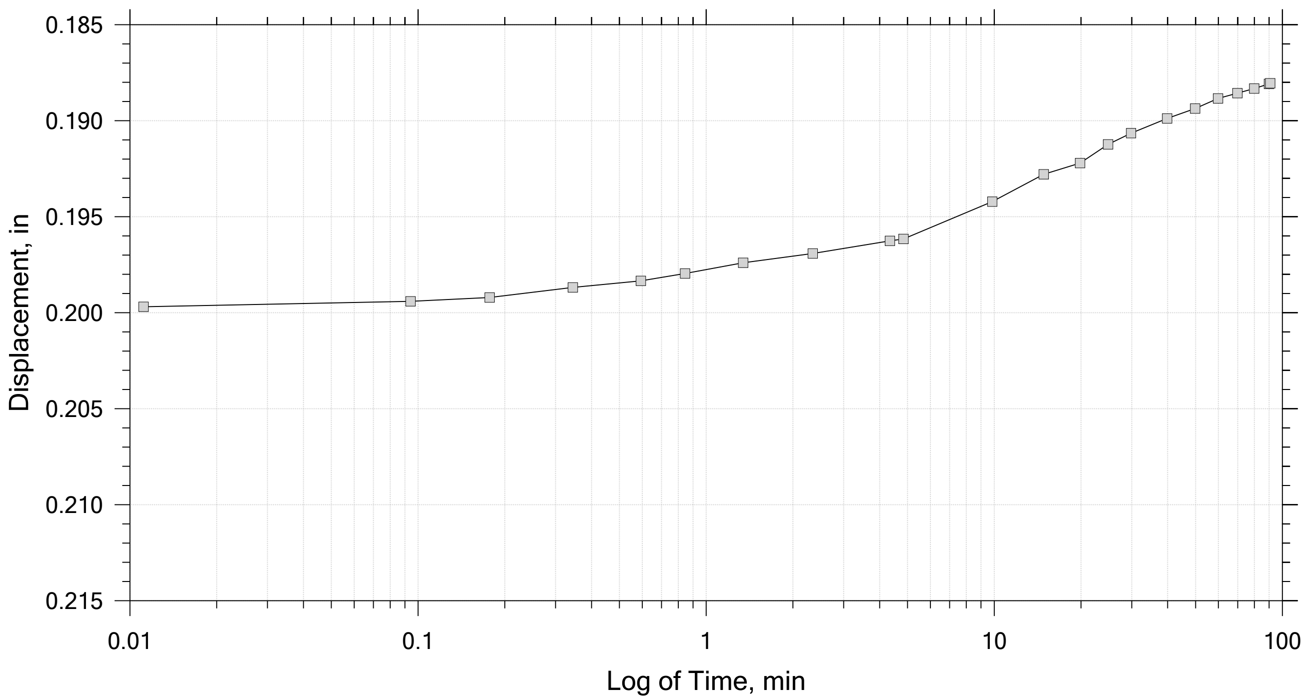
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

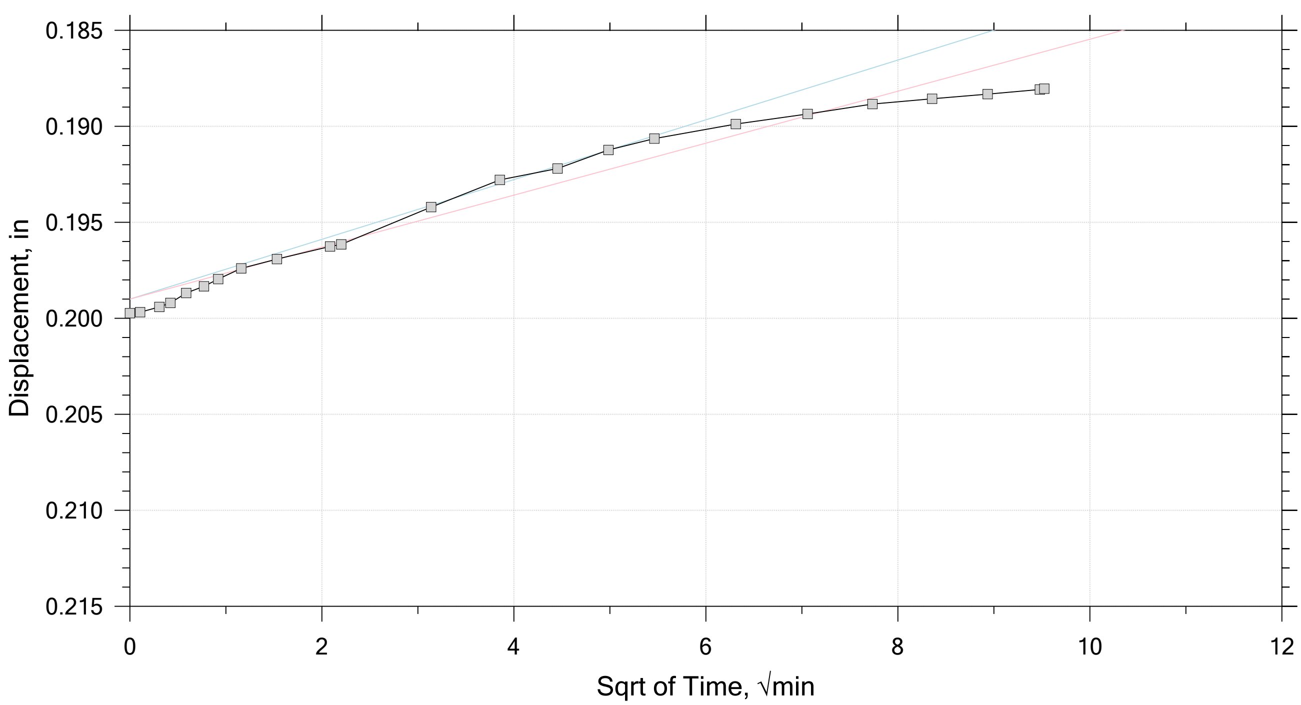
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 22 of 22

Constant Load Step

Stress: 450 psf





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
	Test Number: ICON304	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter, in: 2.50	Specific Gravity: 2.88 (Implied)	Liquid Limit: 39
Specimen Height, in: 1.00	Initial Void Ratio: 1.2	Plastic Limit: 21
Final Height, in: 0.81	Final Void Ratio: 0.784	Plasticity Index: 18

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	214	---	"ring"	317
Mass Container, gm	37.01	110.17	110.17	60.88
Mass Container + Wet Soil, gm	120.08	258.72	244.59	195.2
Mass Container + Dry Soil, gm	98.32	215.81	215.81	166.44
Mass Dry Soil, gm	61.31	105.64	105.64	105.56
Water Content, %	35.49	40.62	27.25	27.25
Void Ratio	---	1.20	0.78	---
Degree of Saturation, %	---	97.73	100.00	---
Dry Unit Weight, pcf	---	81.821	100.72	---

Preconsolidation Stress, psf	2400
Compression Ratio	0.23
Rebound Ratio	0.021
Compression Index	0.5052
Rebound Index	0.04613

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.

Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
Test Number: ICON304	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients

Step	Applied Stress psf	EOP Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv ft²/day	Mv ft²/lb	k cm/s
1	200.	0.006687	1.18	0.667	28.047	7.54e-02	3.34e+01	5.54e-08
2	300.	0.008836	1.18	0.882	46.085	4.55e-02	2.14e+01	2.15e-08
3	450.	0.01164	1.17	1.16	29.536	7.06e-02	1.87e+01	2.90e-08
4	675.	0.01545	1.16	1.54	17.507	1.18e-01	1.69e+01	4.40e-08
5	1.01e+03	0.02096	1.15	2.09	47.822	4.29e-02	1.63e+01	1.54e-08
6	1.52e+03	0.02914	1.13	2.91	80.123	2.53e-02	1.61e+01	8.97e-09
7	2.28e+03	0.04144	1.11	4.14	52.313	3.79e-02	1.62e+01	1.35e-08
8	3.42e+03	0.07006	1.04	6.99	46.404	4.09e-02	2.51e+01	2.26e-08
9	5.13e+03	0.1098	0.956	11.0	47.647	3.70e-02	2.32e+01	1.89e-08
10	7.69e+03	0.1472	0.874	14.7	43.054	3.76e-02	1.45e+01	1.20e-08
11	1.16e+04	0.1760	0.811	17.6	35.432	4.23e-02	7.45e+00	6.93e-09
12	5.12e+03	0.1712	0.821	17.1	1.509	9.64e-01	7.49e-01	1.59e-08
13	2.28e+03	0.1649	0.835	16.5	5.063	2.91e-01	2.19e+00	1.41e-08
14	1.01e+03	0.1565	0.854	15.6	24.983	6.01e-02	6.65e+00	8.80e-09
15	2.28e+03	0.1599	0.846	16.0	11.343	1.33e-01	2.65e+00	7.77e-09
16	5.13e+03	0.1668	0.831	16.6	5.341	2.79e-01	2.43e+00	1.49e-08
17	1.15e+04	0.1817	0.798	18.1	7.837	1.85e-01	2.32e+00	9.46e-09
18	1.73e+04	0.2034	0.751	20.3	21.254	6.54e-02	3.76e+00	5.41e-09
19	2.60e+04	0.2289	0.695	22.8	19.687	6.65e-02	2.94e+00	4.31e-09
20	5.13e+03	0.2146	0.726	21.4	2.036	6.34e-01	6.86e-01	9.58e-09
21	1.52e+03	0.2020	0.754	20.2	12.394	1.08e-01	3.47e+00	8.24e-09
22	450.	0.1880	0.784	18.8	51.940	2.66e-02	1.31e+01	7.64e-09

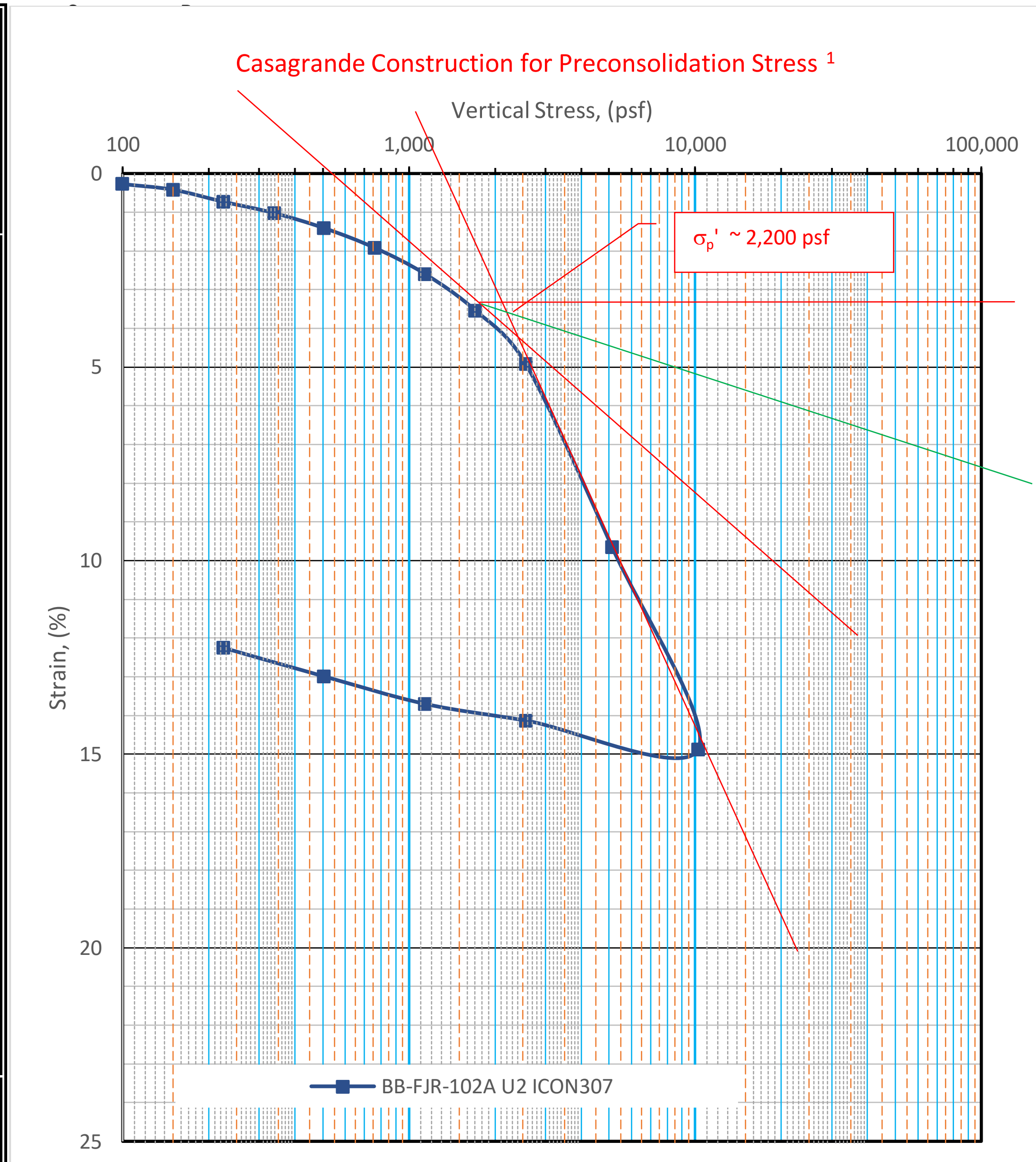
Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
Sample Number: U1	Test Date: 9/22/19	Depth: 31.38
Test Number: ICON304	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks:		
Displacement at End of Primary		

BB-FJR-102A 2U

Consolidation Test Data

Project Name:	Johnson Road Bridge
Project Number:	166-11
Project Location:	Falmouth, Maine
Client:	GZA Proj. No. 09.0026024.00
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	ICON 307			
Boring No.	BB-FJR-102A			
Sample No:	2U			
Boring Elevation (ft).	36.9			
Sample Depth (ft):	40 - 42			
Test Specimen Depth (Ft):	41.2			
Test Specimen Elevation:	-4.3			
Water Content (%):	31.19			
Dry Unit Weight (pcf):	89.72			
Wet Unit Weight (pcf):	117.70			
Saturation Before (%):	97.5			
Saturation After (%):	100			
Void Ratio Before:	0.97			
Void Ratio After:	0.73			
Overburden Pressure (psf):				
Max Previous stress (psf):	2,200			
Max Prev. stress (Work) (psf):	2,300			
OCR:				
Compression Index (C_{CE}):	0.16			
Recompression Index (C_{RE}):	0.02			
Liquid Limit:	31			
Plastic Limit:	19			
Plasticity Index:	12			
Liquidity Index:	1.0			
Specific Gravity (implied)	2.84			
Lab Vane Su at __ ft. (psf)				
Tested By:	sjr			
Date Tested:	9/25/2019			
Checked By:	sjr			

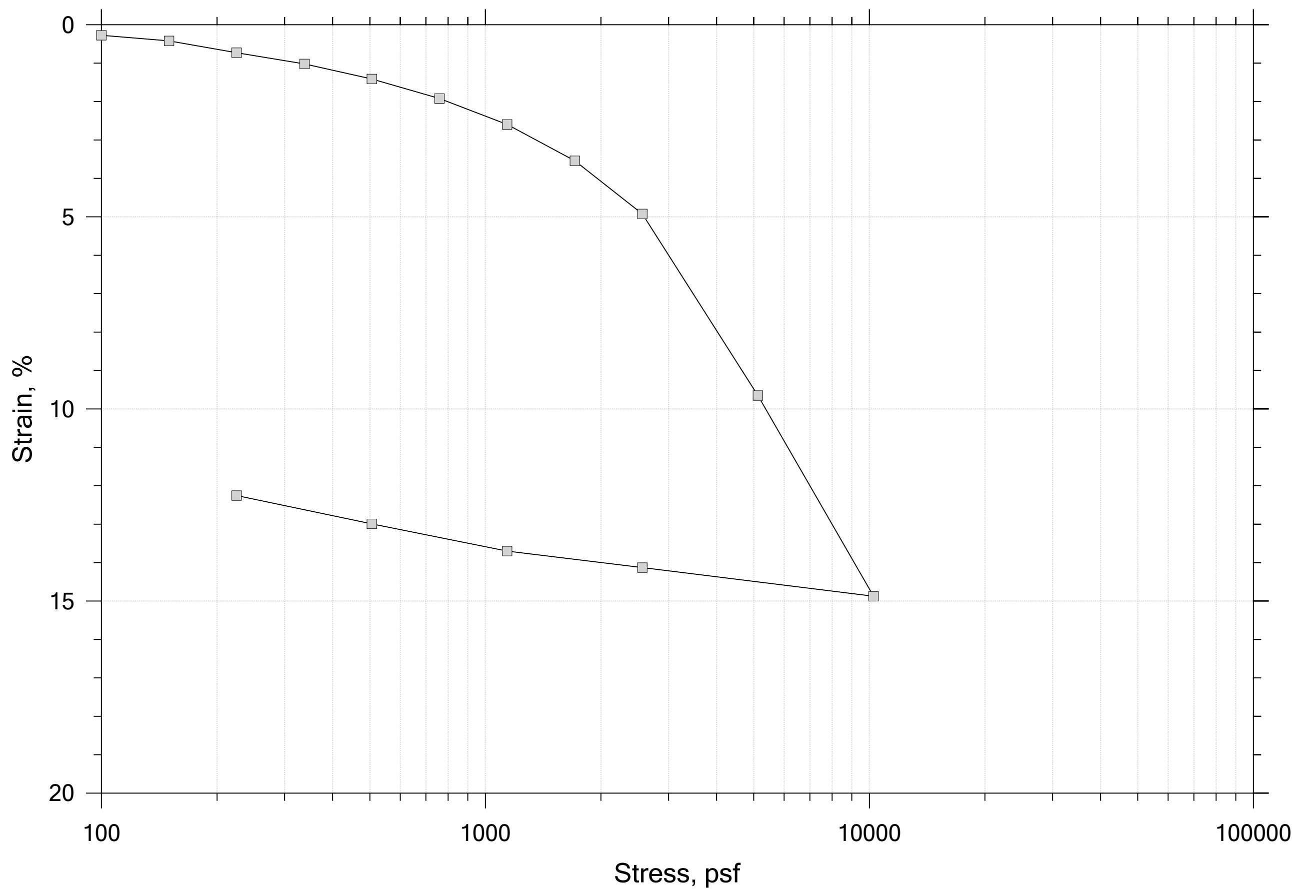


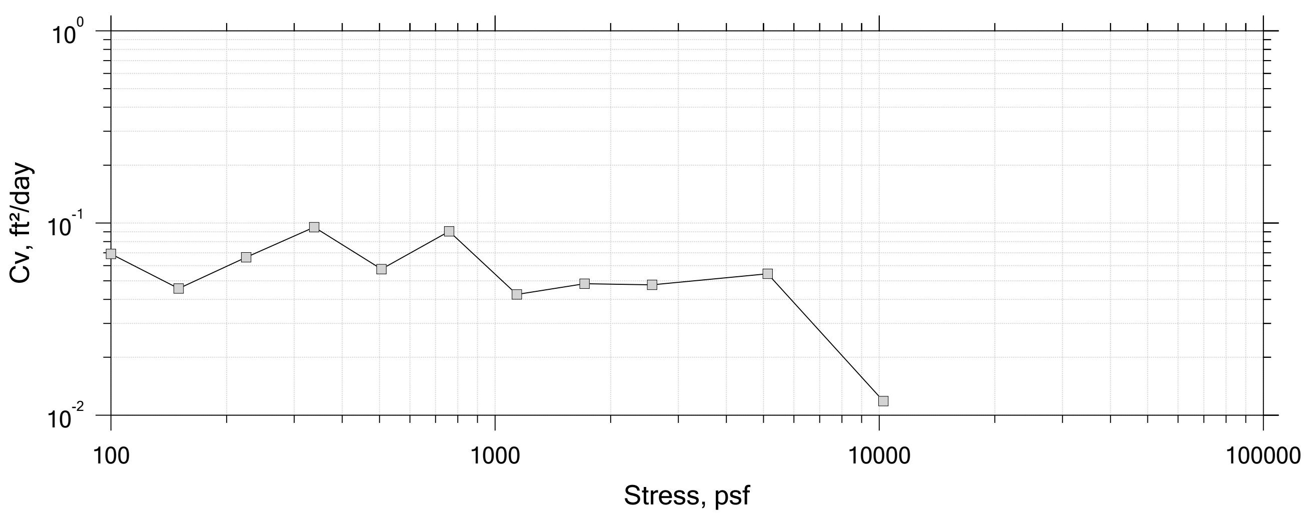
Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convienience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.

Soil Metrics LLC Geotechnical Engineering

One-Dimensional Consolidation by ASTM D2435 - Method B

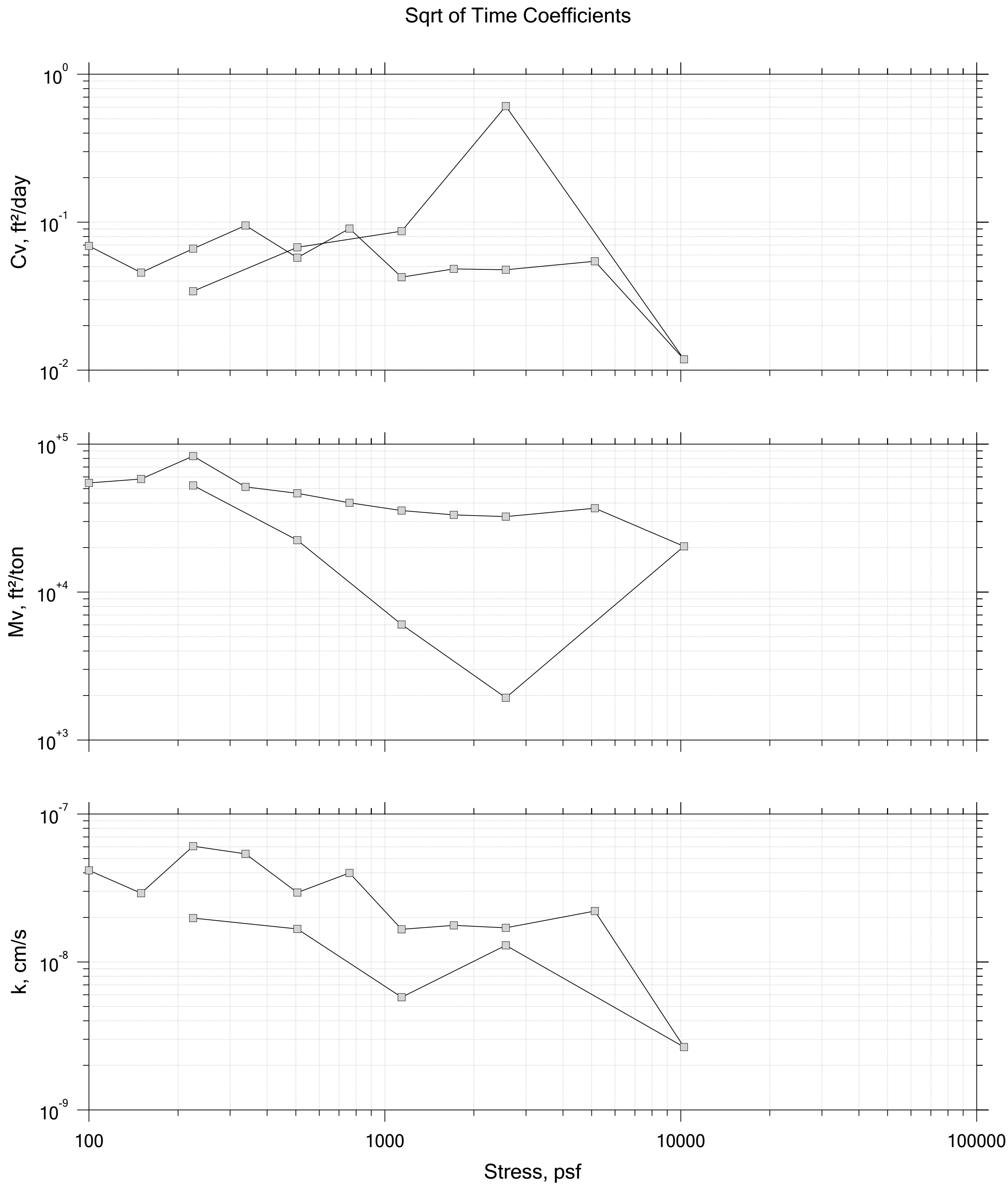
Summary Report





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		
	Displacement at End of Primary		

One-Dimensional Consolidation by ASTM D2435 - Method B



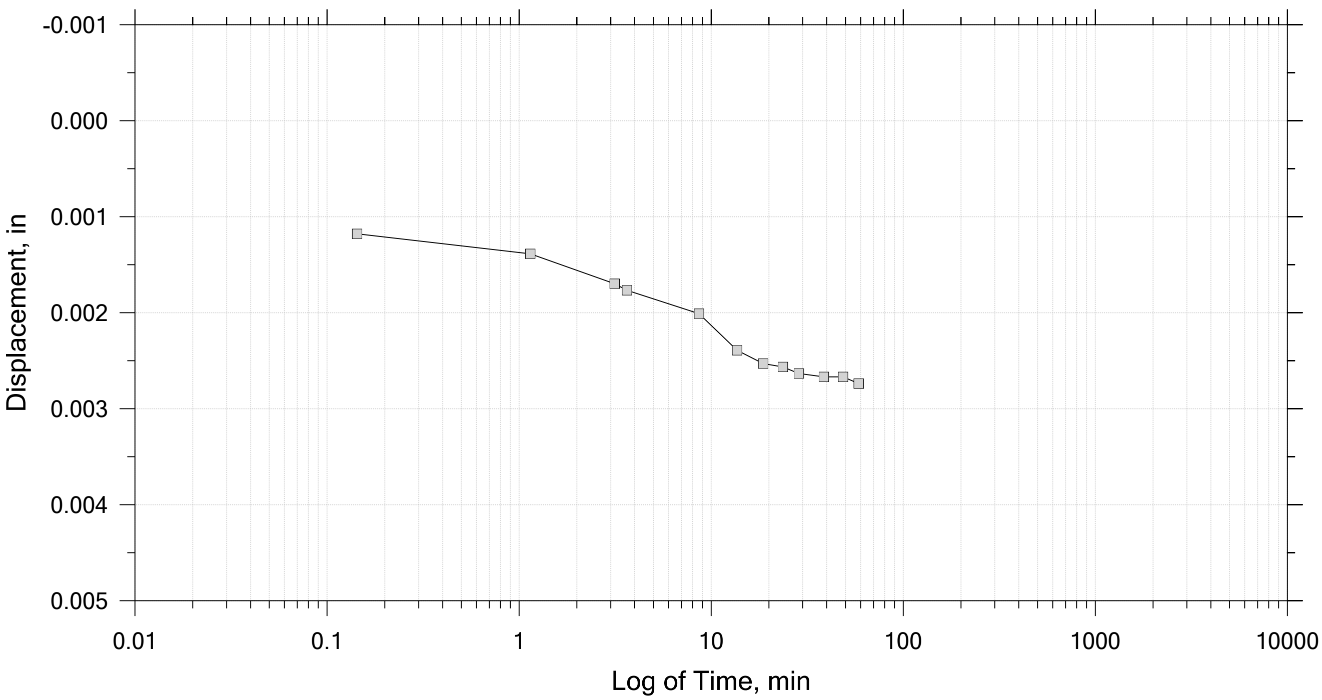
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

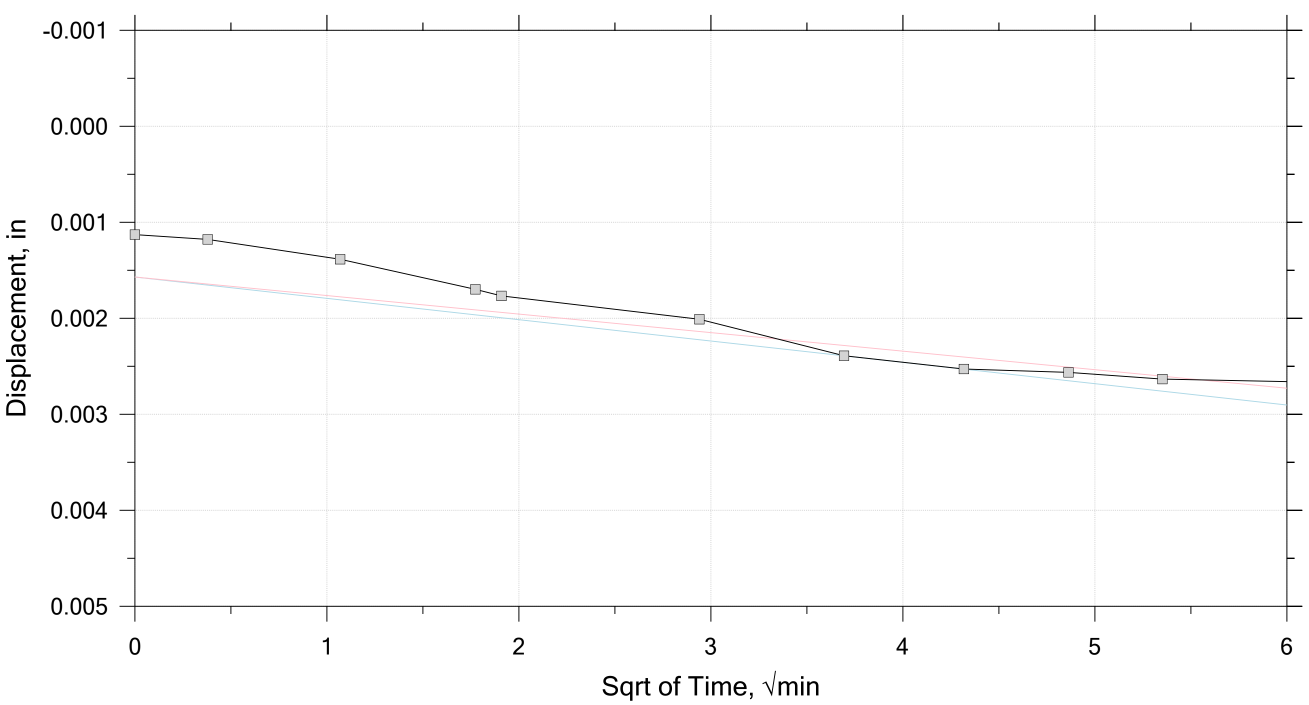
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 15

Constant Load Step

Stress: 100 psf





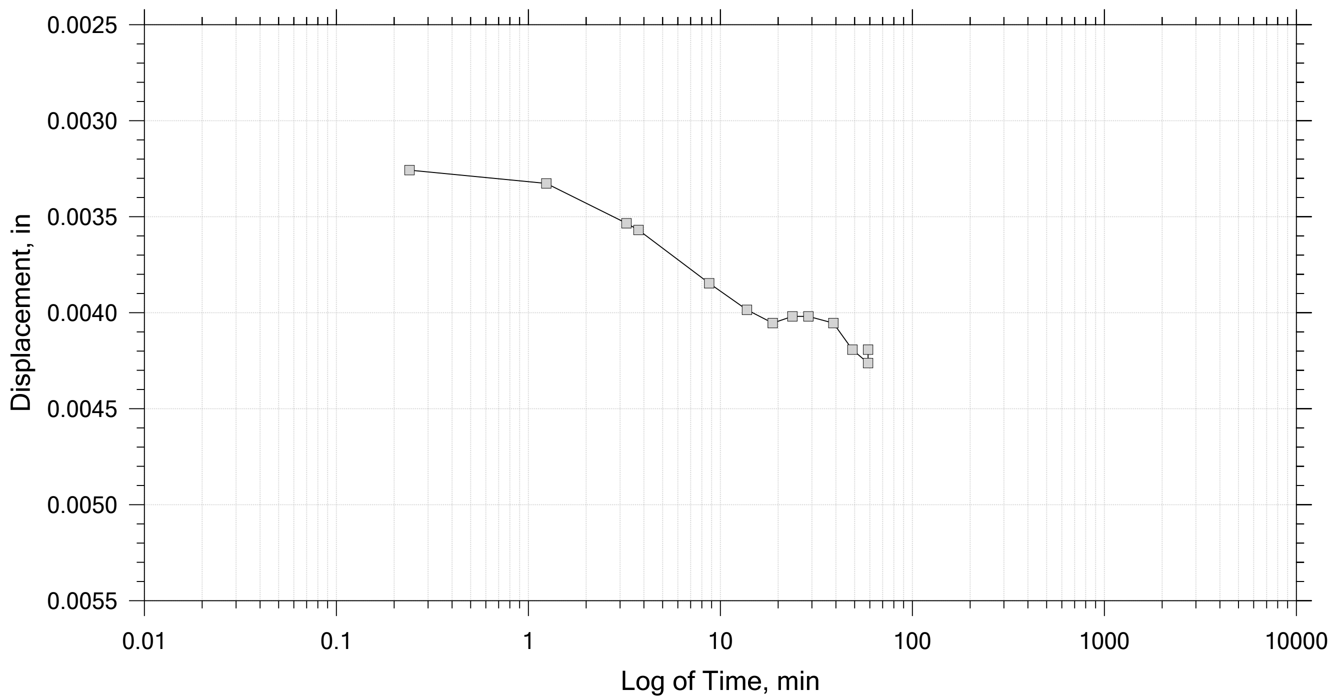
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

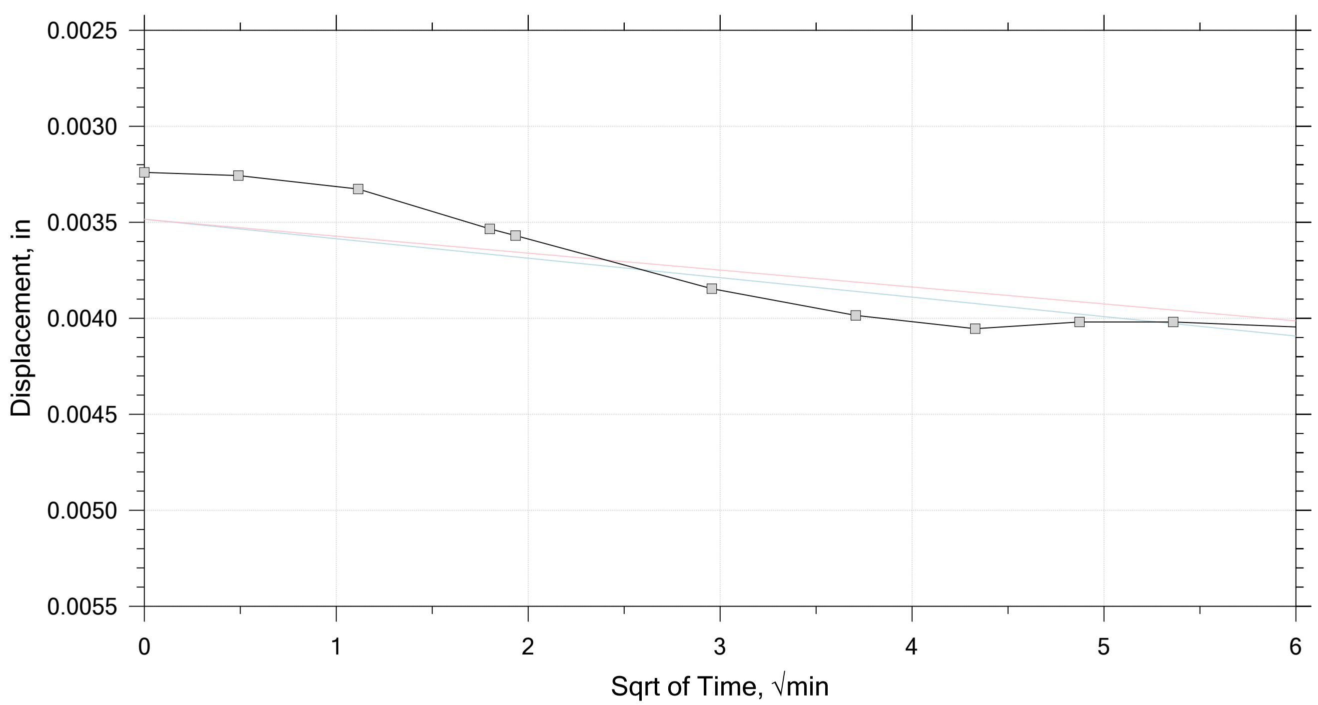
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 15

Constant Load Step

Stress: 150 psf





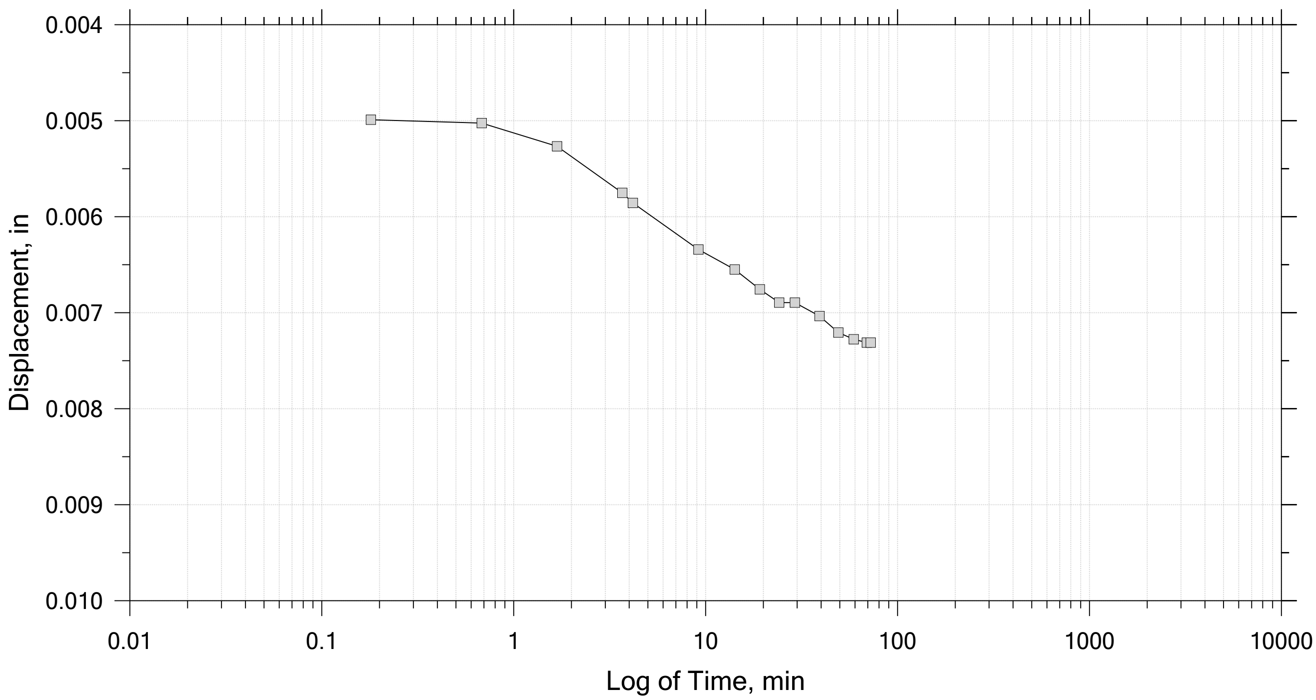
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

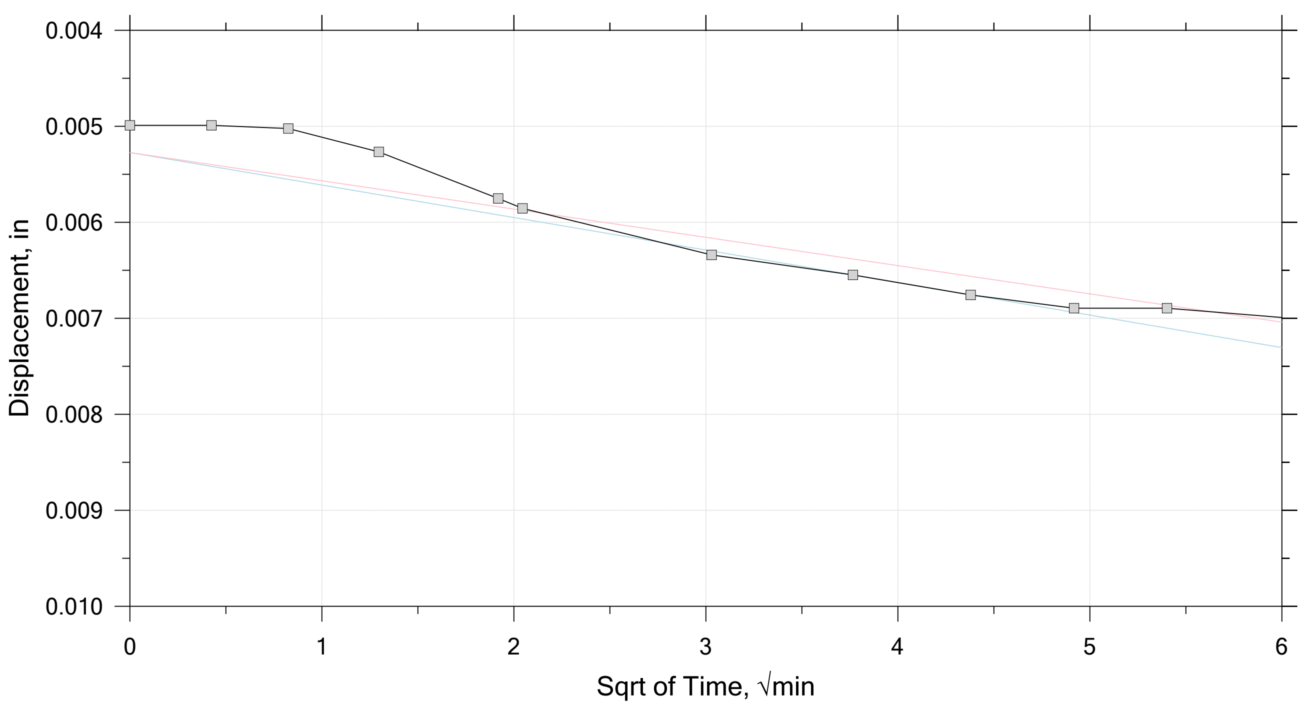
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 15

Constant Load Step

Stress: 225 psf





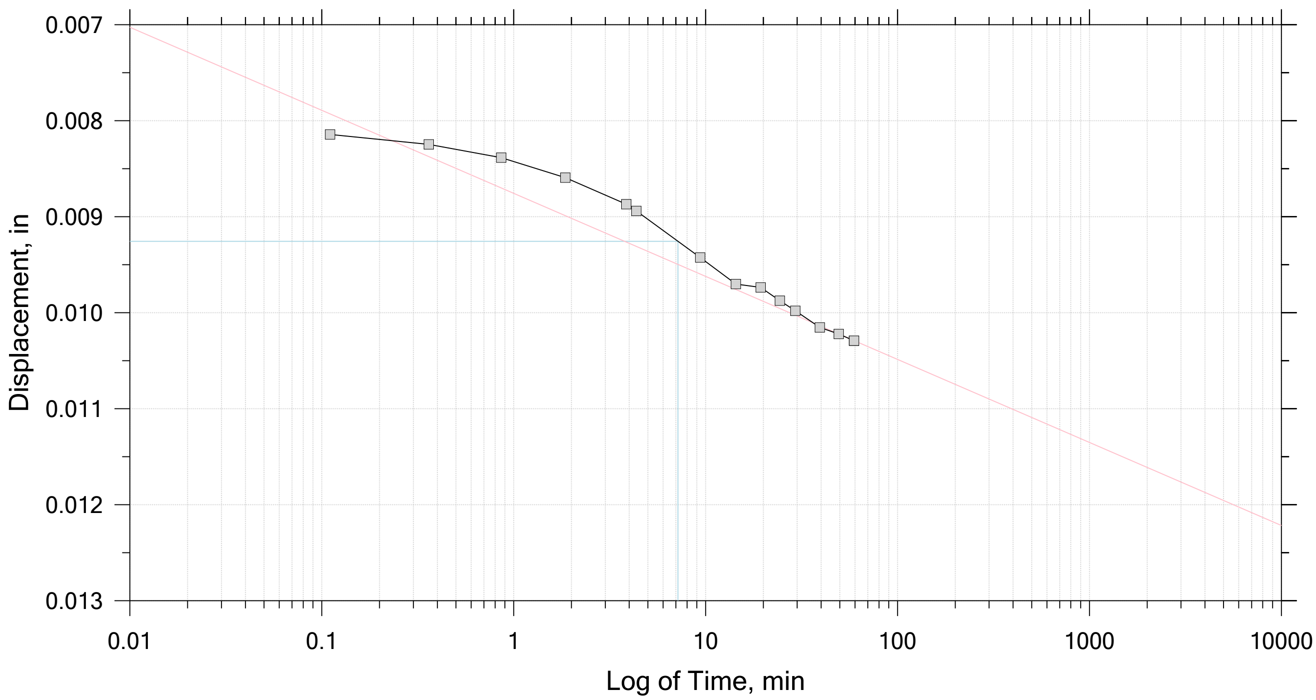
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

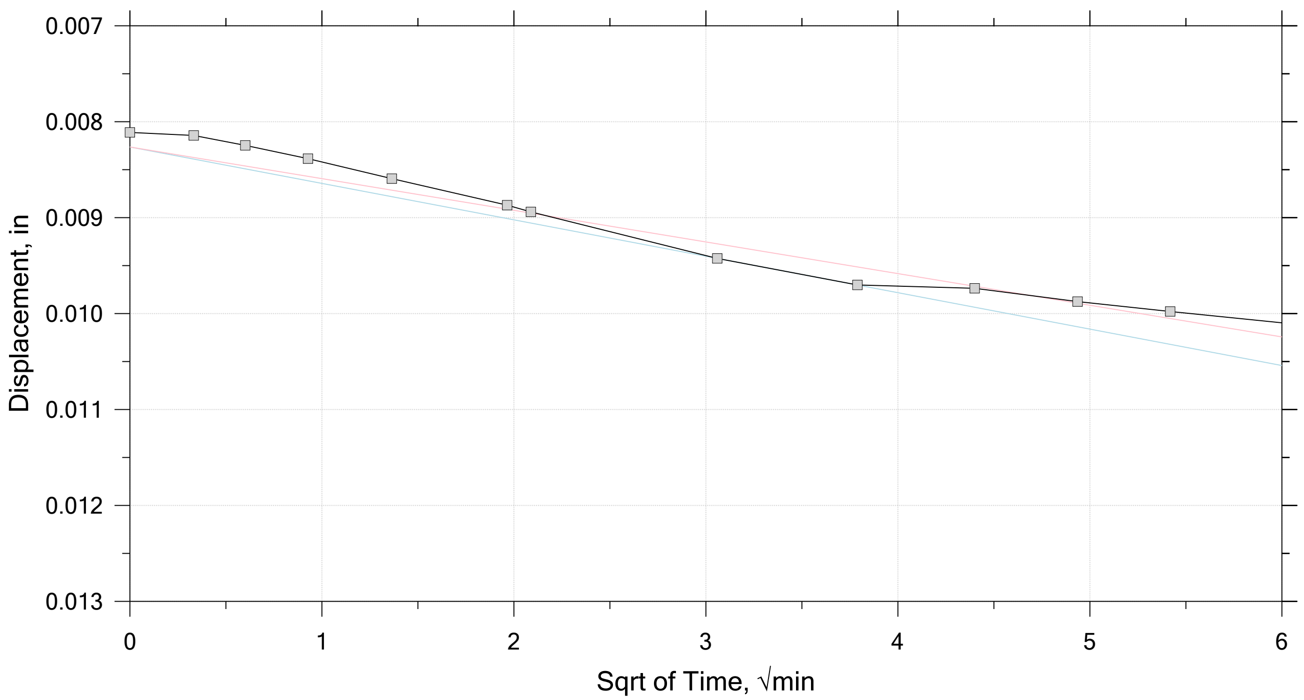
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 15

Constant Load Step

Stress: 338 psf

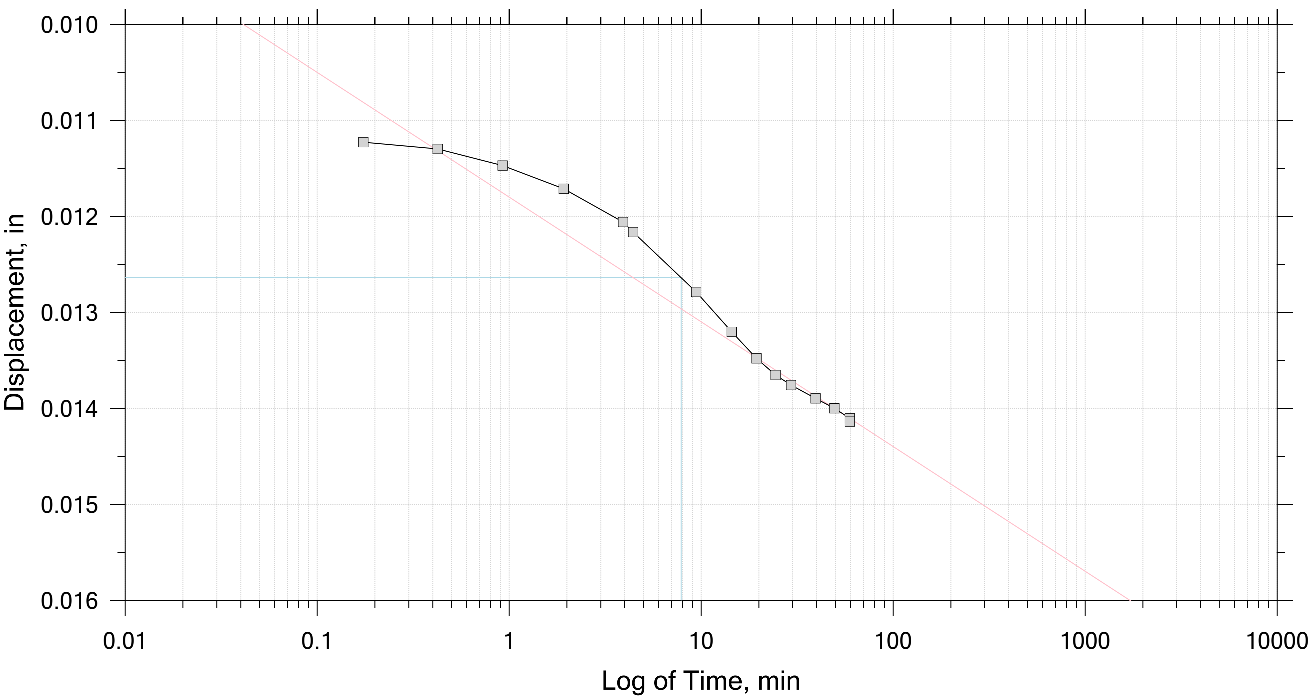


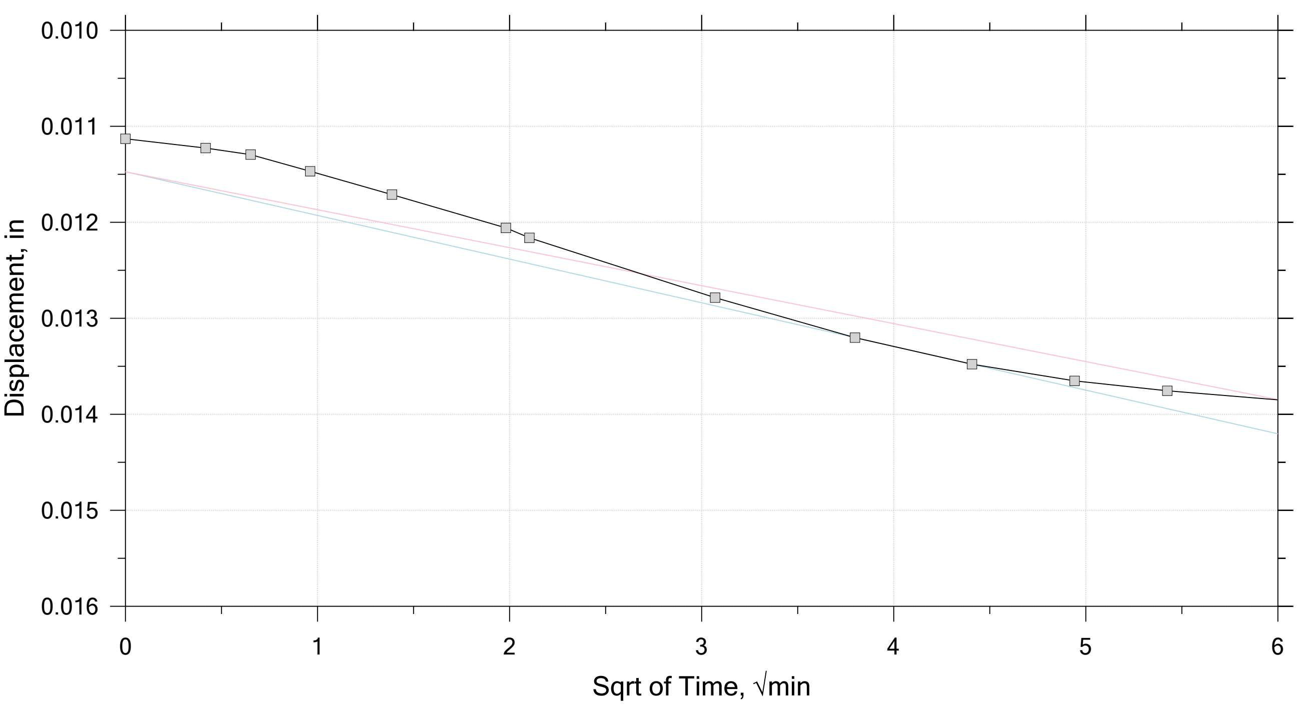


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 15
Constant Load Step
Stress: 506 psf





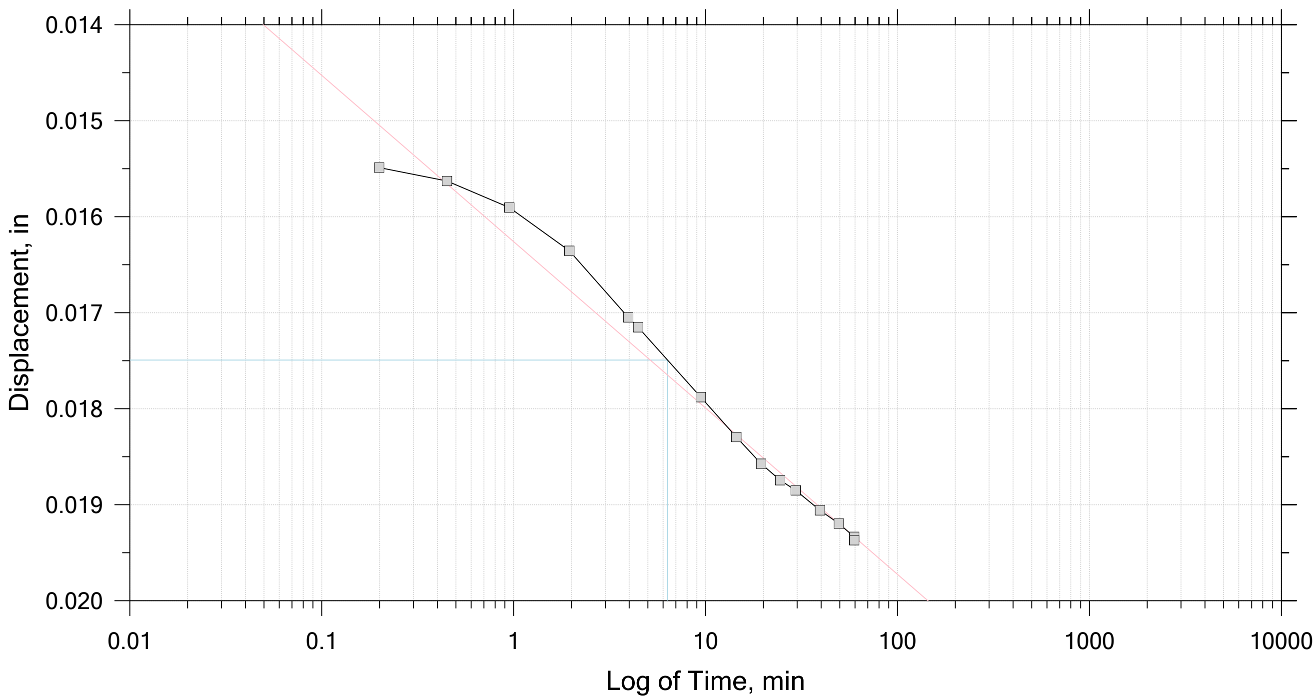
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

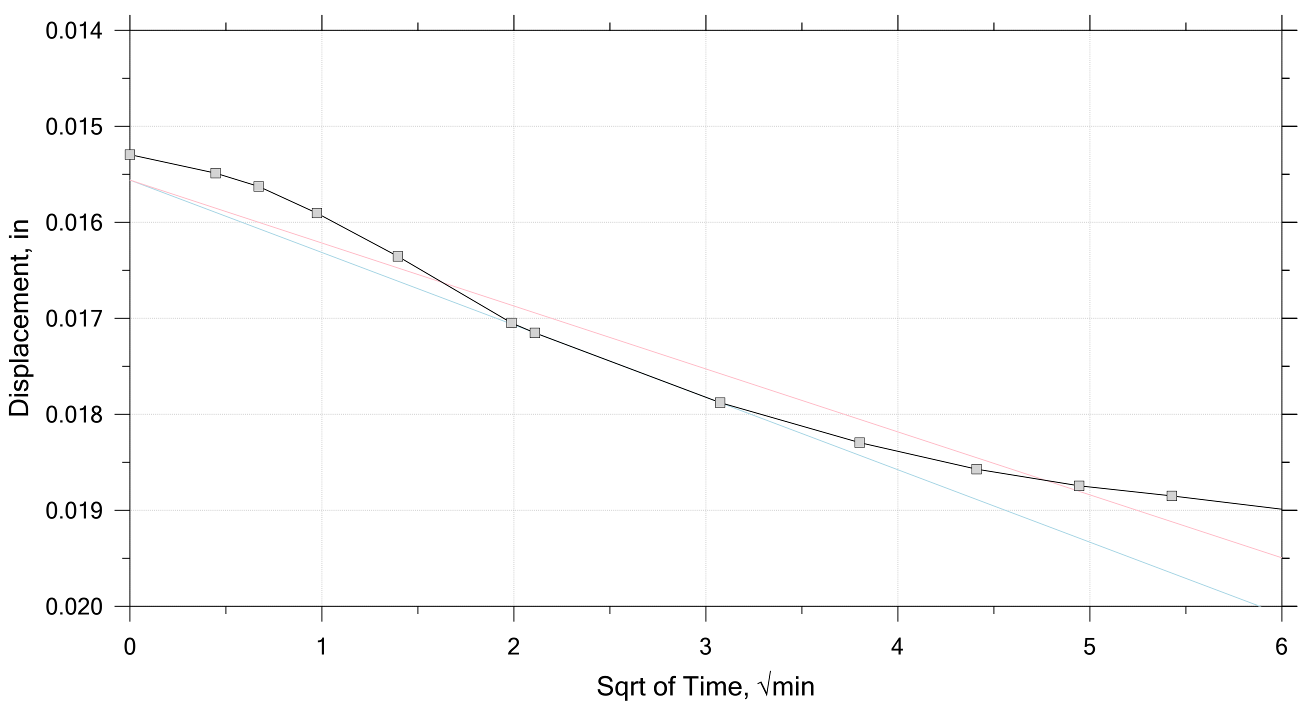
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 15

Constant Load Step

Stress: 759 psf





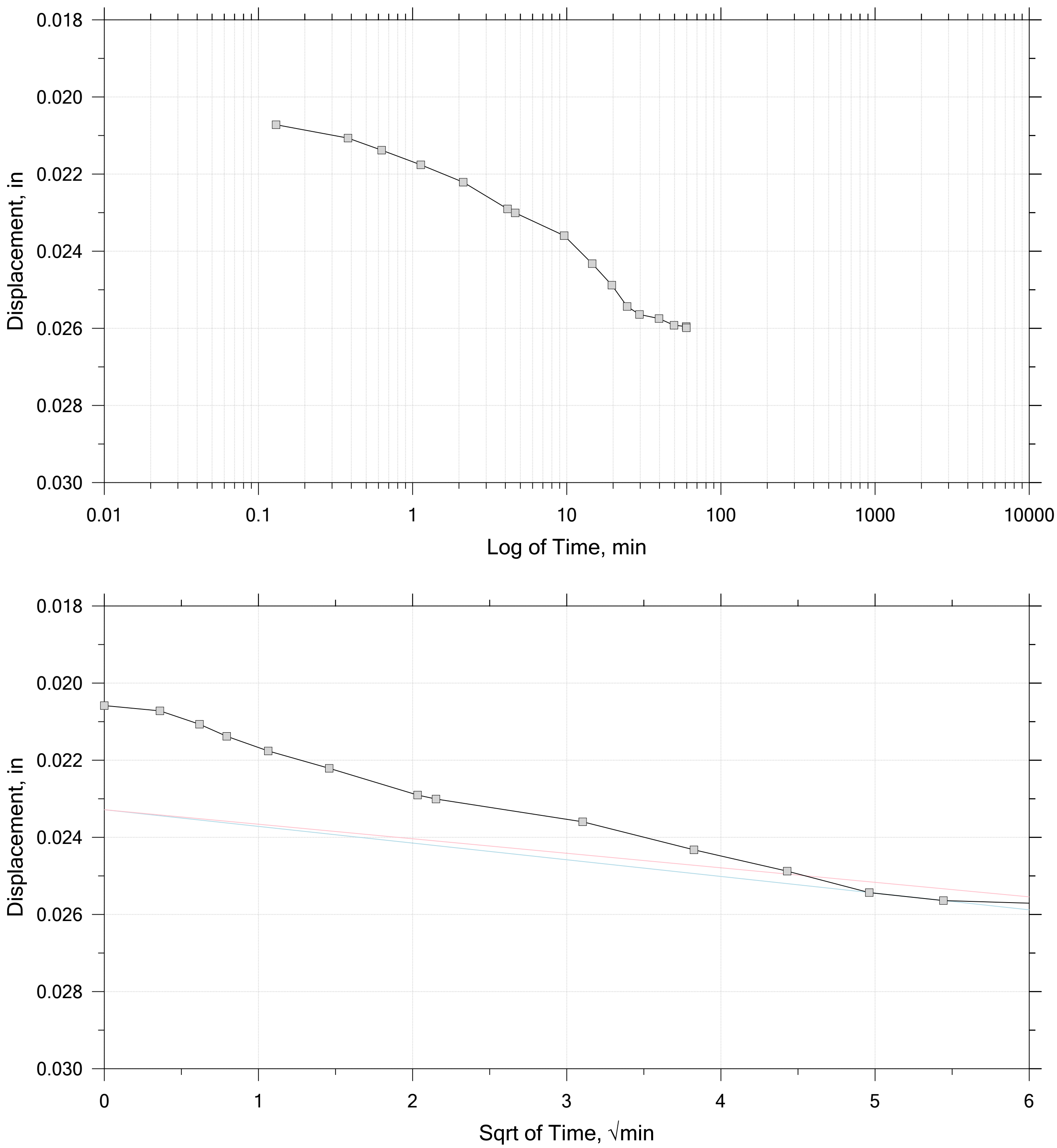
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 15

Constant Load Step

Stress: 1.14e+03 psf



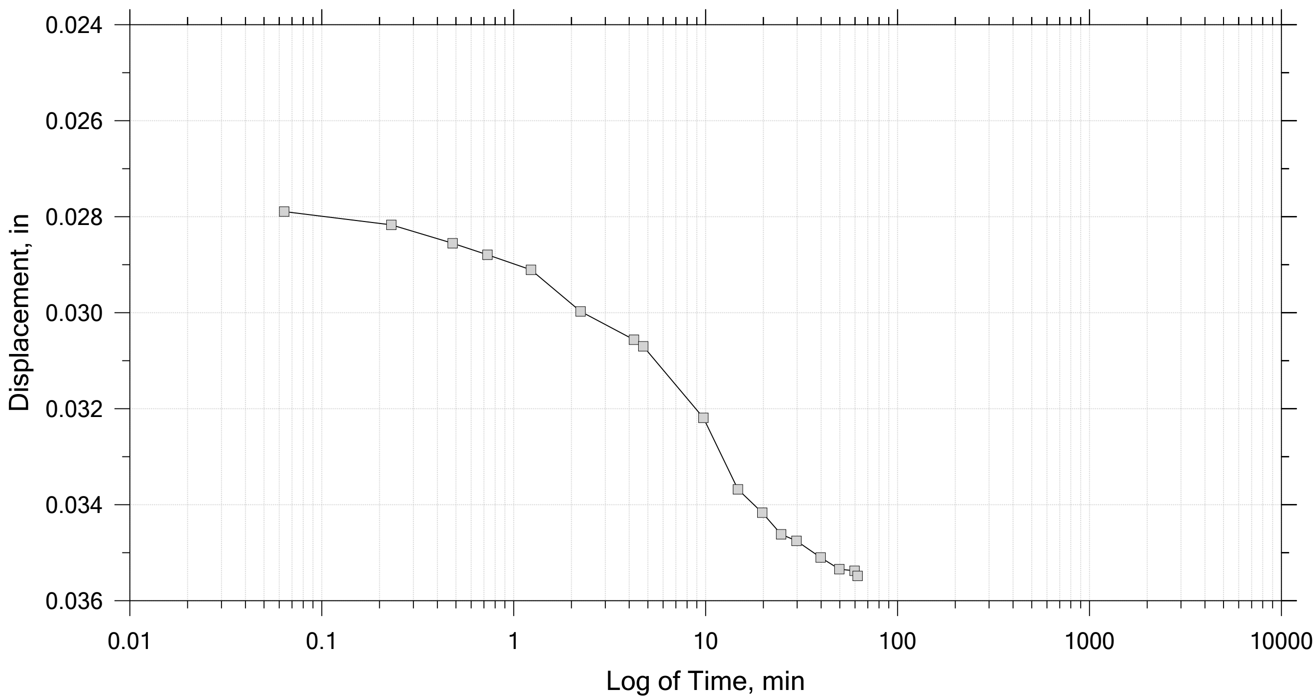
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

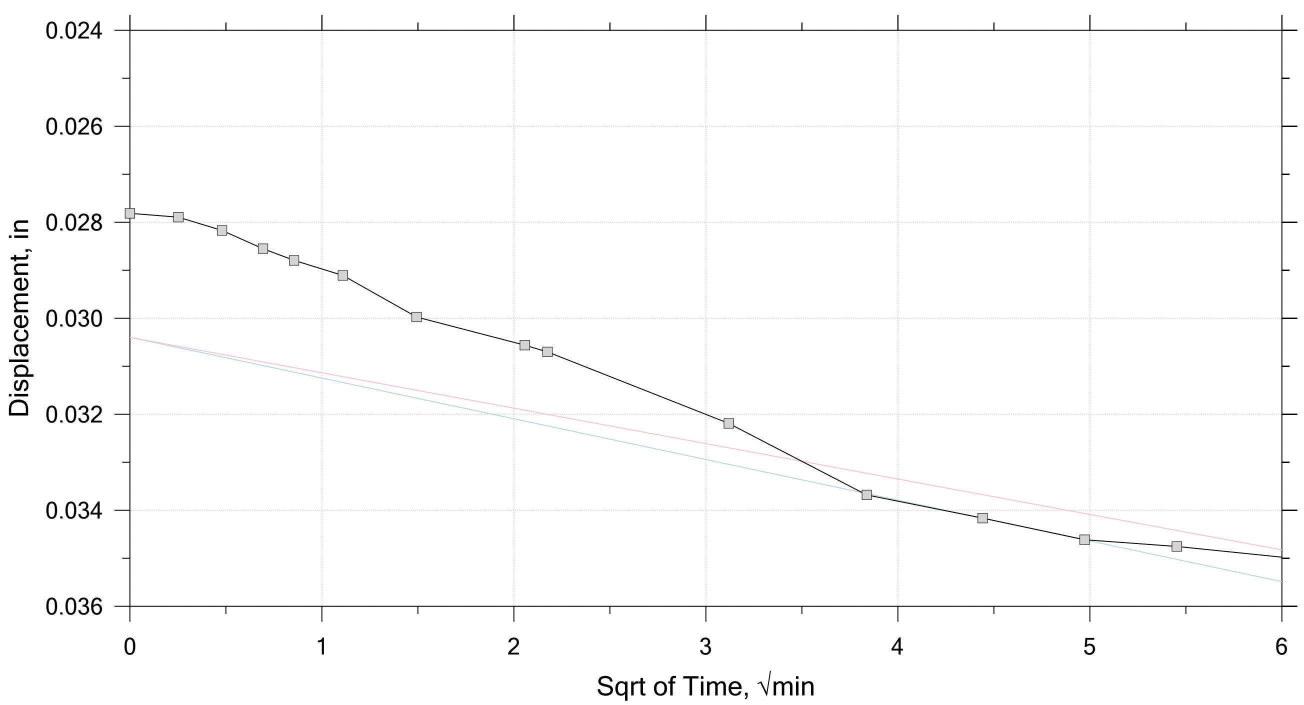
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 15

Constant Load Step

Stress: 1.71e+03 psf

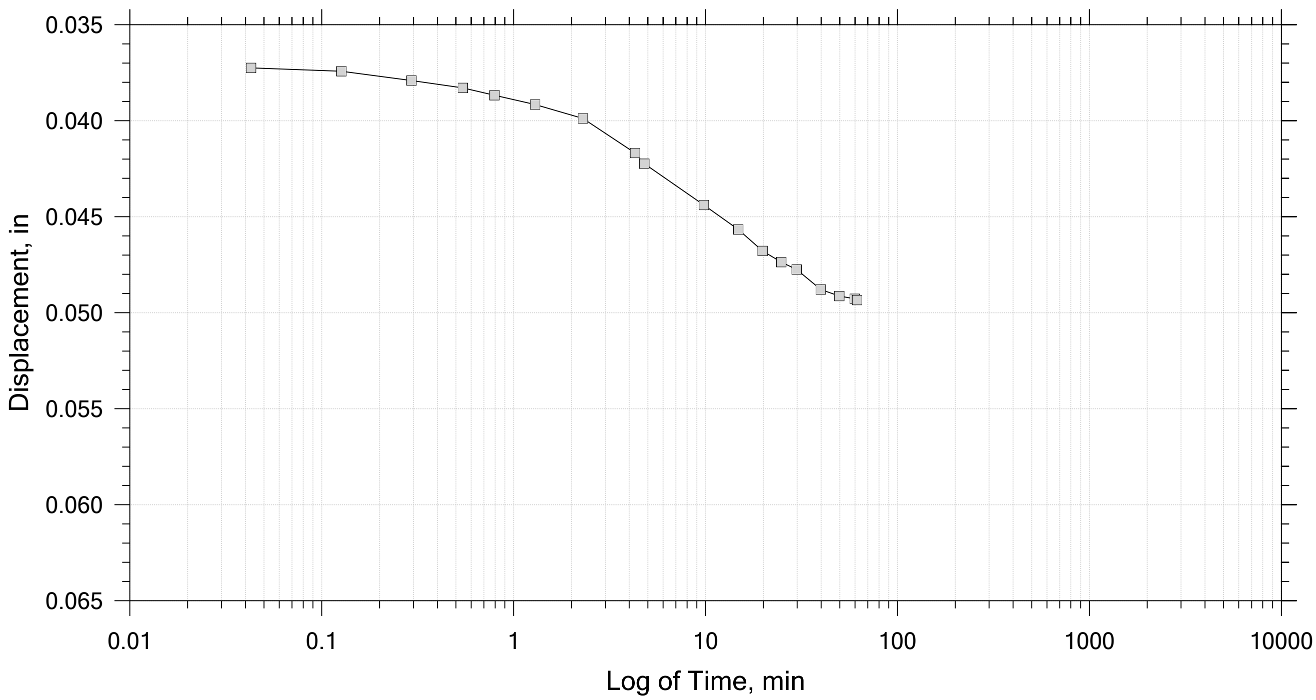


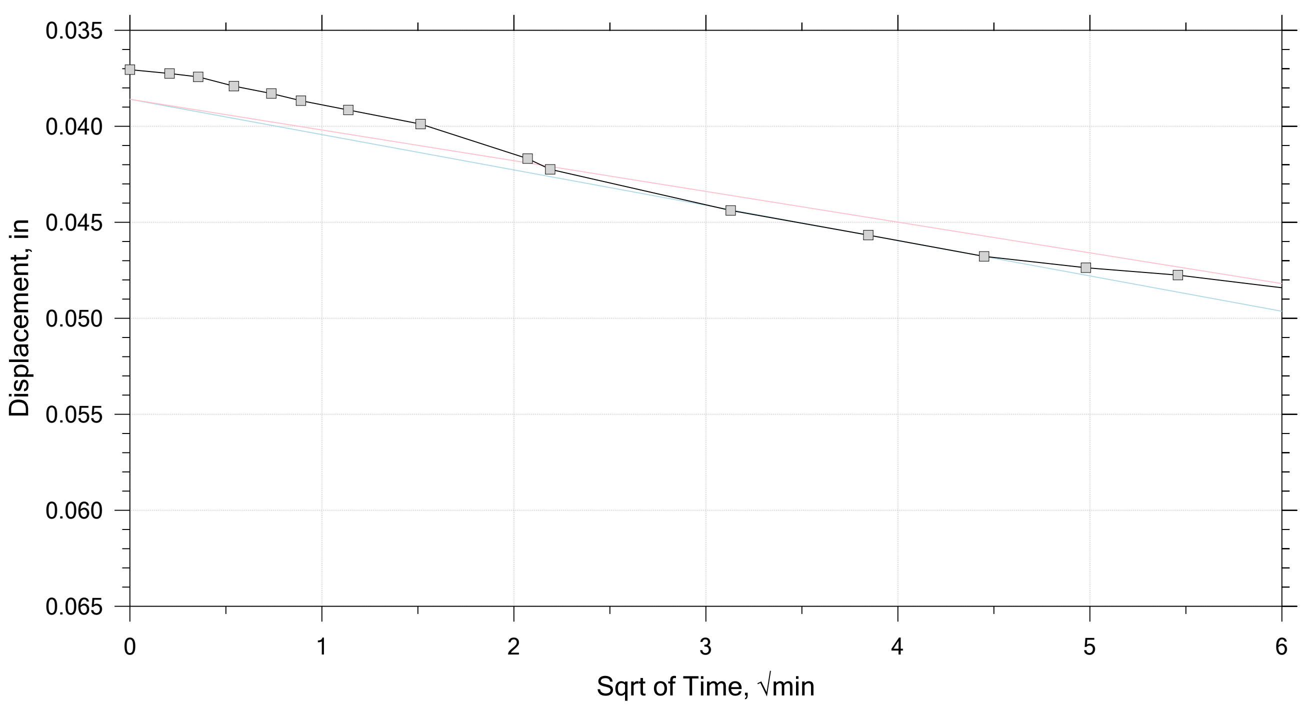


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 15
Constant Load Step
Stress: 2.56e+03 psf

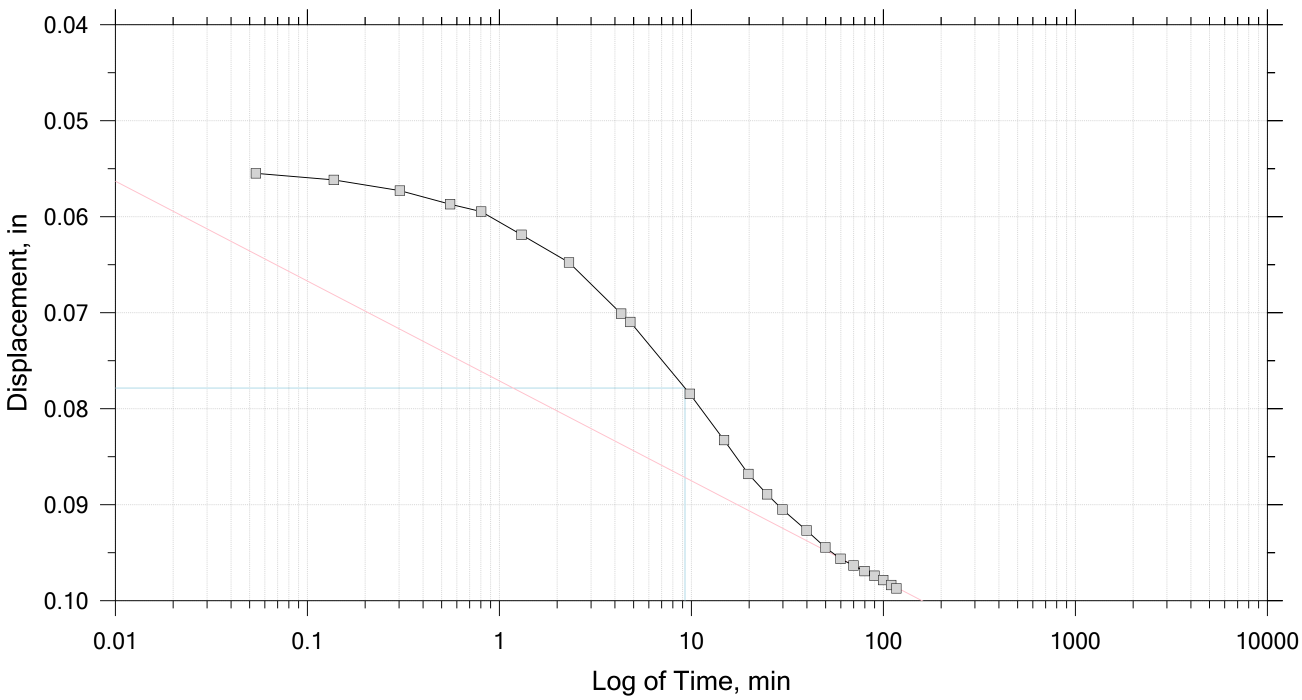


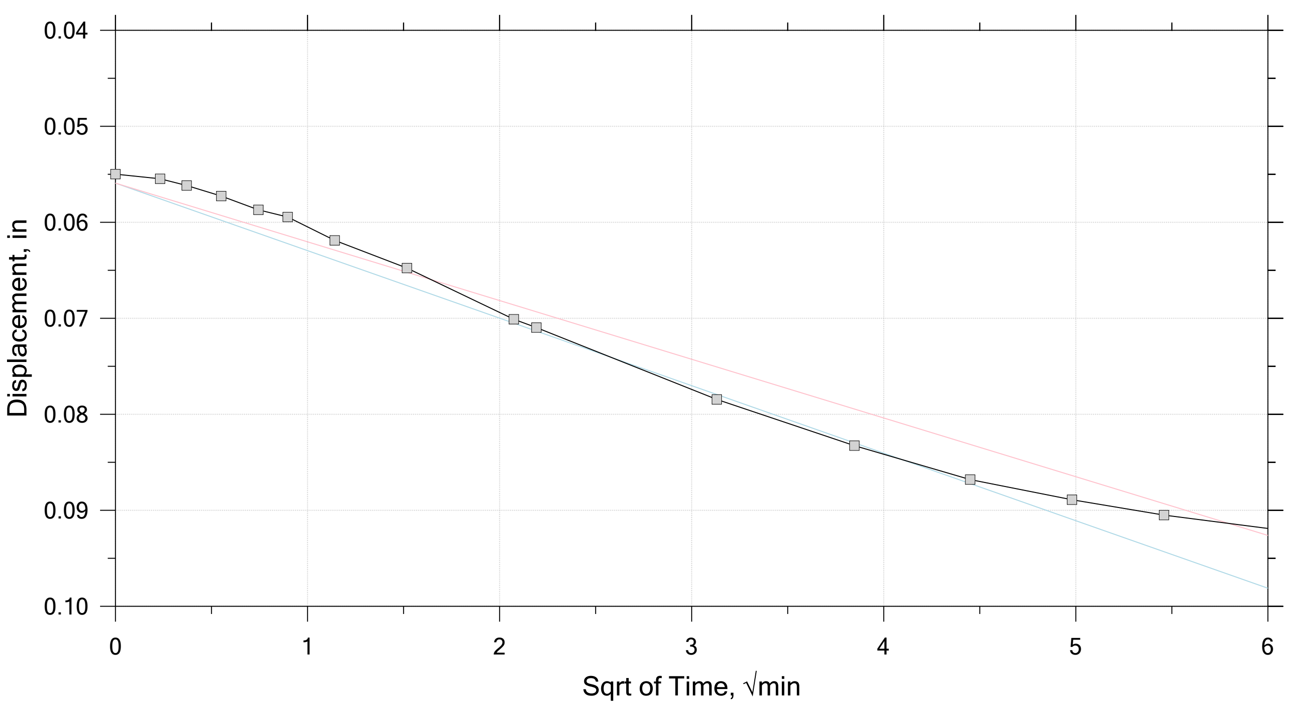


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 15
Constant Load Step
Stress: 5.13e+03 psf





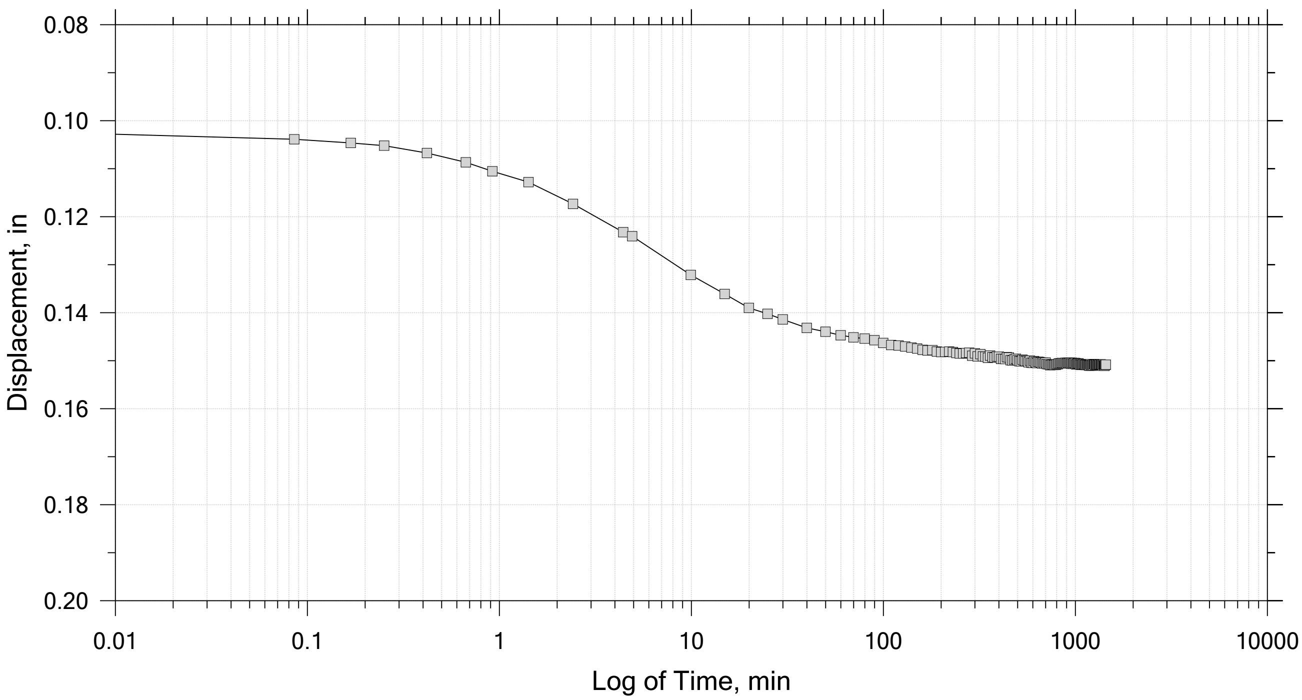
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

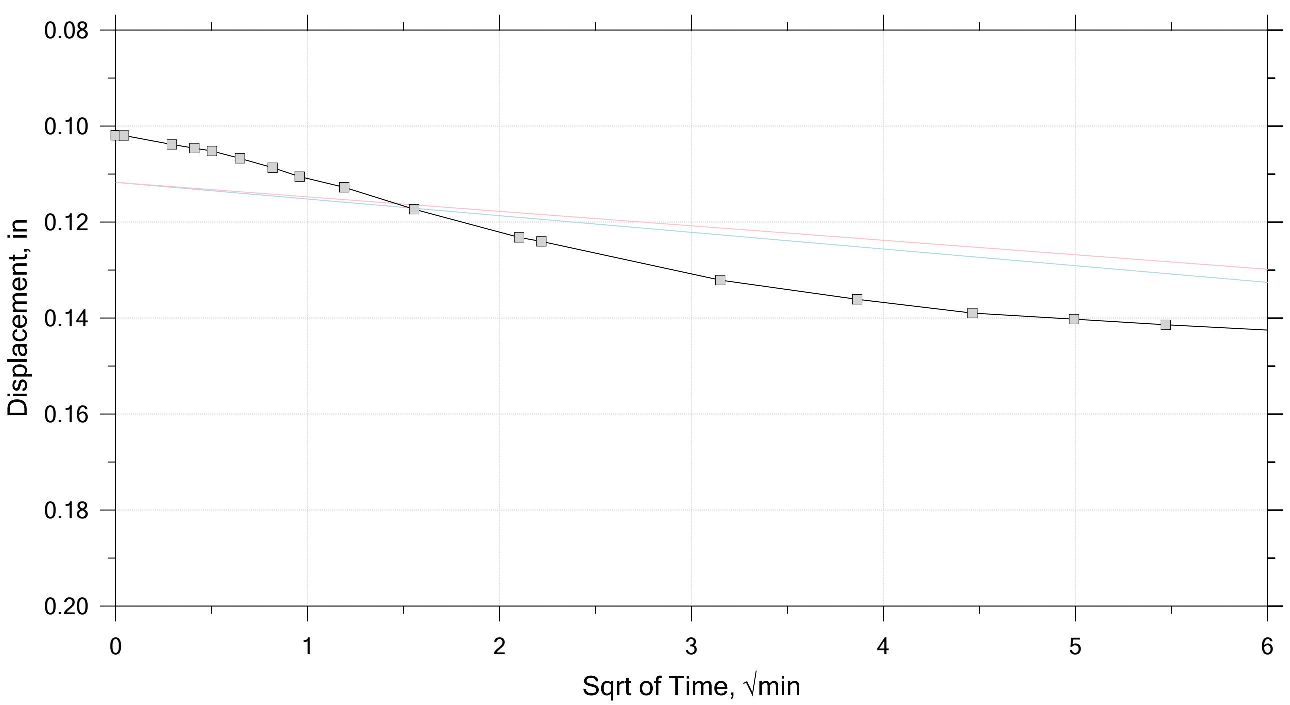
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 15

Constant Load Step

Stress: 1.03e+04 psf





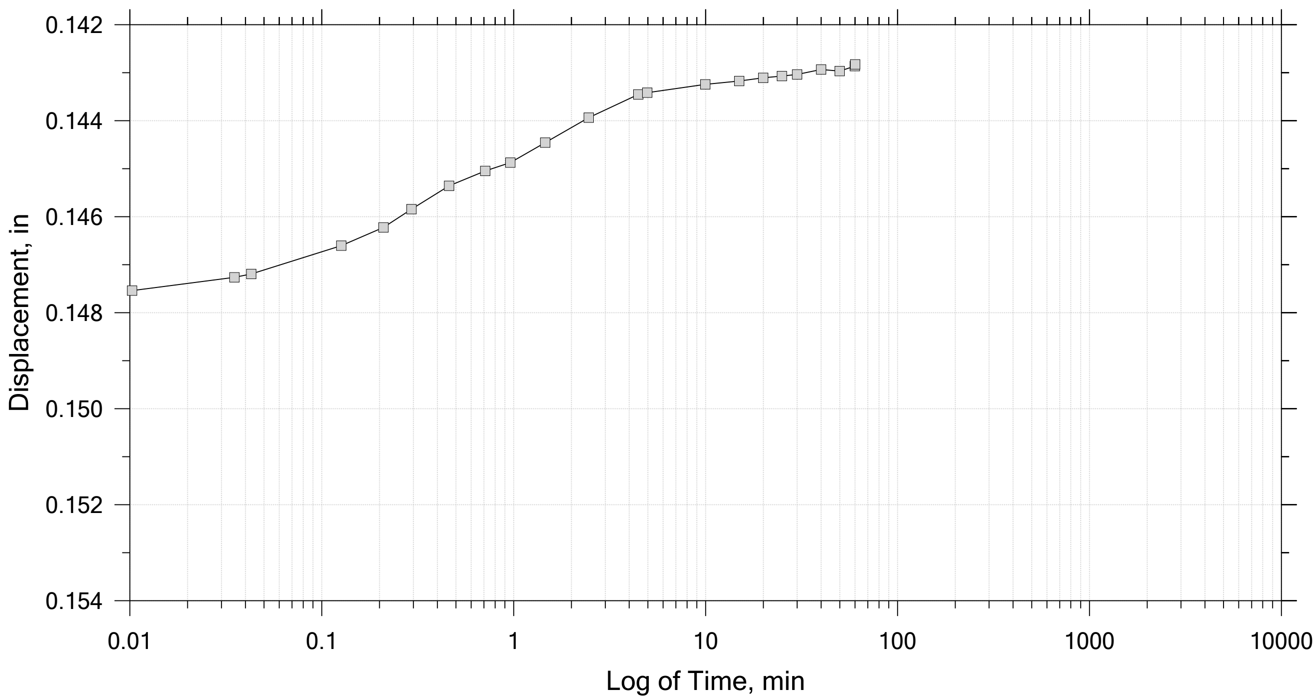
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

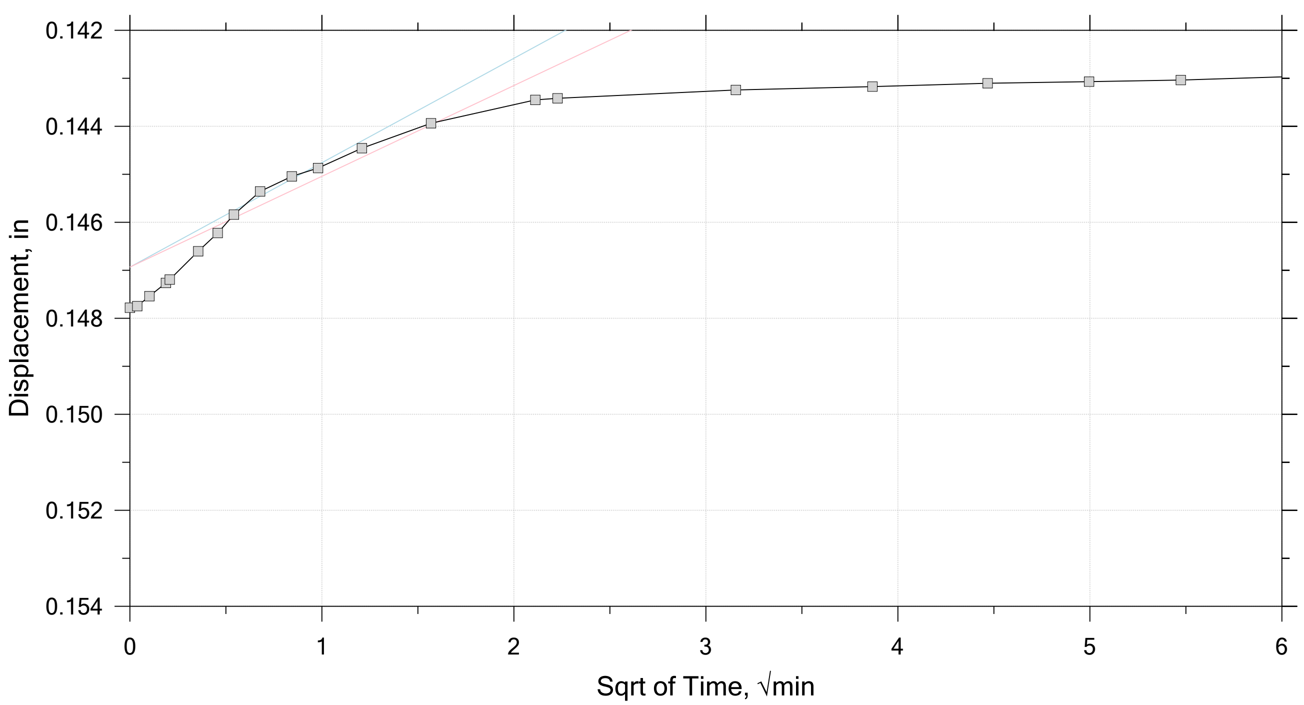
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 15

Constant Load Step

Stress: 2.56e+03 psf





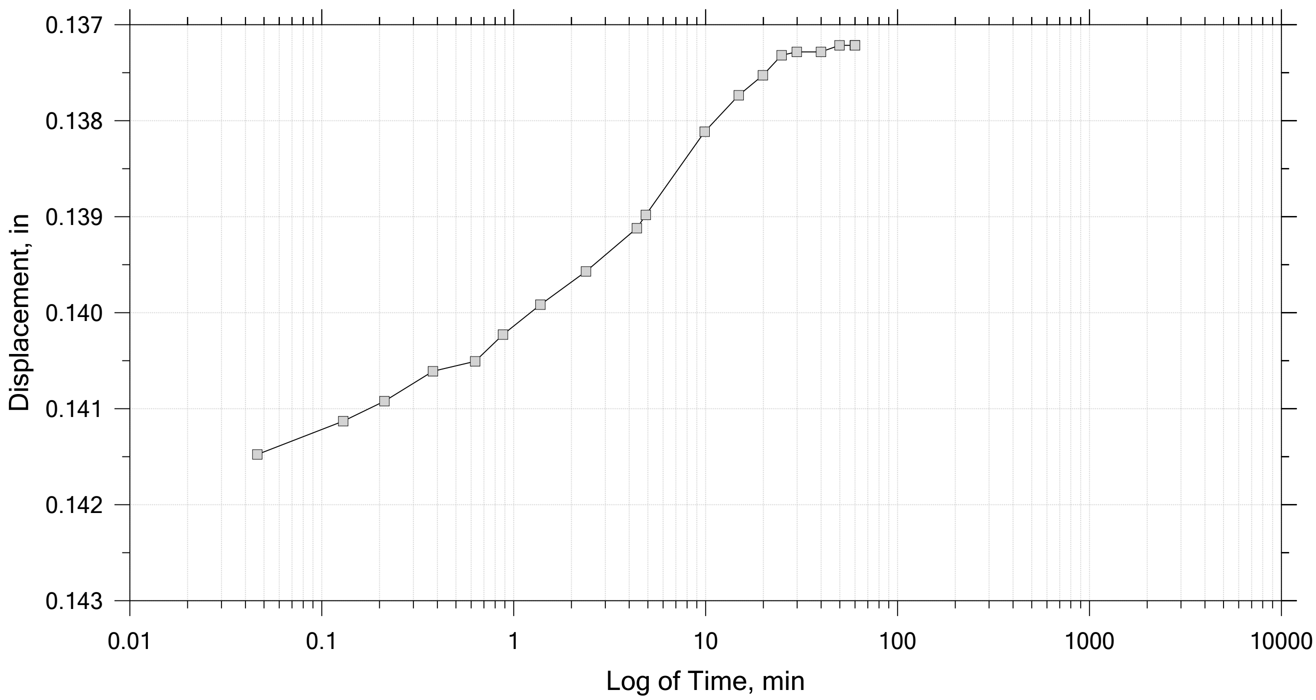
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

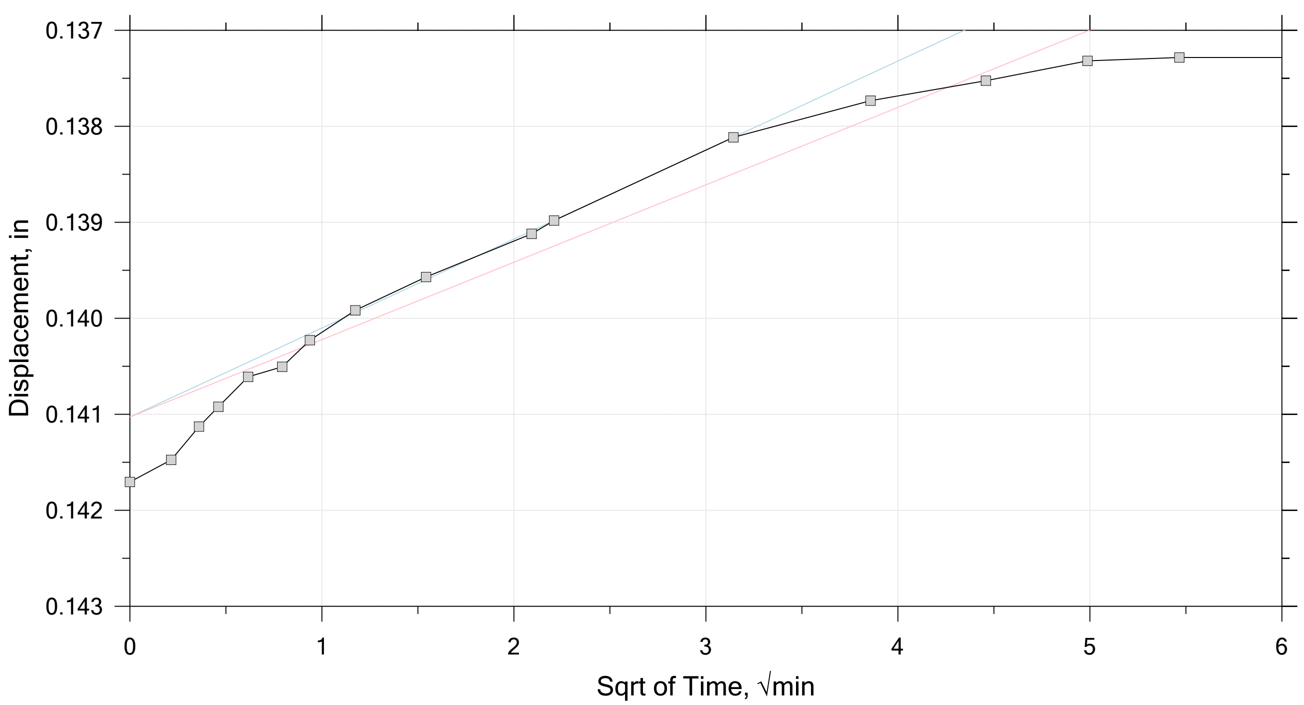
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 15

Constant Load Step

Stress: 1.14e+03 psf





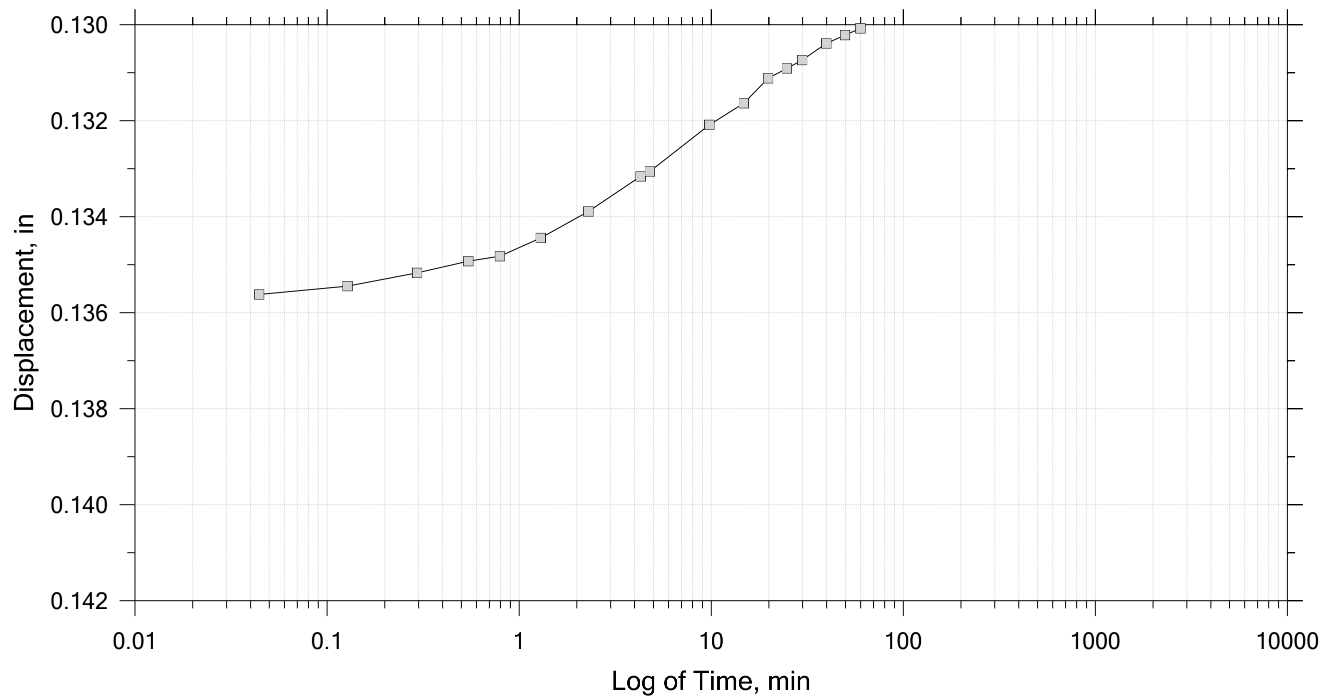
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

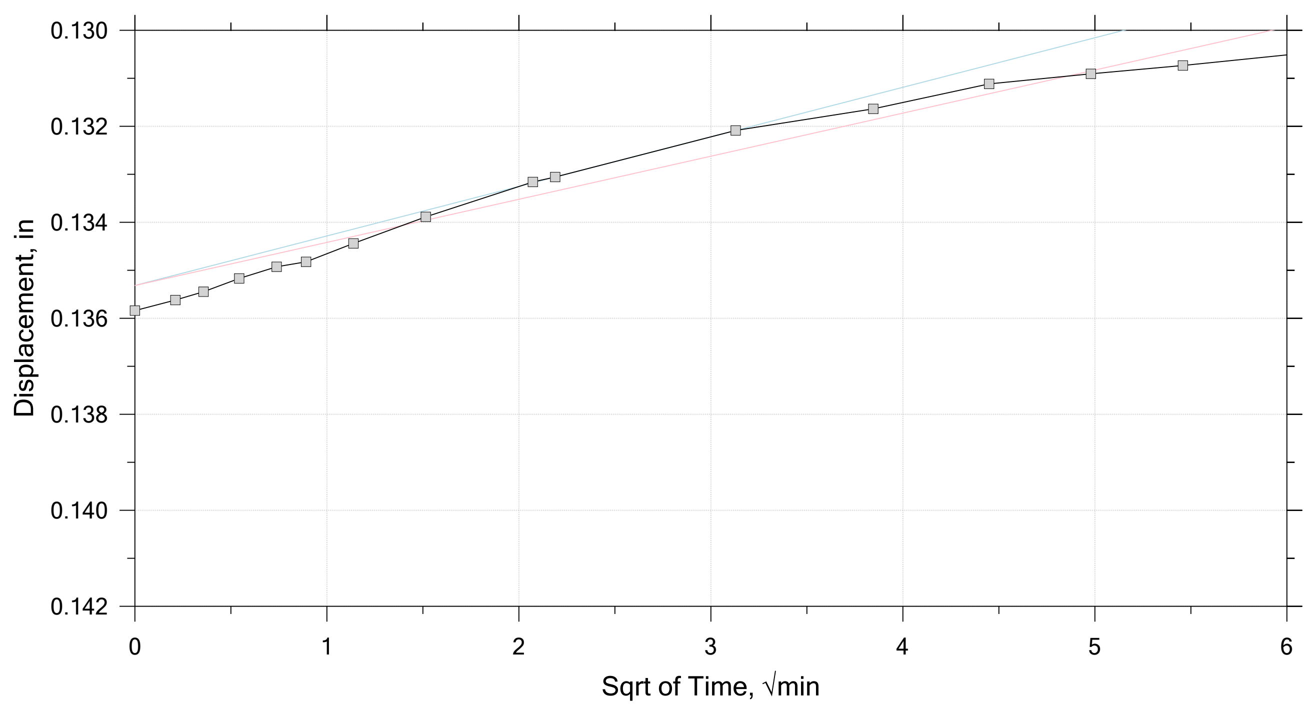
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 15

Constant Load Step

Stress: 506 psf





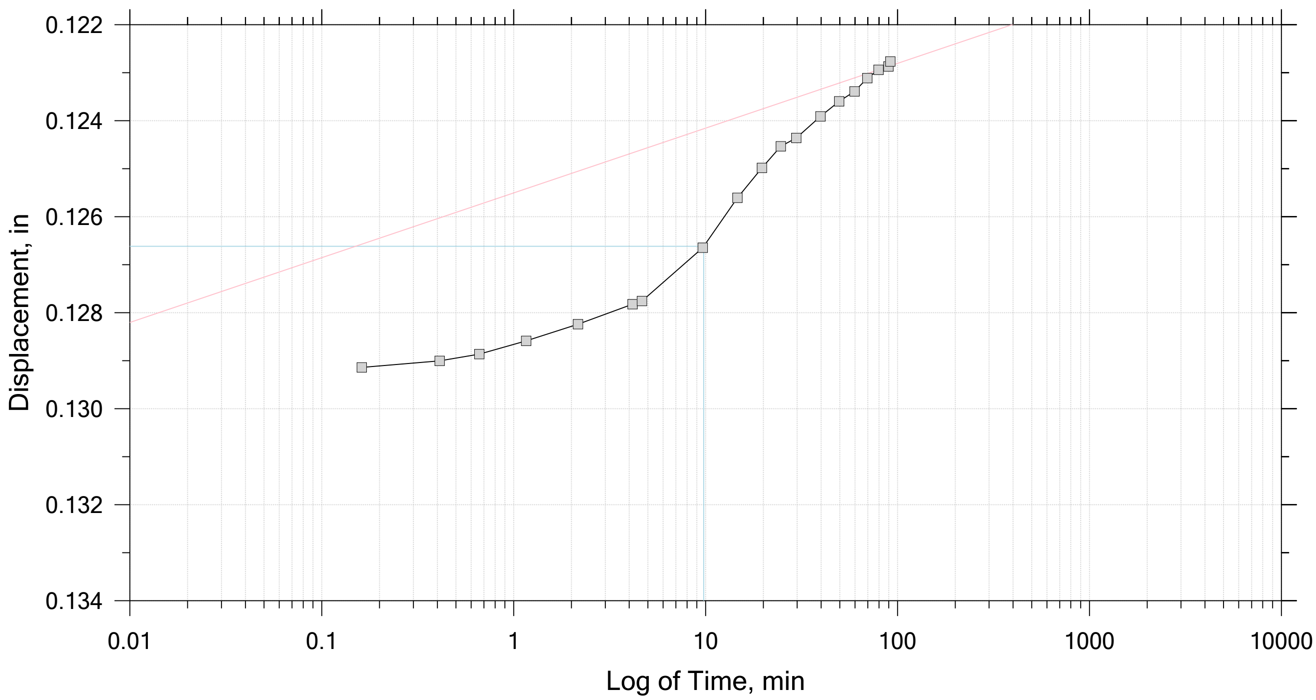
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

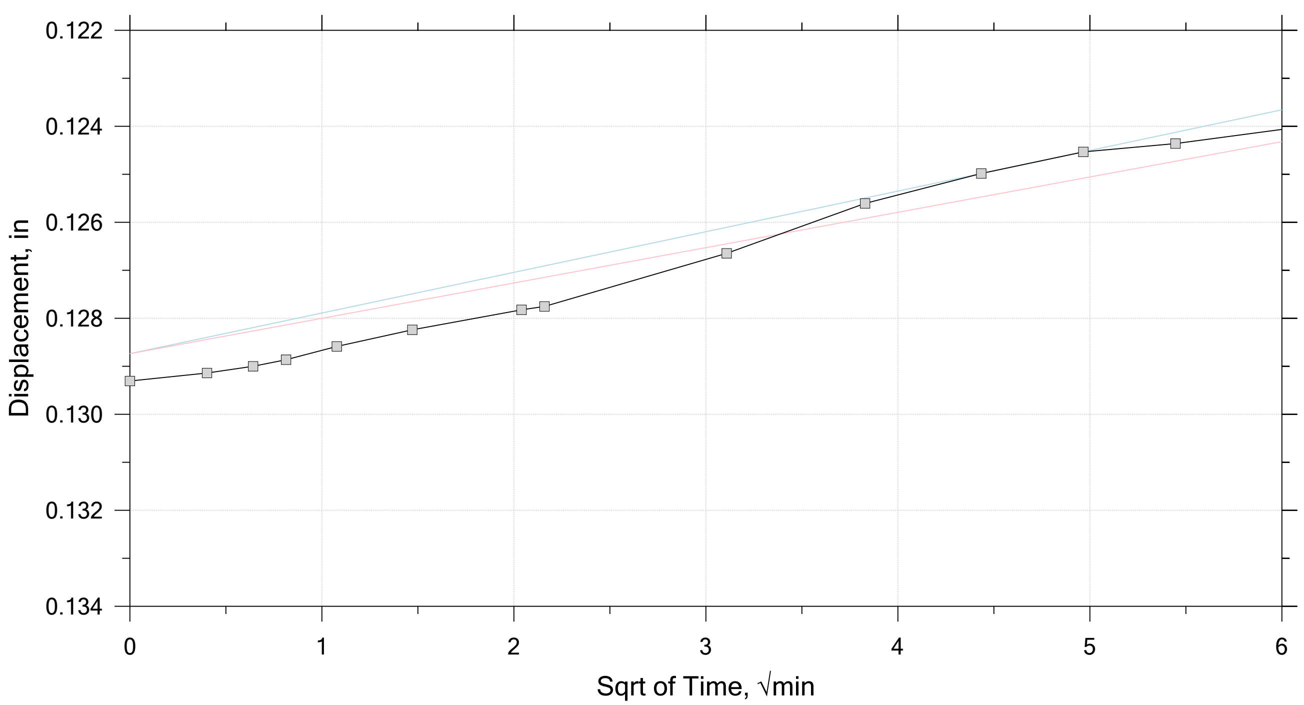
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 15

Constant Load Step

Stress: 225 psf





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter, in: 2.50	Specific Gravity: 2.84 (Implied)	Liquid Limit: 31
Specimen Height, in: 1.00	Initial Void Ratio: 0.973	Plastic Limit: 19
Final Height, in: 0.88	Final Void Ratio: 0.731	Plasticity Index: 12

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	207	---	"ring"	319
Mass Container, gm	36.81	110.17	110.17	60.59
Mass Container + Wet Soil, gm	144.36	264.76	255.88	206.11
Mass Container + Dry Soil, gm	118.79	226.01	226.01	176.28
Mass Dry Soil, gm	81.98	115.84	115.84	115.69
Water Content, %	31.19	33.45	25.78	25.78
Void Ratio	---	0.97	0.73	---
Degree of Saturation, %	---	97.50	100.00	---
Dry Unit Weight, pcf	---	89.723	102.25	---

Preconsolidation Stress, psf	2200
Compression Ratio	0.16
Rebound Ratio	0.02
Compression Index	0.3156
Rebound Index	0.03946

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.

	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
	Test Number: ICON307	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients

Step	Applied Stress psf	EOP Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv ft²/day	Mv ft²/ton	k cm/s
1	100.	0.002737	0.967	0.273	30.731	6.91e-02	5.46e+04	4.16e-08
2	150.	0.004193	0.965	0.418	46.321	4.56e-02	5.81e+04	2.92e-08
3	225.	0.007311	0.958	0.730	31.754	6.63e-02	8.30e+04	6.06e-08
4	338.	0.01022	0.953	1.02	22.008	9.50e-02	5.14e+04	5.38e-08
5	506.	0.01414	0.945	1.41	36.101	5.75e-02	4.65e+04	2.95e-08
6	759.	0.01922	0.935	1.92	22.743	9.05e-02	4.01e+04	4.00e-08
7	1.14e+03	0.02599	0.922	2.59	47.880	4.25e-02	3.55e+04	1.66e-08
8	1.71e+03	0.03548	0.903	3.54	41.401	4.83e-02	3.32e+04	1.77e-08
9	2.56e+03	0.04934	0.876	4.92	40.948	4.77e-02	3.24e+04	1.70e-08
10	5.13e+03	0.09667	0.782	9.65	33.610	5.44e-02	3.69e+04	2.21e-08
11	1.03e+04	0.1490	0.679	14.9	138.395	1.18e-02	2.04e+04	2.66e-09
12	2.56e+03	0.1416	0.694	14.1	2.558	6.08e-01	1.93e+03	1.29e-08
13	1.14e+03	0.1373	0.702	13.7	18.150	8.69e-02	6.04e+03	5.78e-09
14	506.	0.1302	0.716	13.0	23.609	6.77e-02	2.24e+04	1.67e-08
15	225.	0.1228	0.731	12.3	47.553	3.42e-02	5.26e+04	1.98e-08

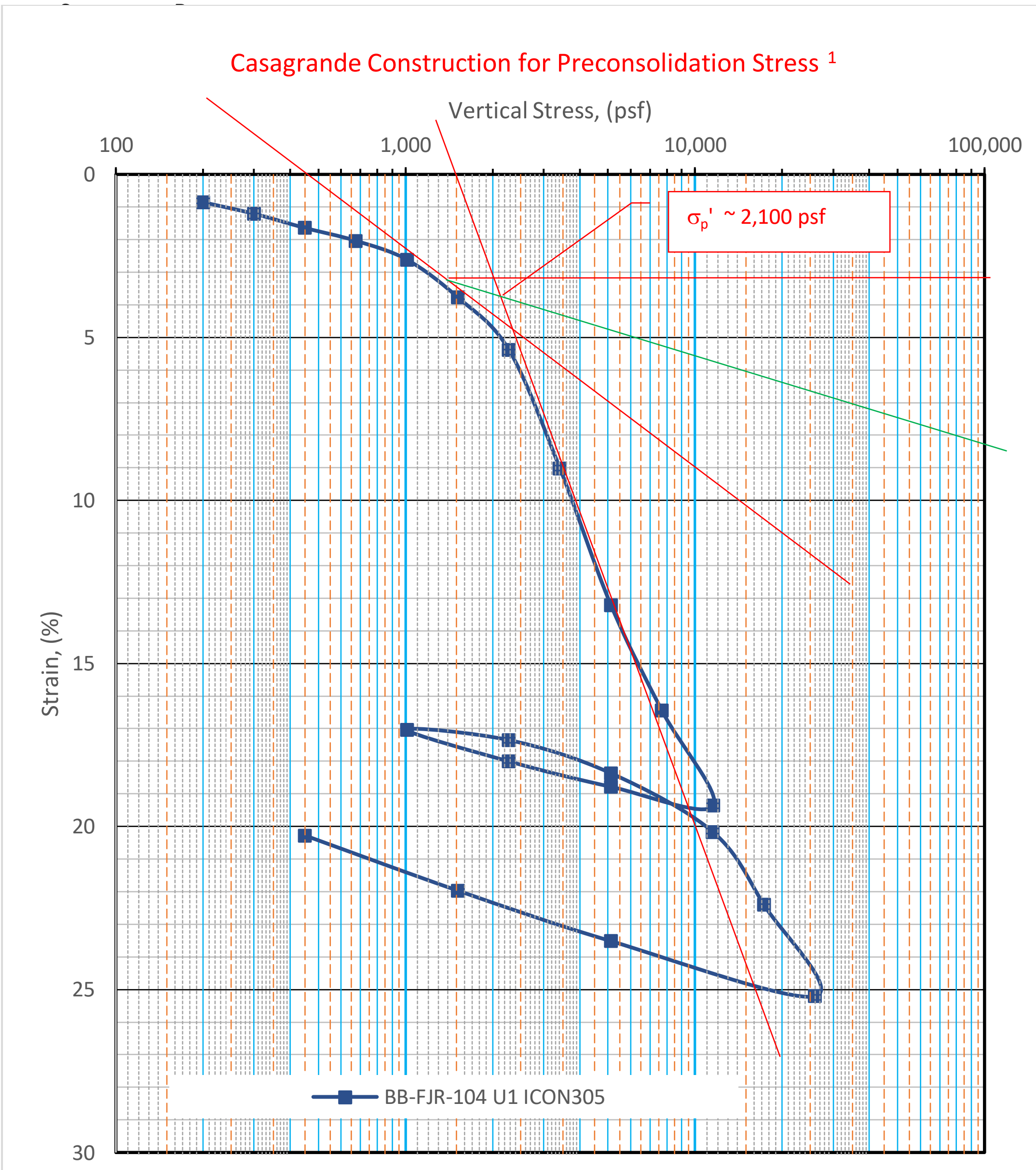
Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-102A	Tester: SJR	Checker: SJR
Sample Number: U2	Test Date: 9/27/19	Depth: 41.3
Test Number: ICON307	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks:		
Displacement at End of Primary		

BB-FJR-104 1U

Consolidation Test Data

Project Name:	Johnson Road Bridge
Project Number:	166-11
Project Location:	Falmouth, Maine
Client:	GZA Proj. No. 09.0026024.00
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	ICON 305			
Boring No.	BB-FJR-104			
Sample No:	1U			
Boring Elevation (ft).	36.9			
Sample Depth (ft):	30 - 32			
Test Specimen Depth (Ft):	31.05			
Test Specimen Elevation:	5.85			
Water Content (%):	44.32			
Dry Unit Weight (pcf):	76.05			
Wet Unit Weight (pcf):	109.76			
Saturation Before (%):	97.75			
Saturation After (%):	100			
Void Ratio Before:	1.39			
Void Ratio After:	0.9			
Overburden Pressure (psf):				
Max Previous stress (psf):	2,100			
Max Prev. stress (Work) (psf):	2,000			
OCR:				
Compression Index (C_{CE}):	0.24			
Recompression Index (C_{RE}):	0.025			
Liquid Limit:	42			
Plastic Limit:	23			
Plasticity Index:	19			
Liquidity Index:	1.1			
Specific Gravity (implied)	2.91			
Lab Vane Su at __ ft. (psf)				
Tested By:	sjr			
Date Tested:	9/22/2019			
Checked By:	sjr			

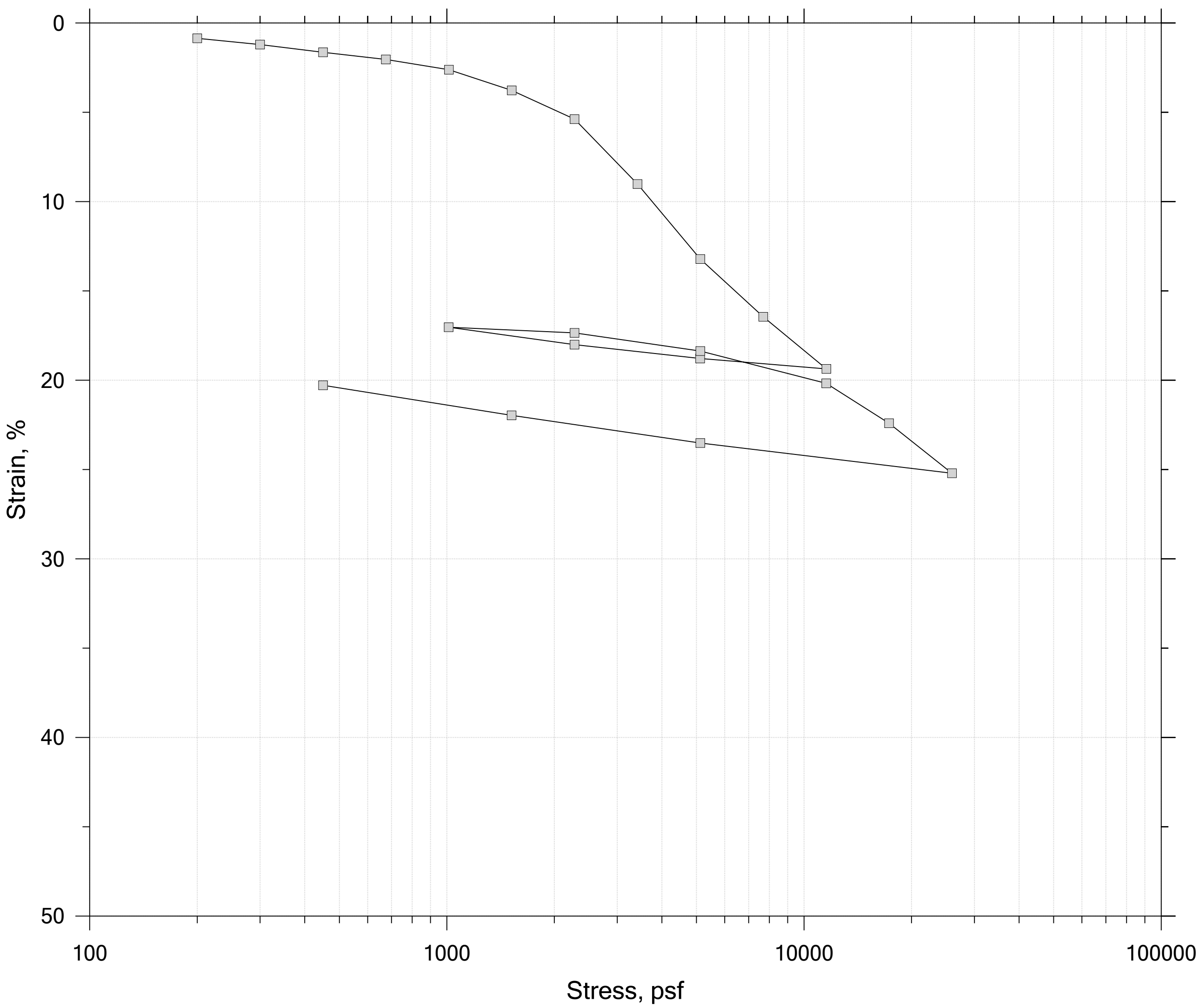


Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convienience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.

Soil Metrics LLC Geotechnical Engineering

One-Dimensional Consolidation by ASTM D2435 - Method B

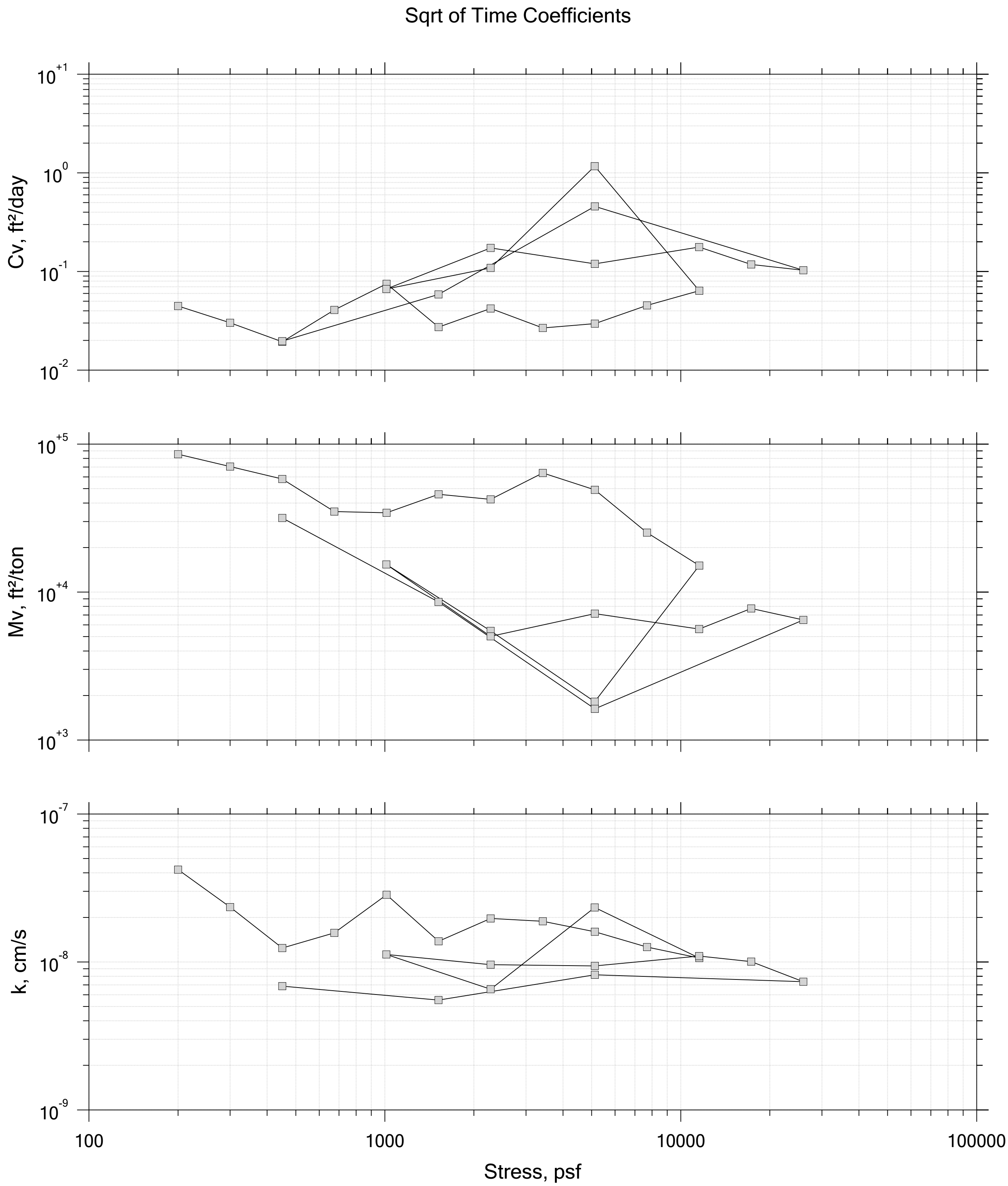
Summary Report



					Before Test	After Test
Current Vertical Effective Stress, psf: 0				Water Content, %	46.63	31.06
Preconsolidation Stress, psf: 2100				Dry Unit Weight, pcf	76.053	95.398
Compression Ratio: 0.24				Saturation, %	97.75	100.00
Specimen Diameter, in: 2.5		Specimen Height, in: 1.002		Void Ratio	1.39	0.90
LL: 42	PL: 23	PI: 19	GS: 2.91			

	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		
	Displacement at End of Primary		

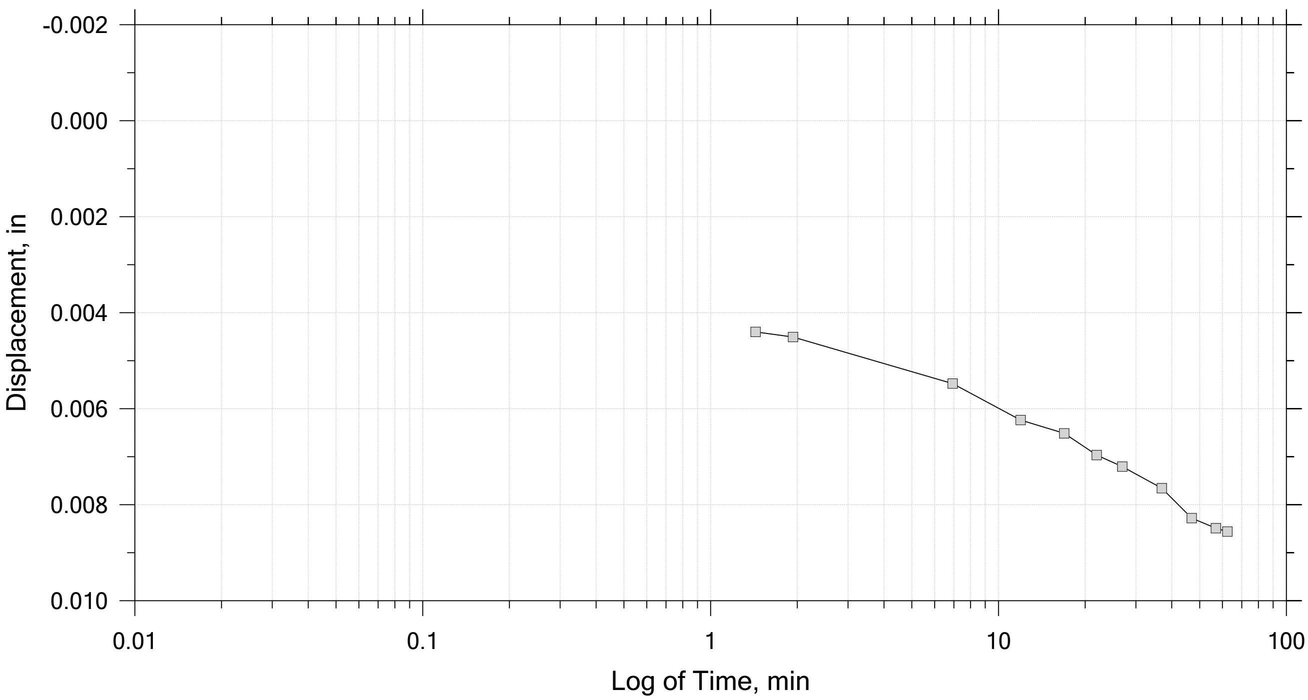
One-Dimensional Consolidation by ASTM D2435 - Method B

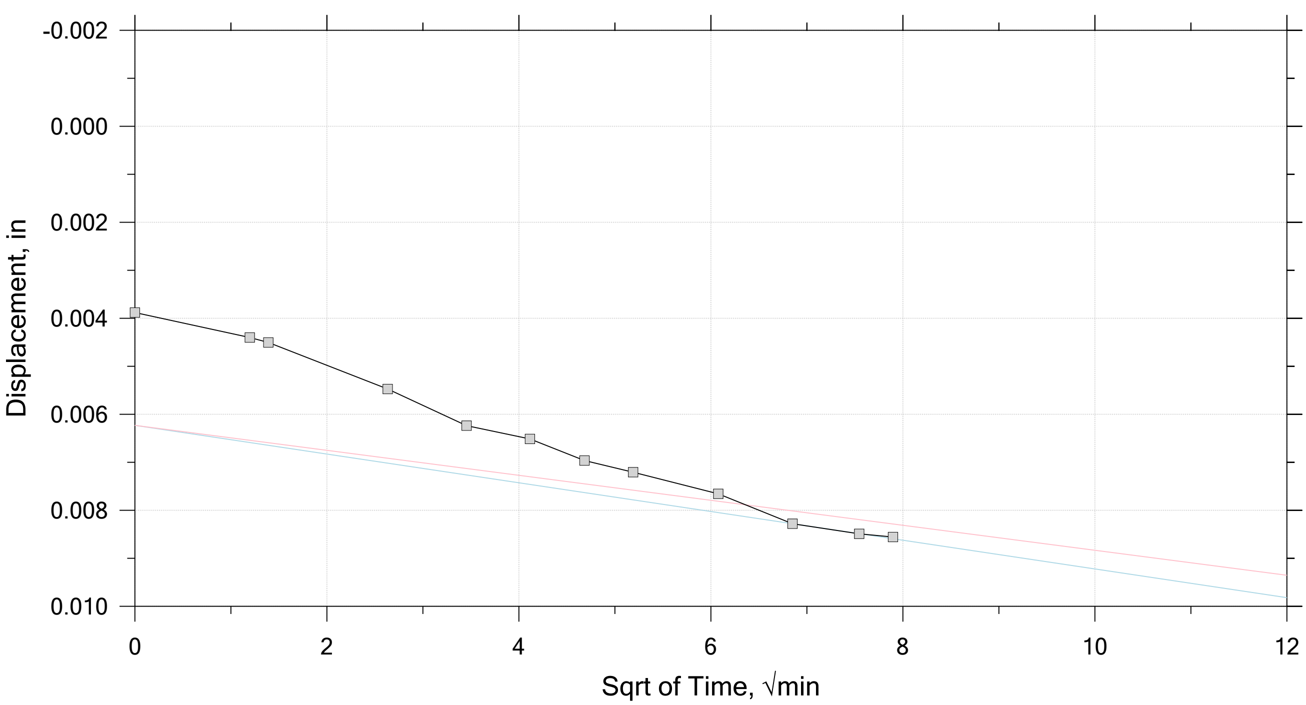


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 22
Constant Load Step
Stress: 200 psf





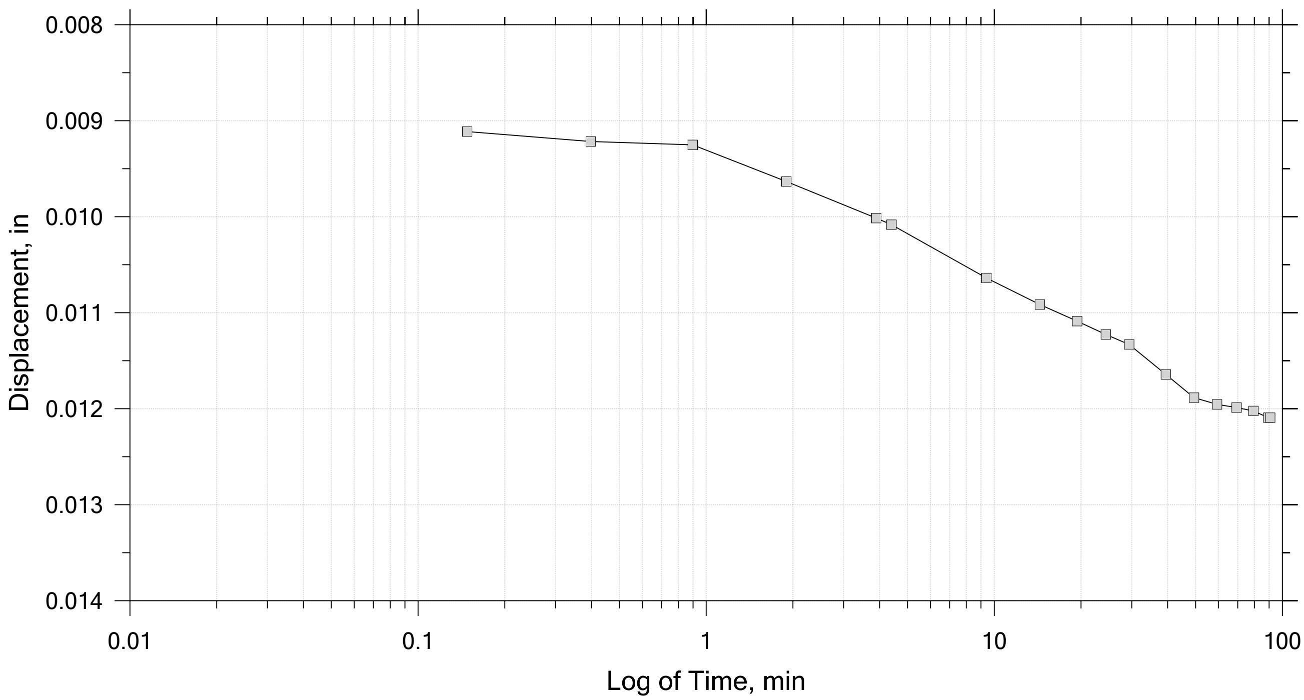
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

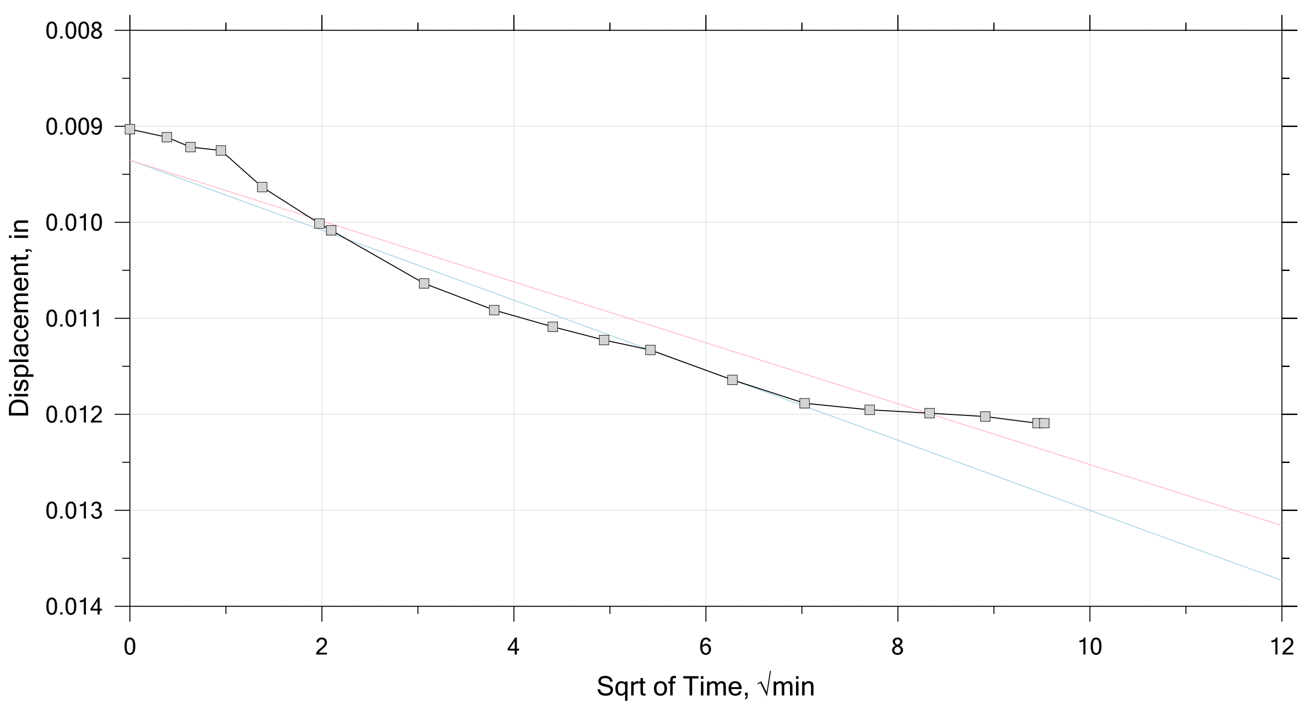
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 22

Constant Load Step

Stress: 300 psf





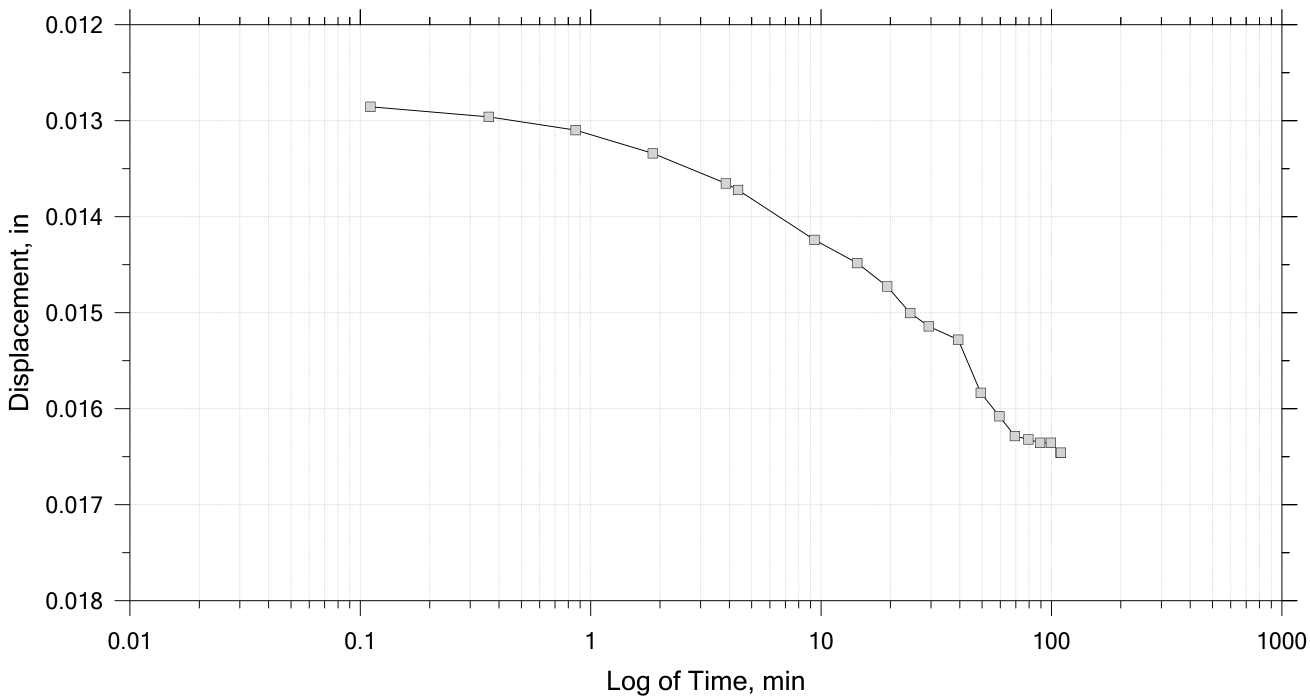
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

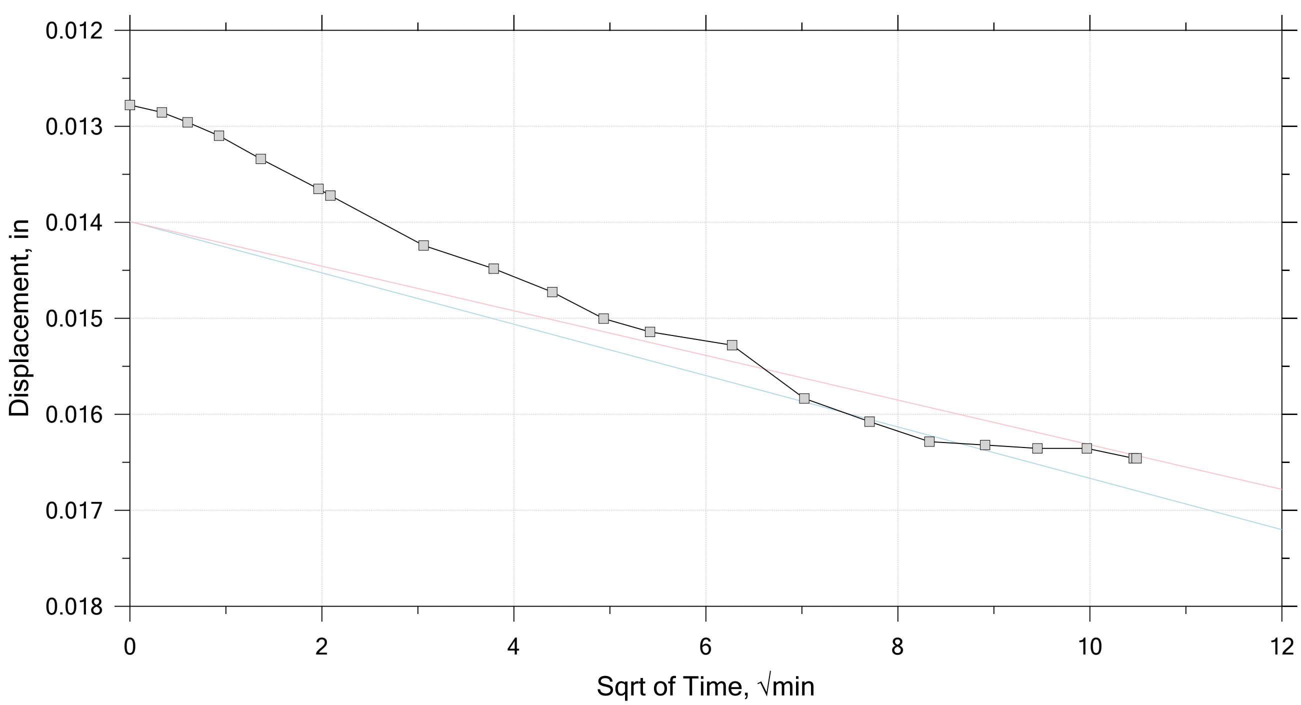
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 22

Constant Load Step

Stress: 450 psf





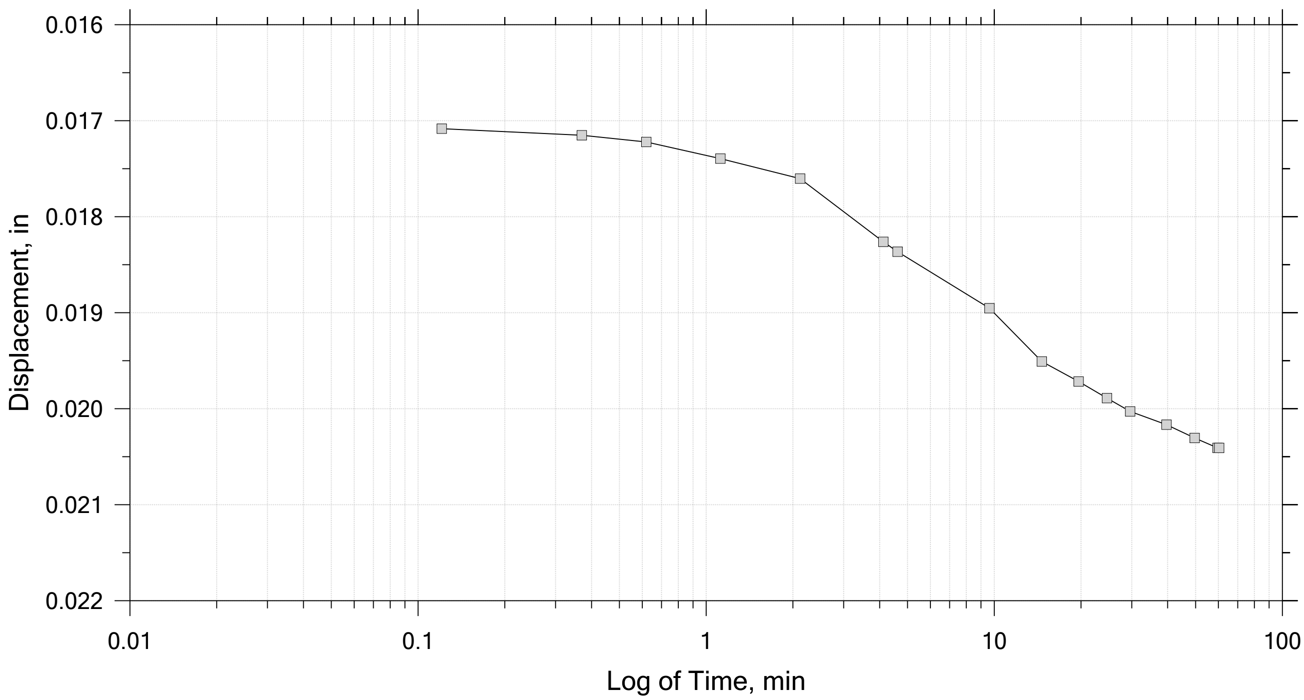
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

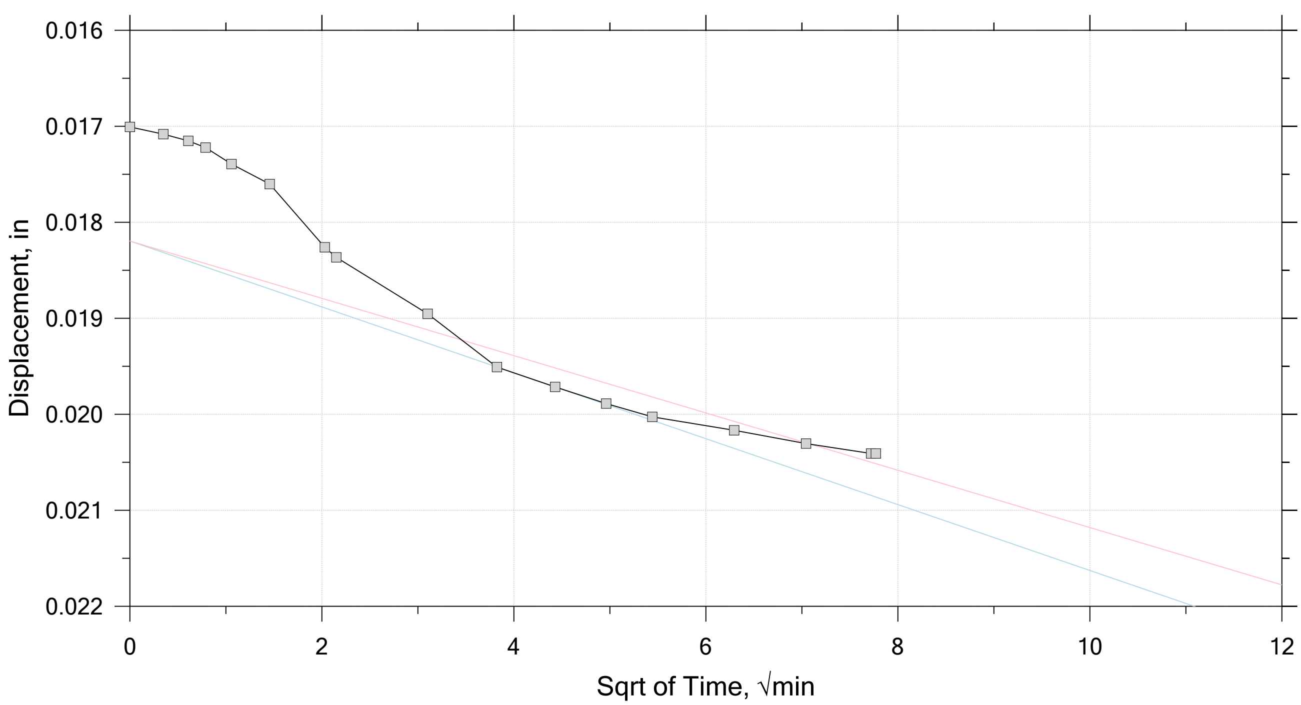
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 22

Constant Load Step

Stress: 675 psf





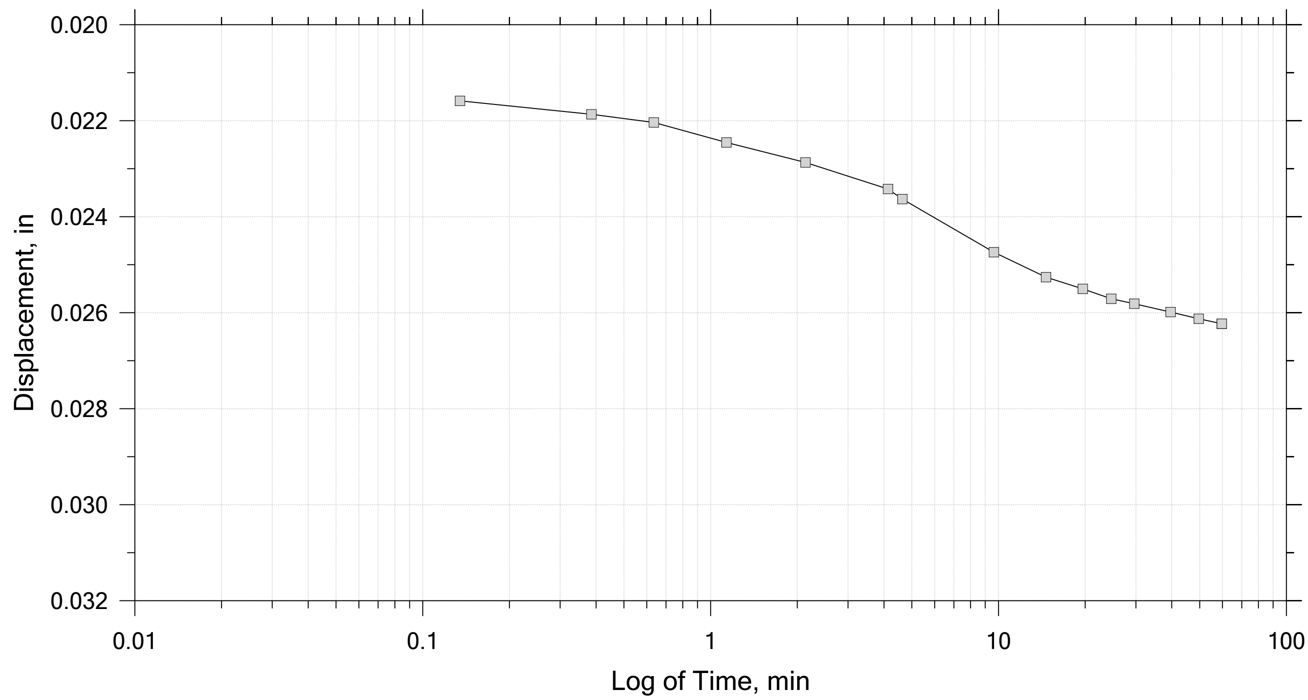
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

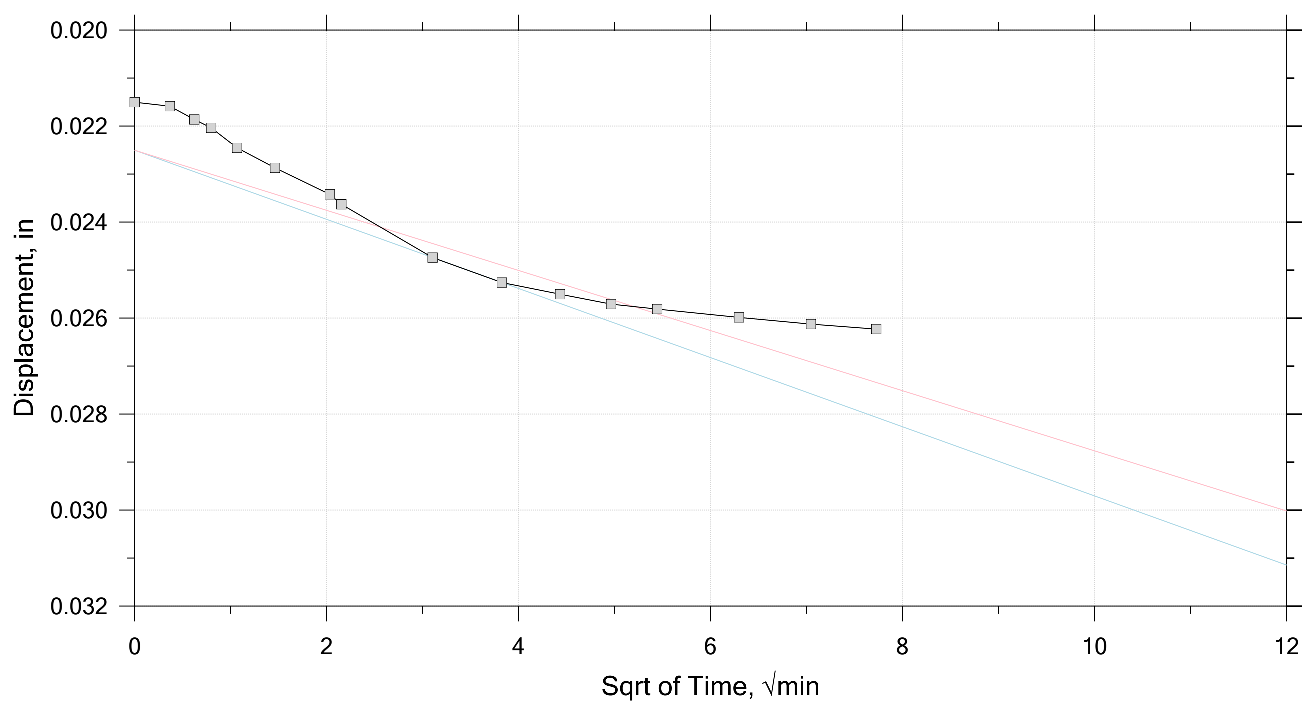
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 22

Constant Load Step

Stress: 1.01e+03 psf





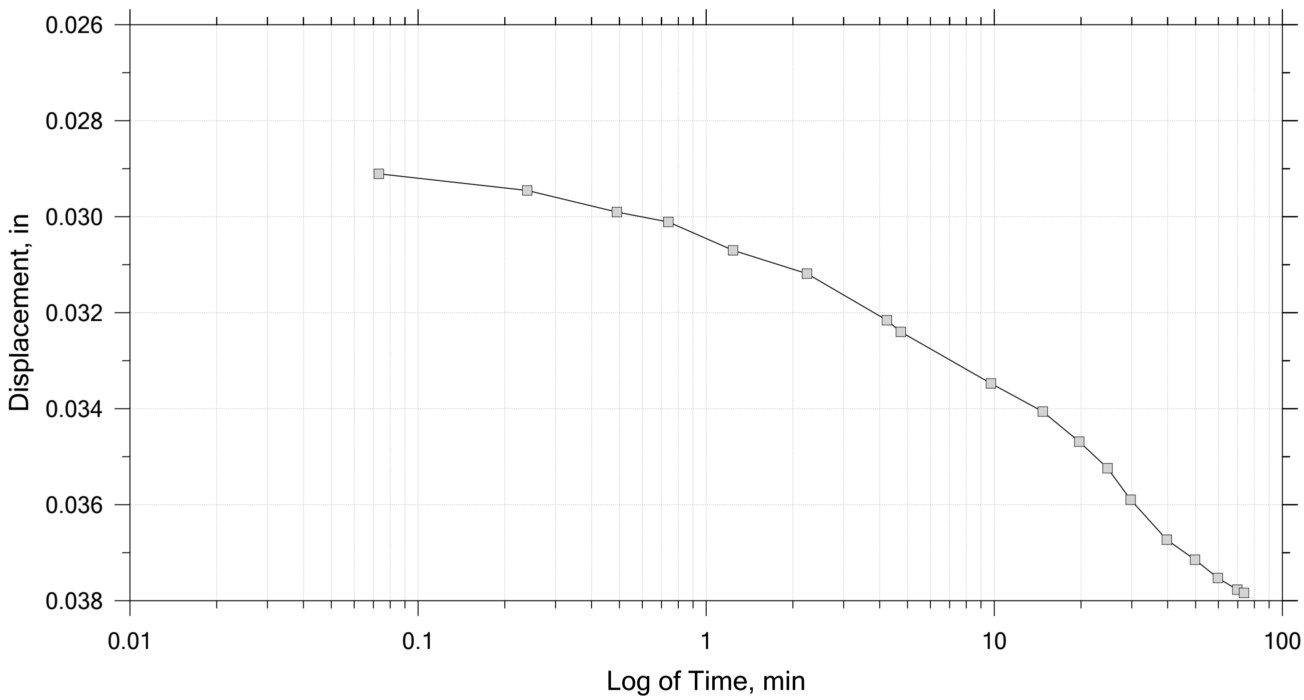
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

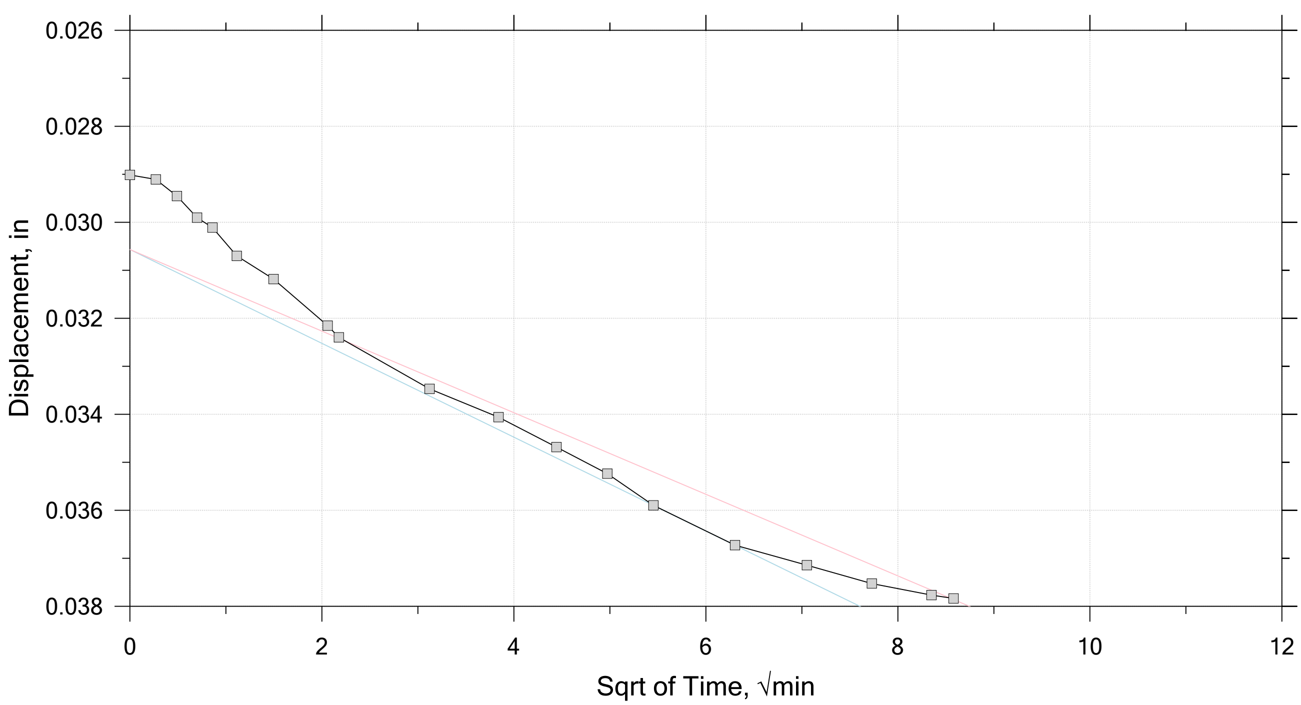
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 22

Constant Load Step

Stress: 1.52e+03 psf





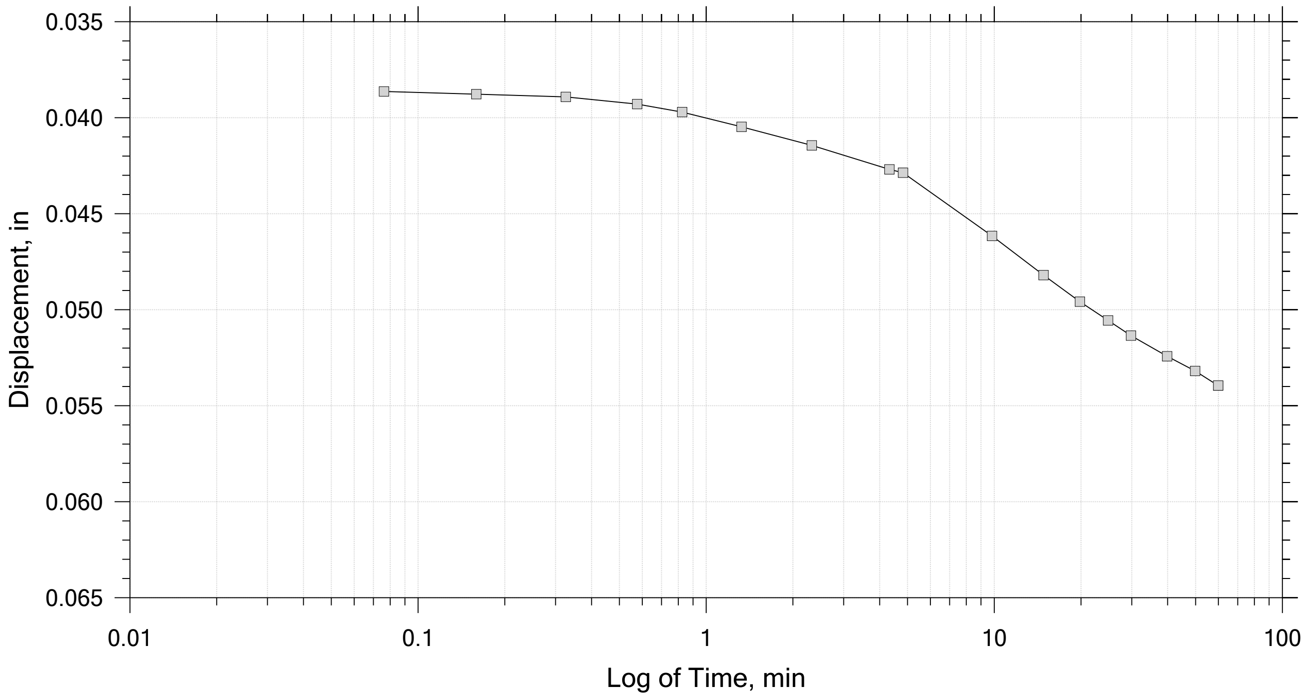
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

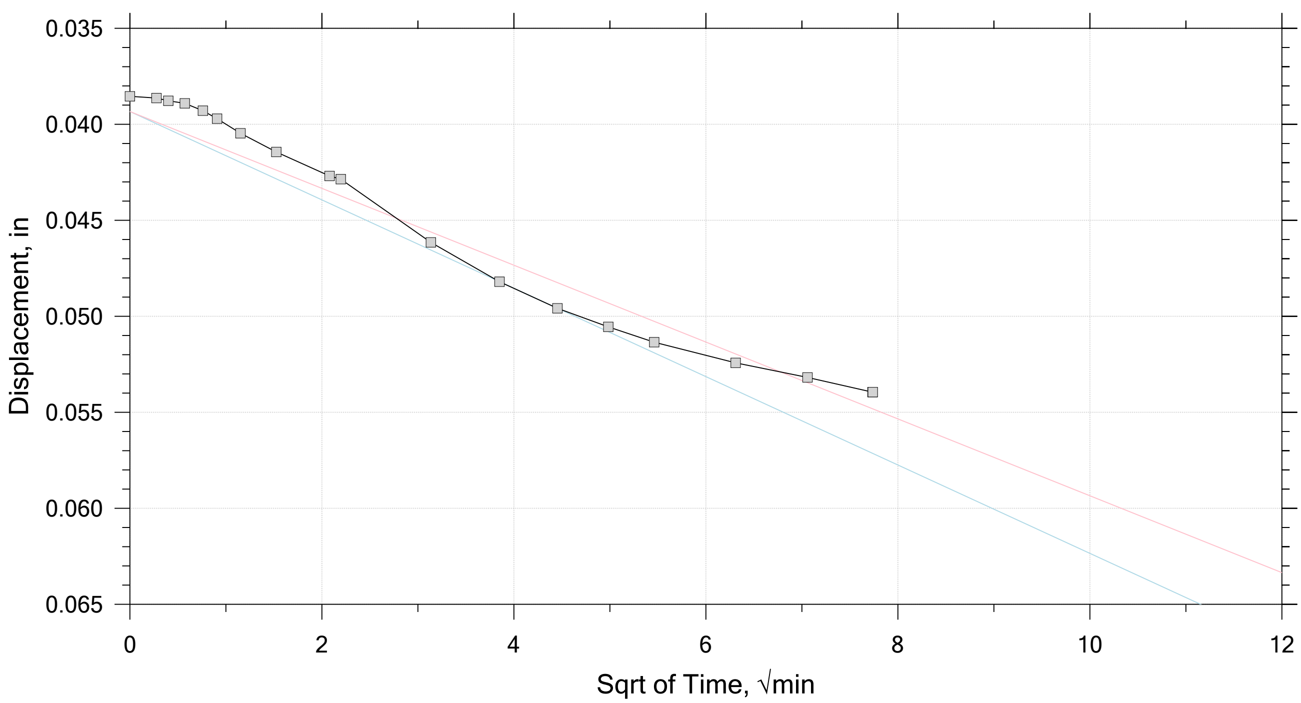
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 22

Constant Load Step

Stress: 2.28e+03 psf





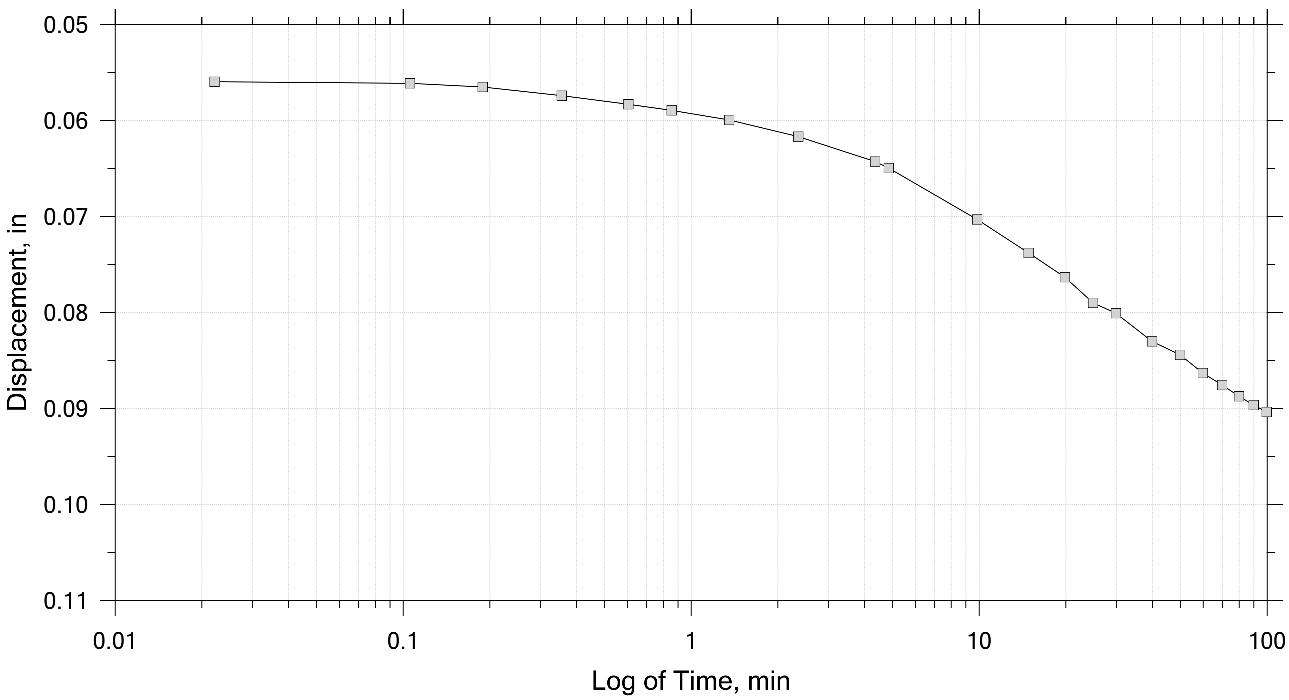
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

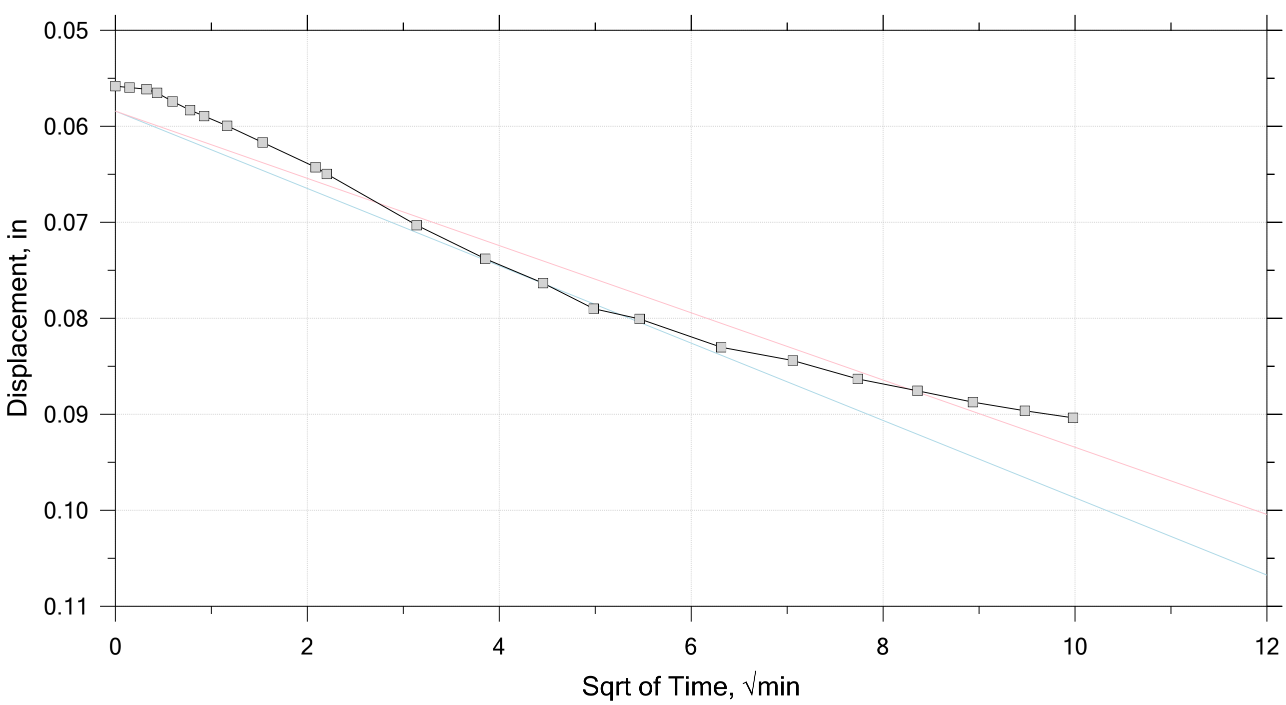
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 22

Constant Load Step

Stress: 3.42e+03 psf

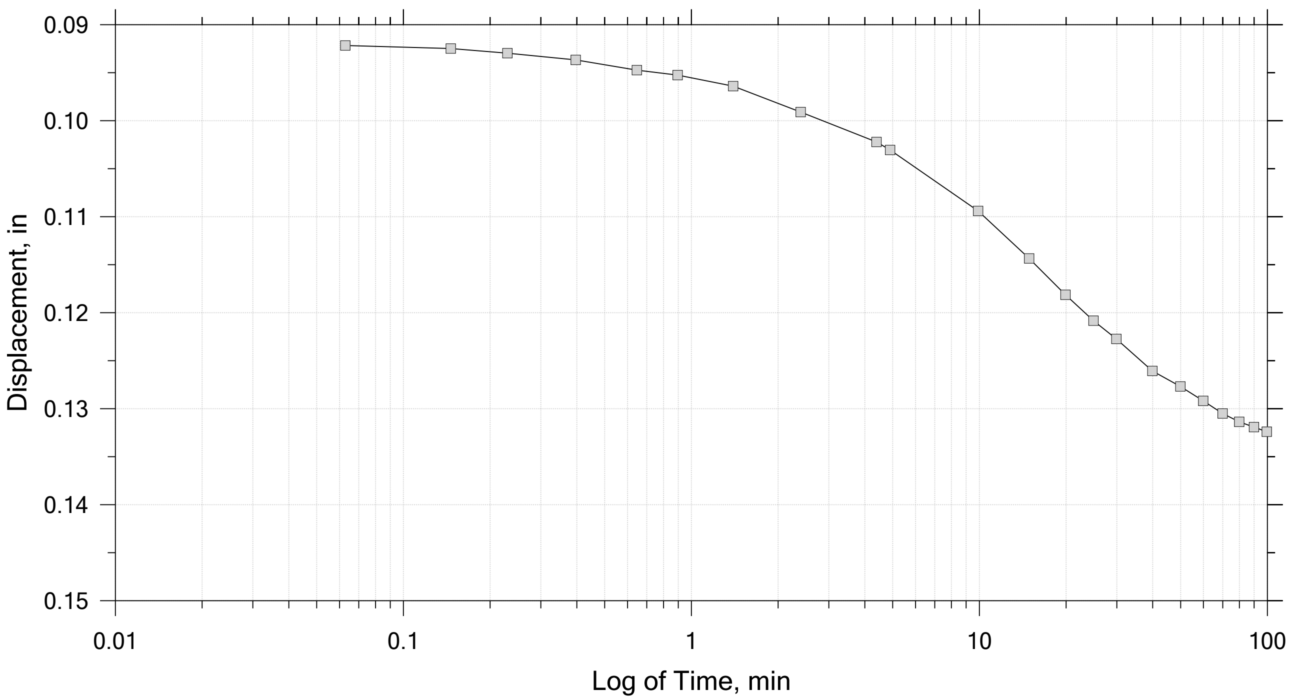


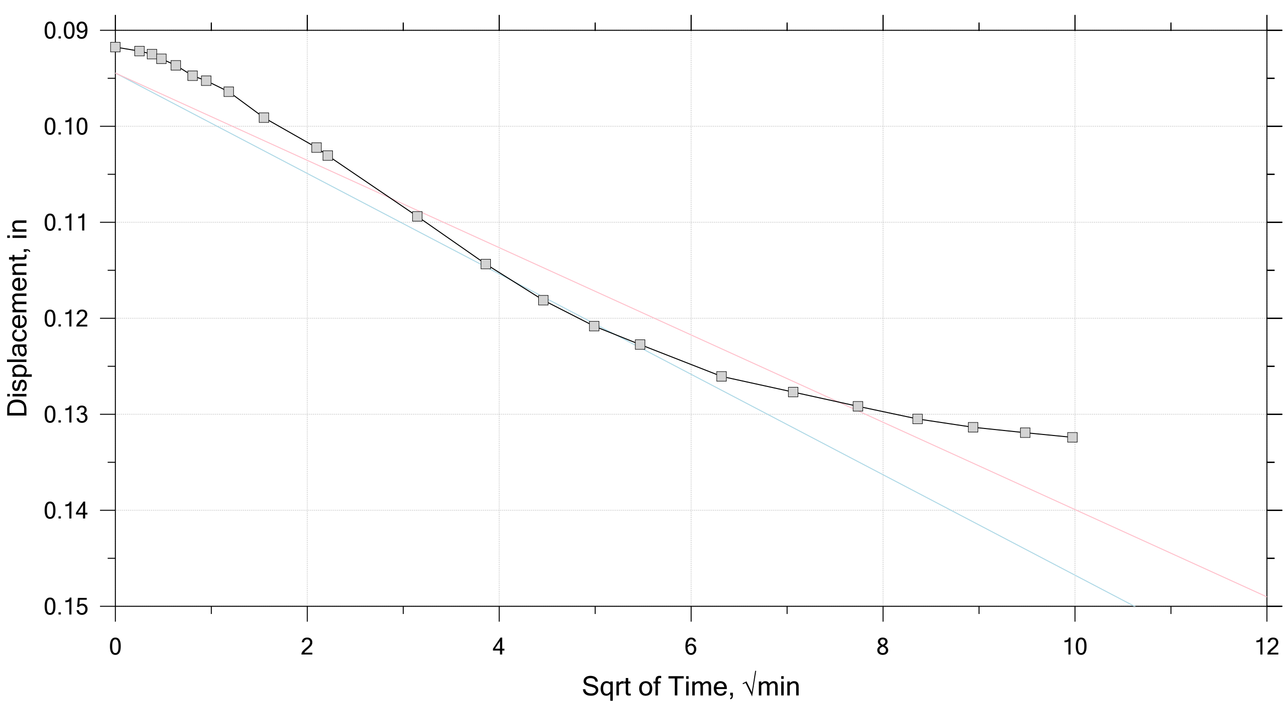


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 22
Constant Load Step
Stress: 5.13e+03 psf

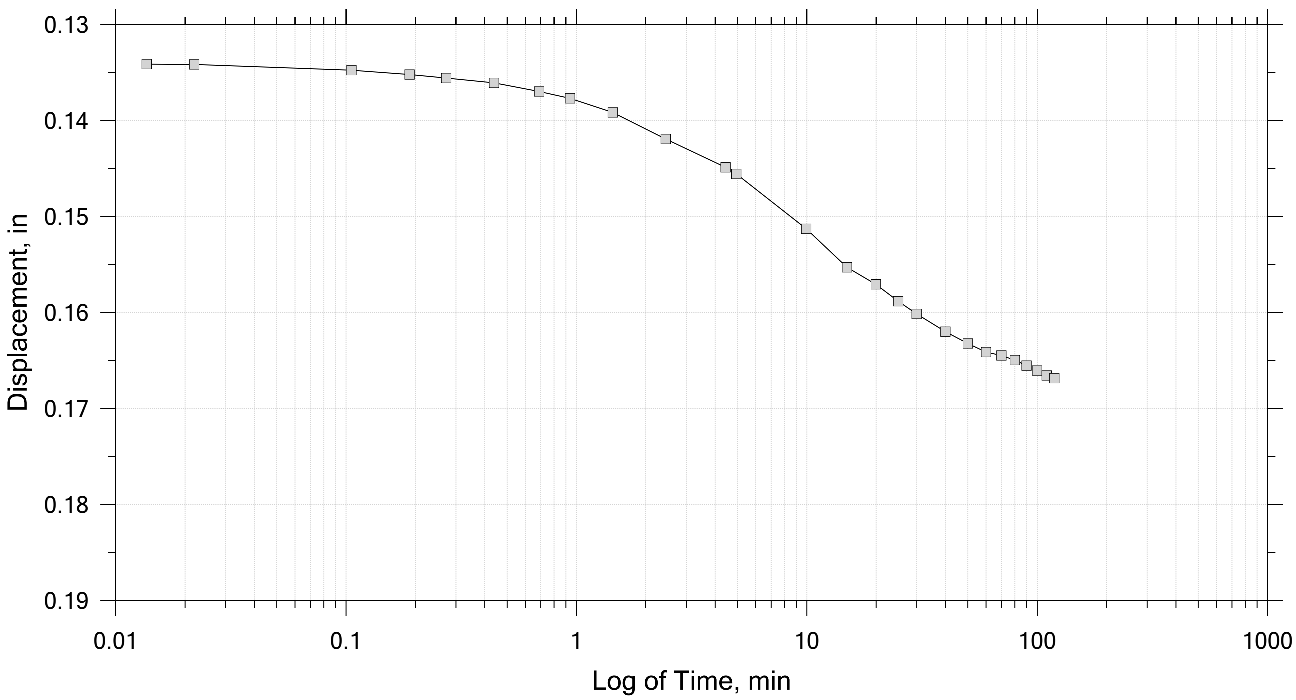


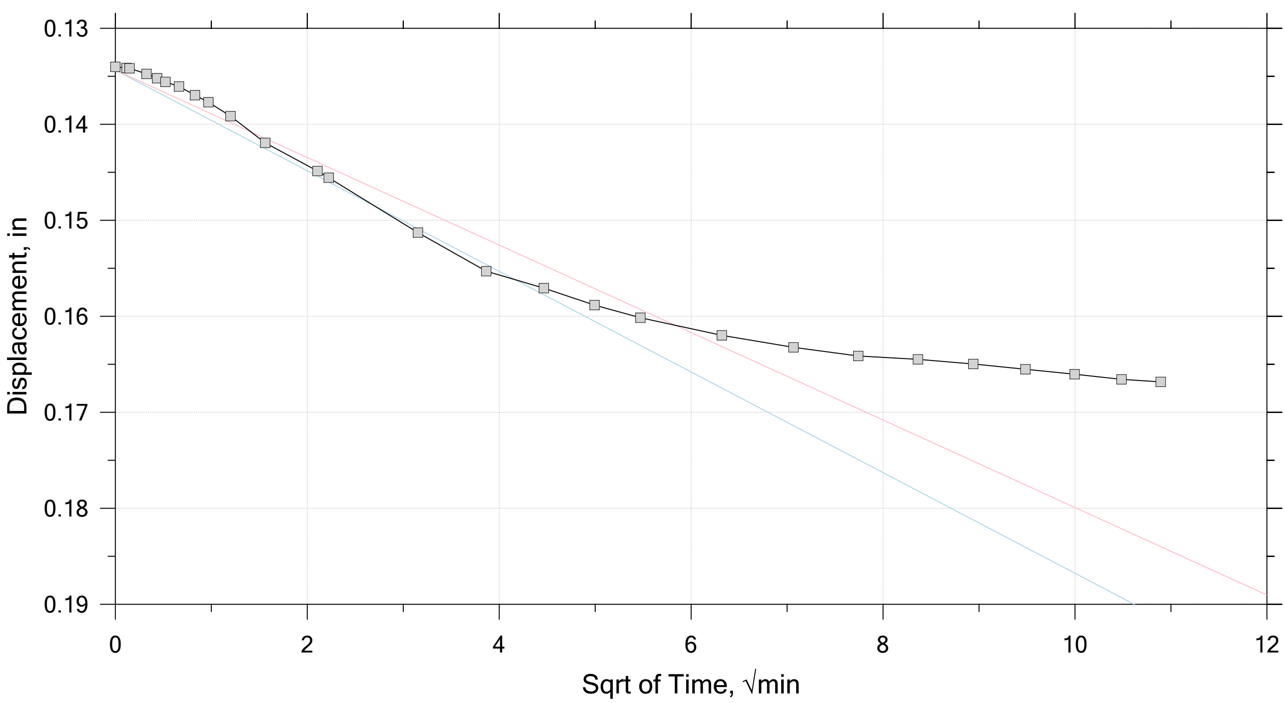


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 22
Constant Load Step
Stress: 7.69e+03 psf





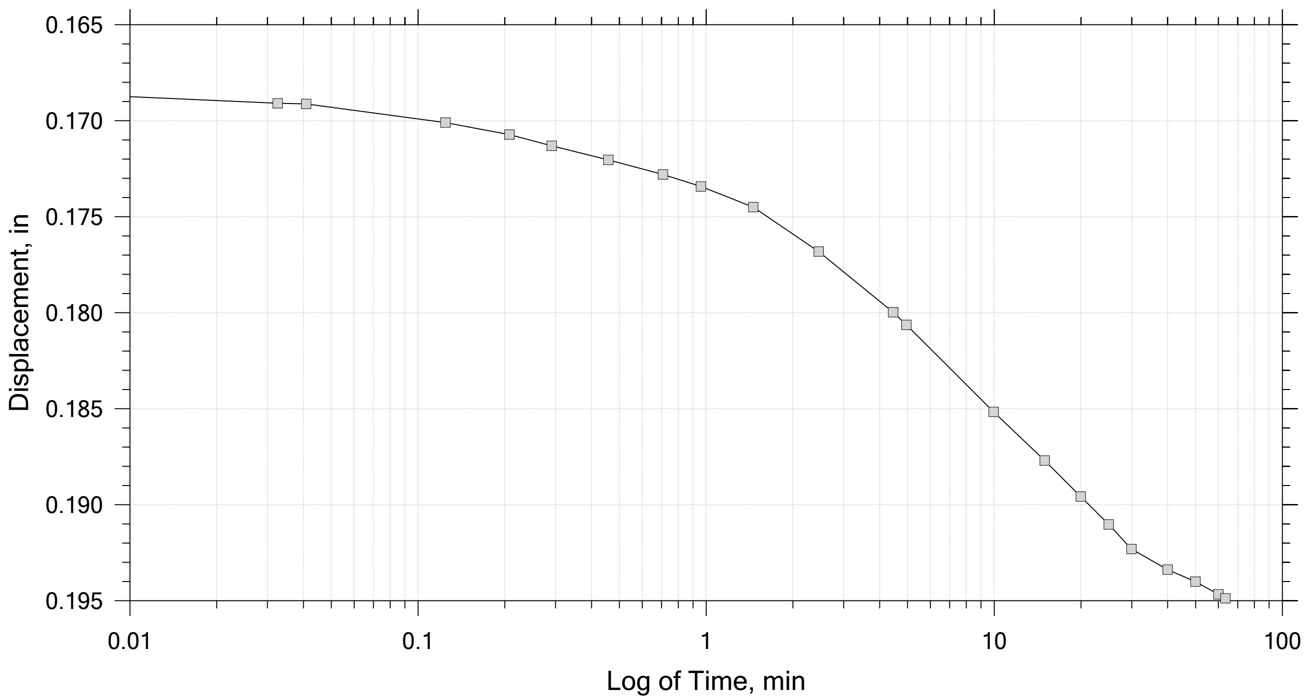
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

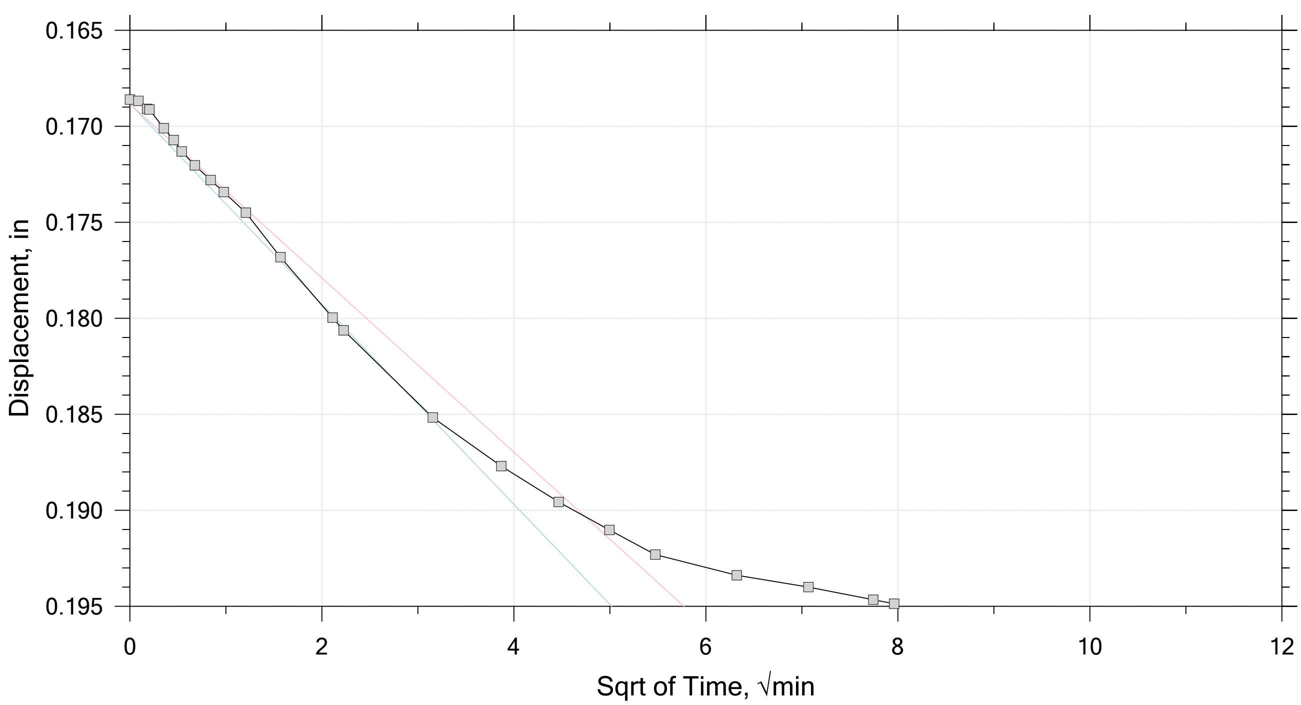
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 22

Constant Load Step

Stress: 1.16e+04 psf

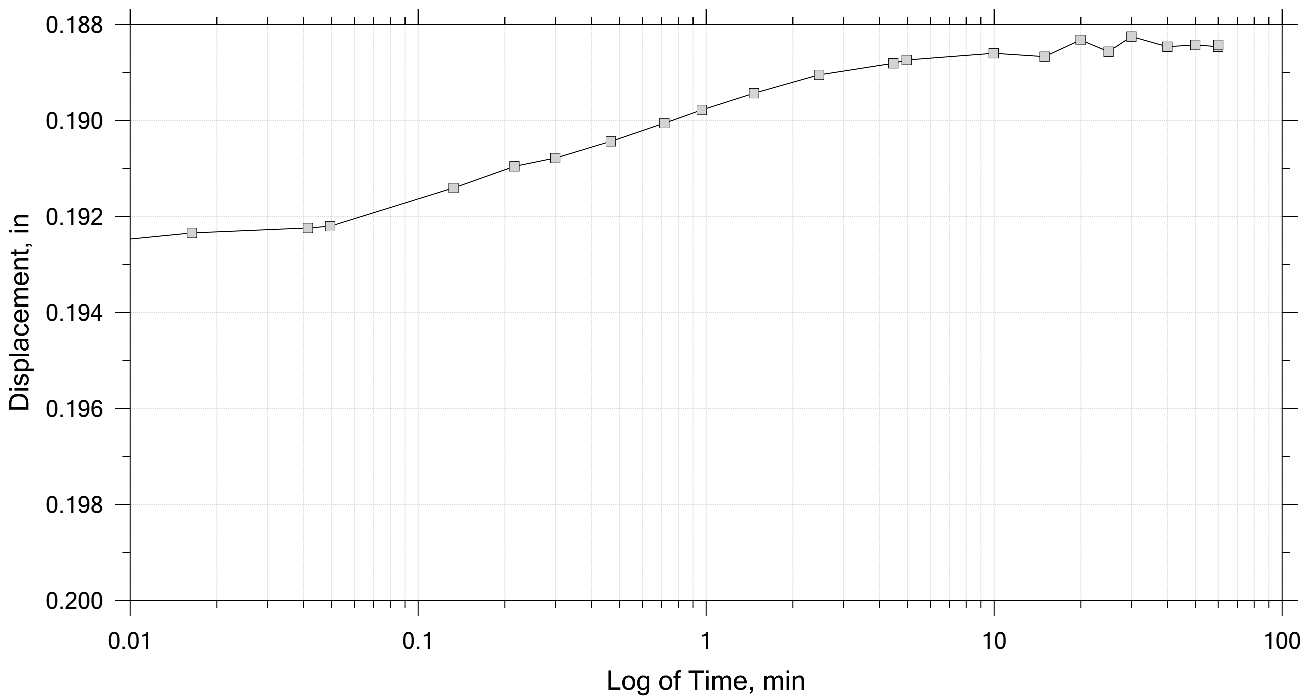


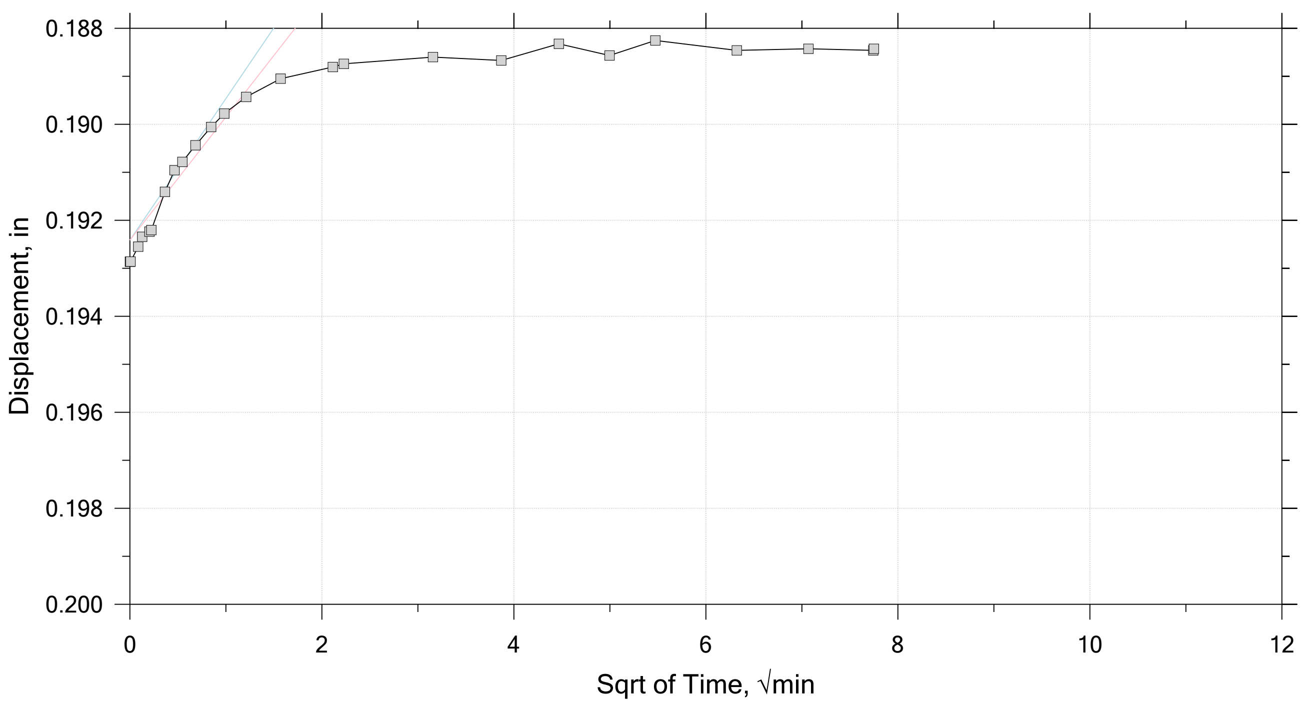


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 22
Constant Load Step
Stress: 5.12e+03 psf

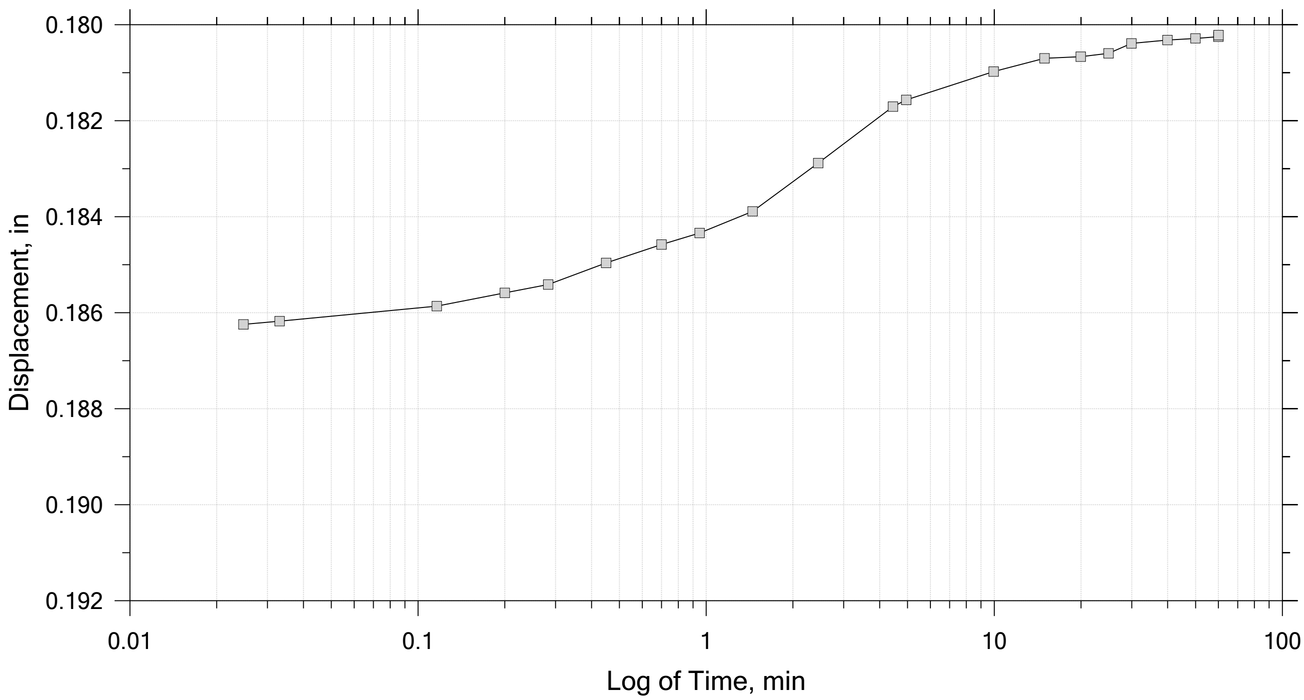


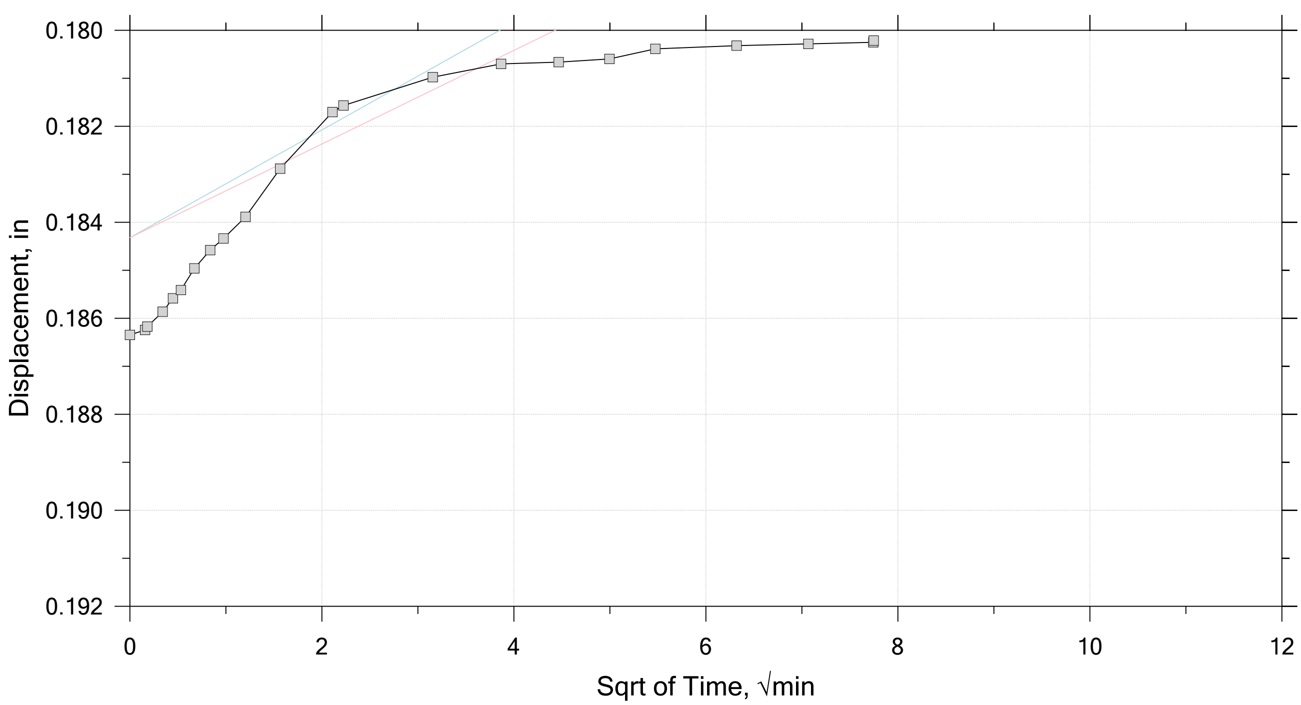


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 22
Constant Load Step
Stress: 2.28e+03 psf

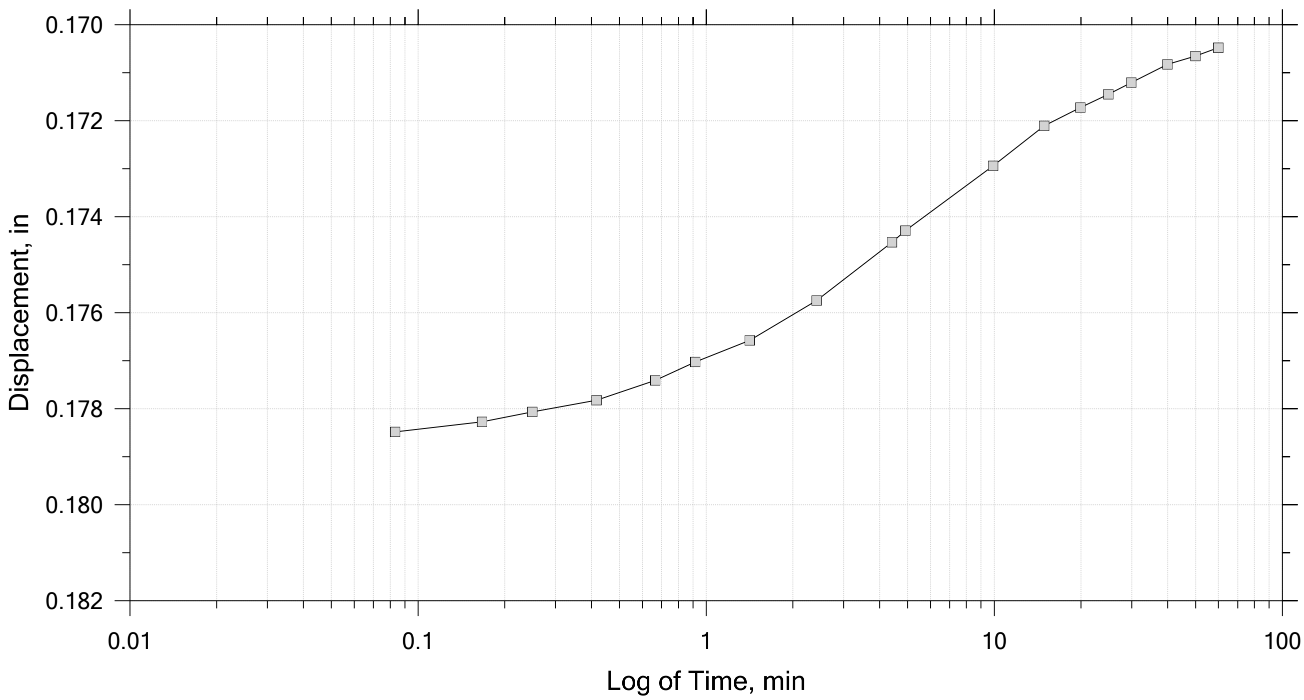


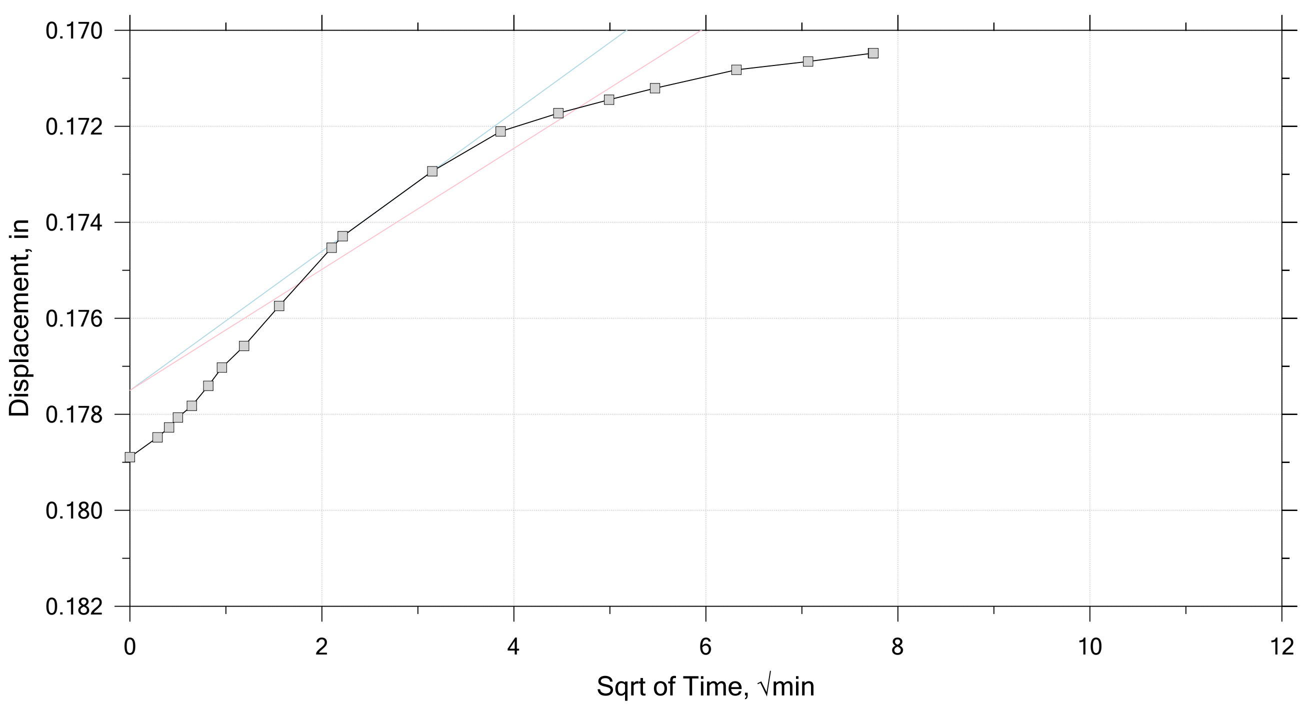


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 22
Constant Load Step
Stress: 1.01e+03 psf

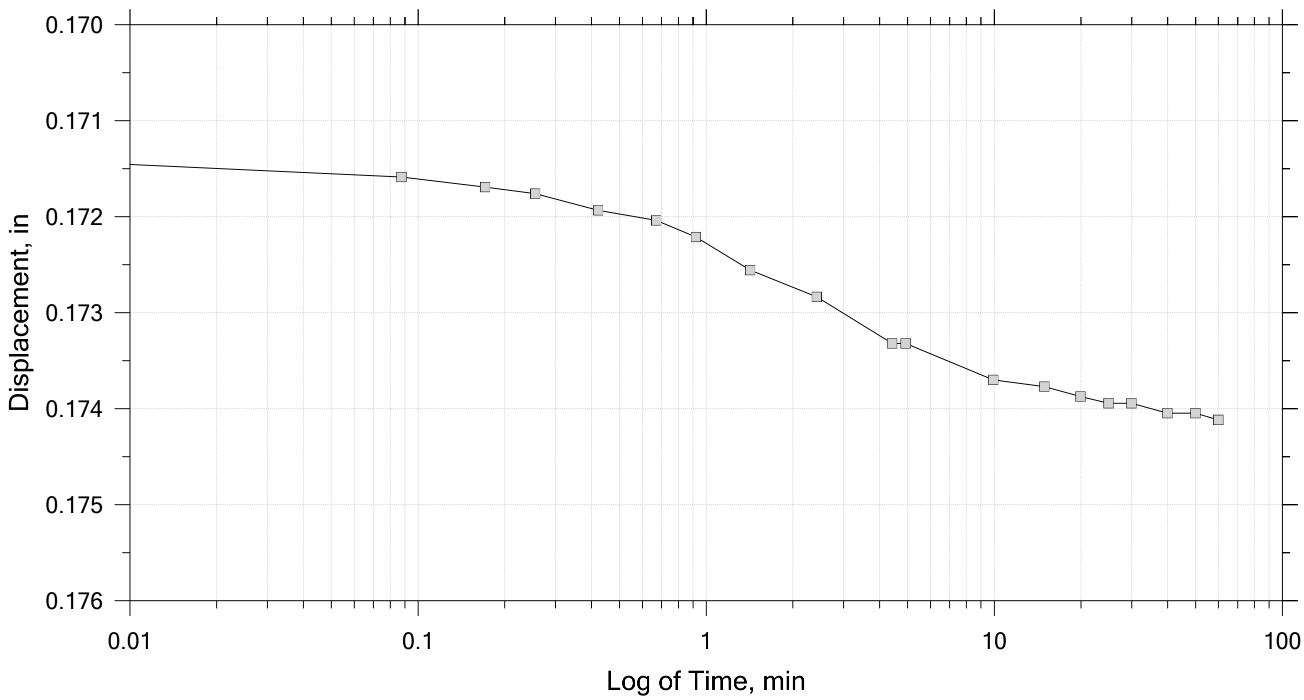


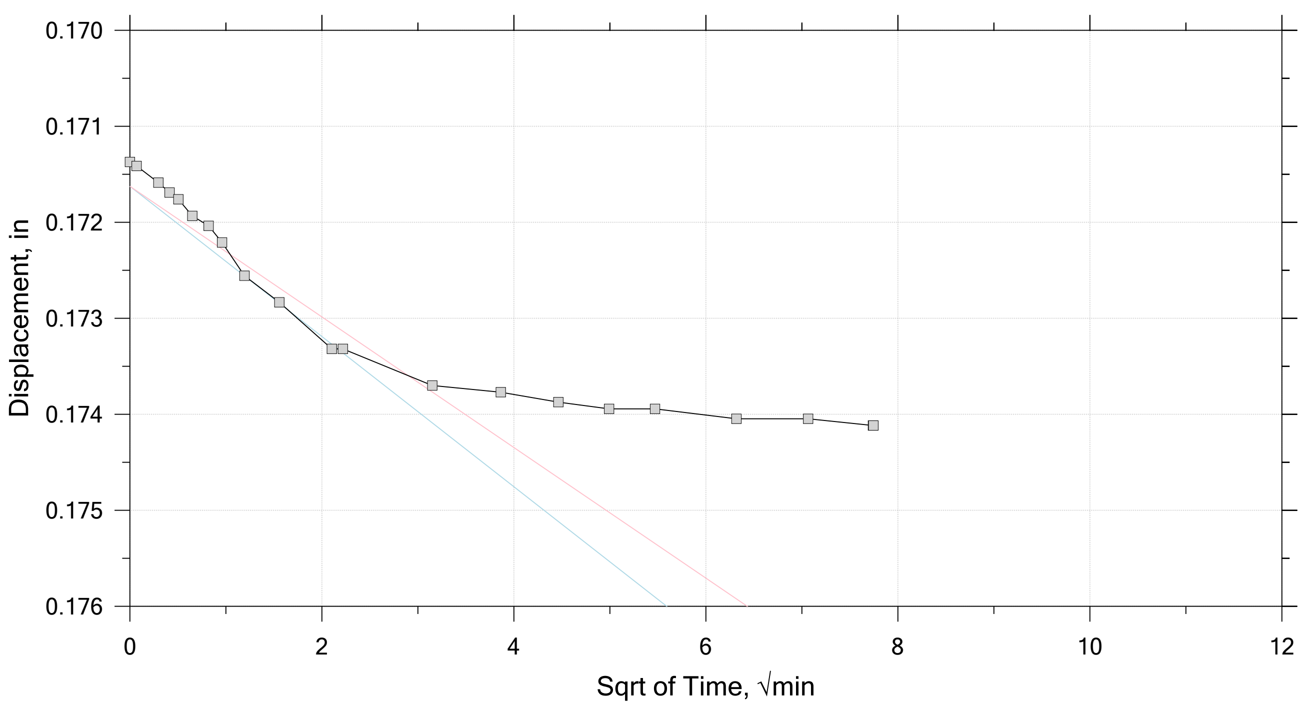


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 22
Constant Load Step
Stress: 2.28e+03 psf





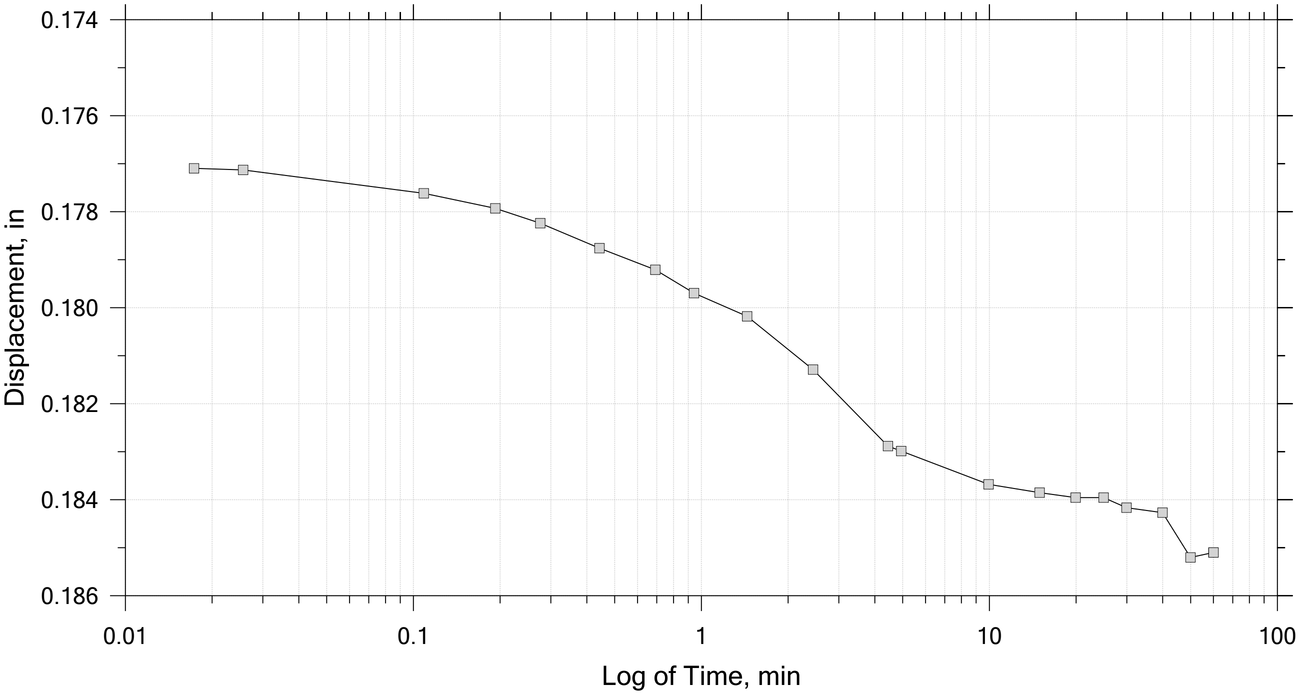
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

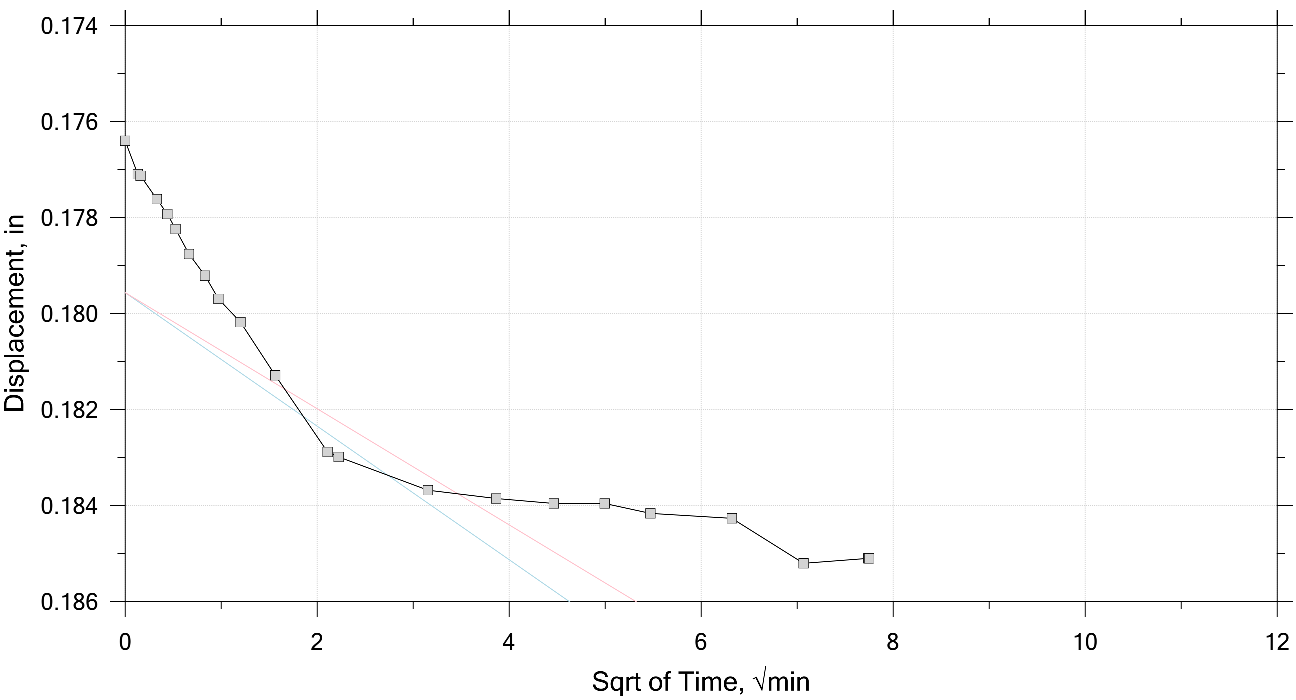
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 22

Constant Load Step

Stress: 5.13e+03 psf

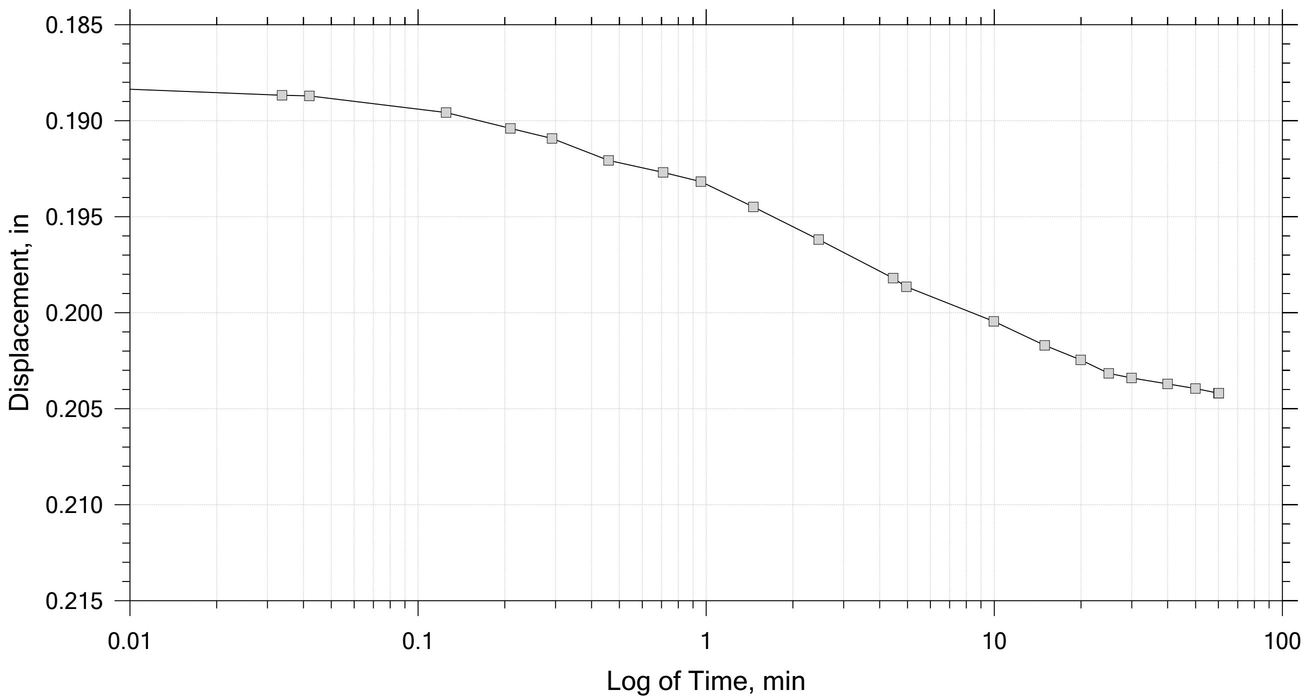


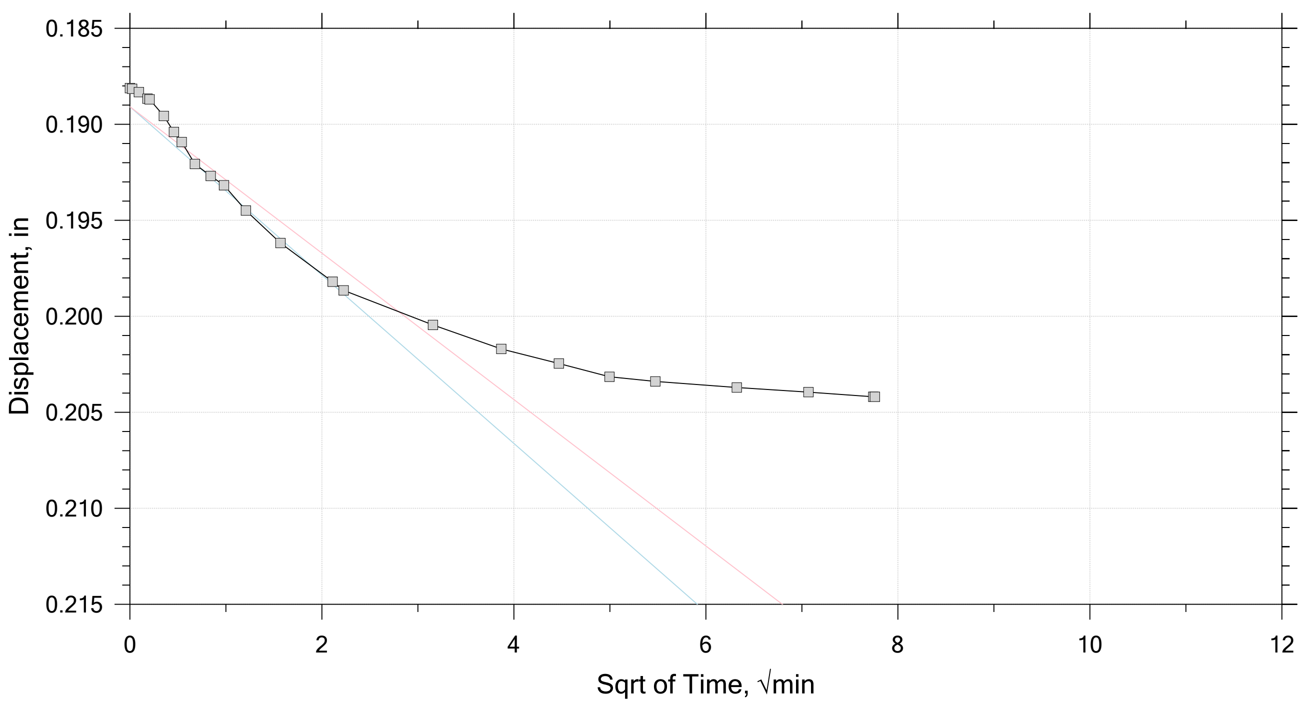


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 22
Constant Load Step
Stress: 1.15e+04 psf





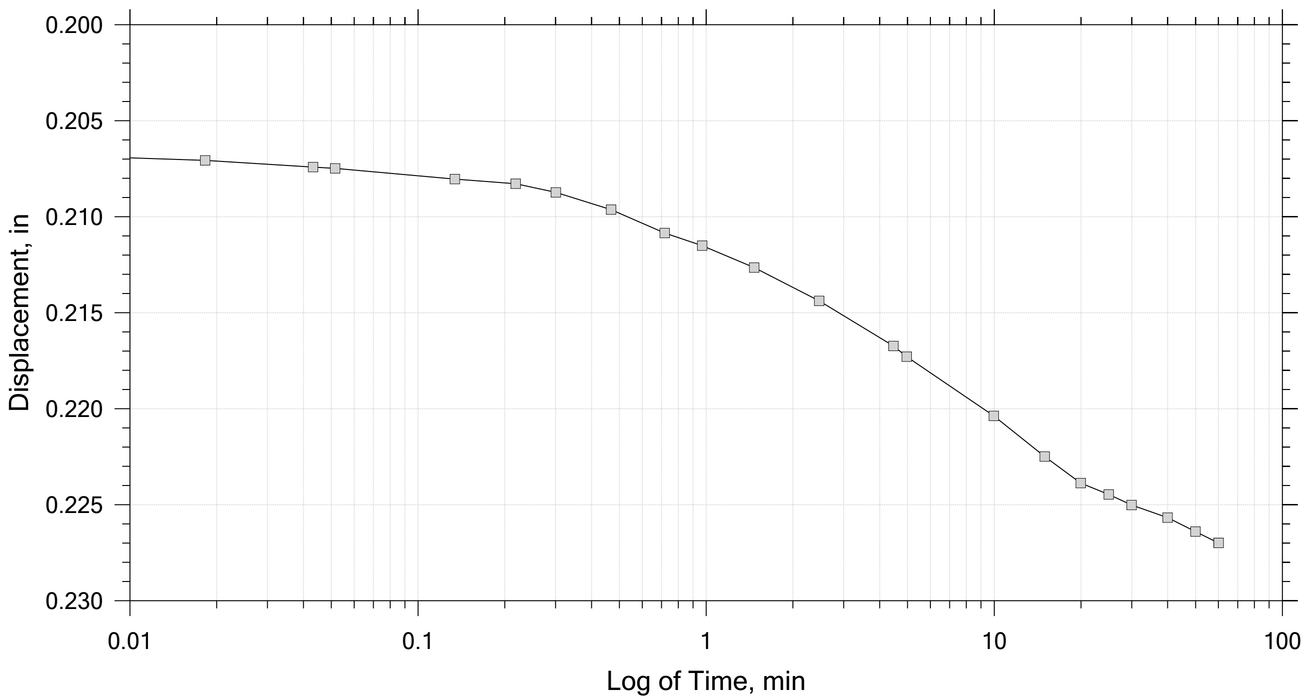
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

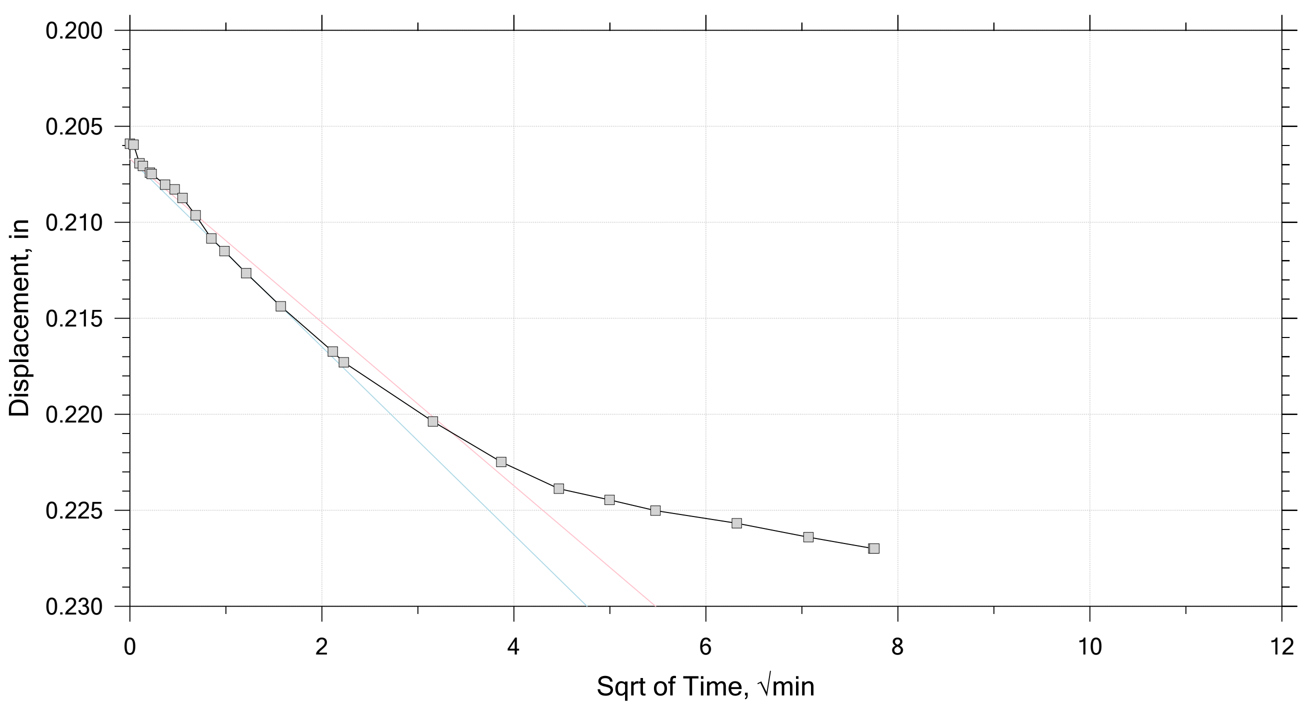
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 18 of 22

Constant Load Step

Stress: 1.73e+04 psf





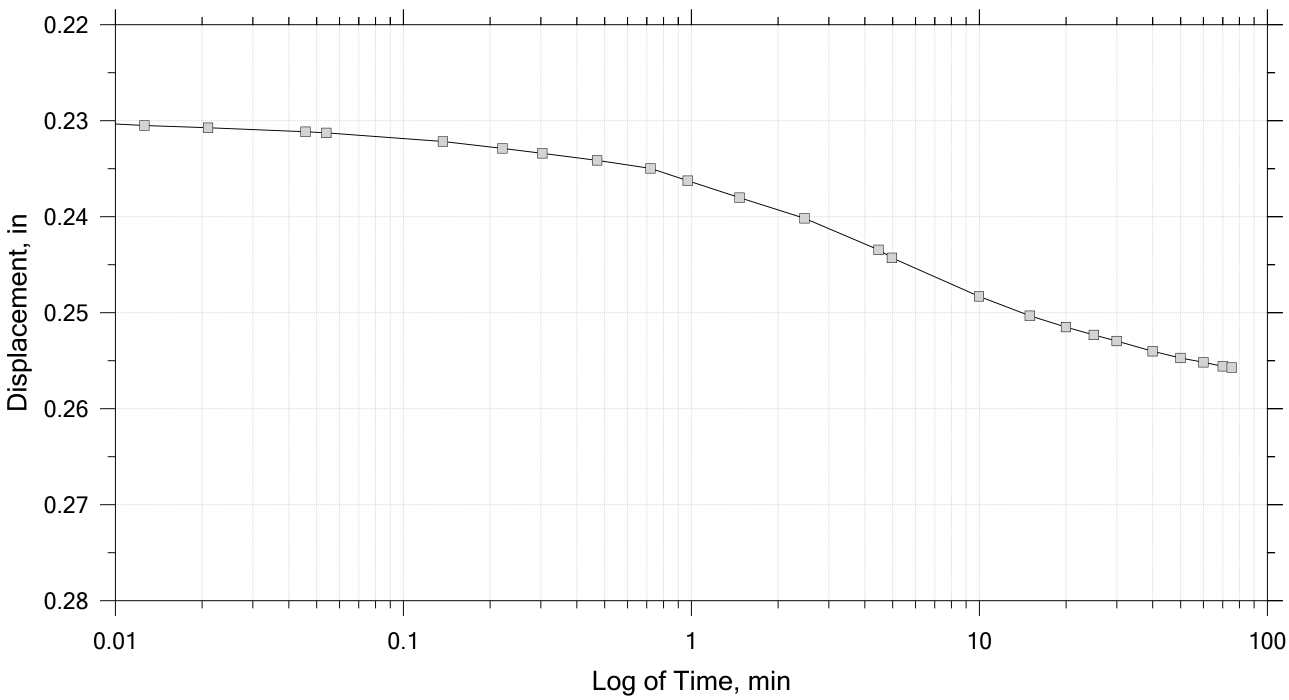
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

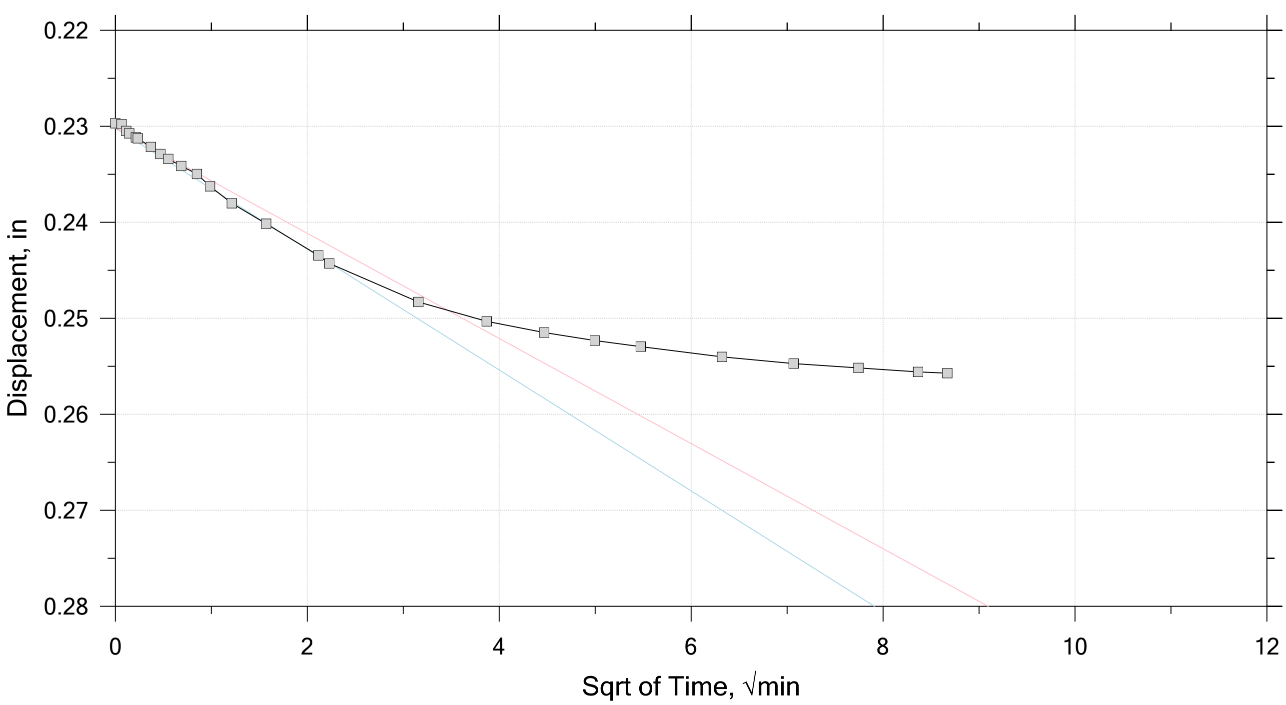
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 19 of 22

Constant Load Step

Stress: 2.6e+04 psf

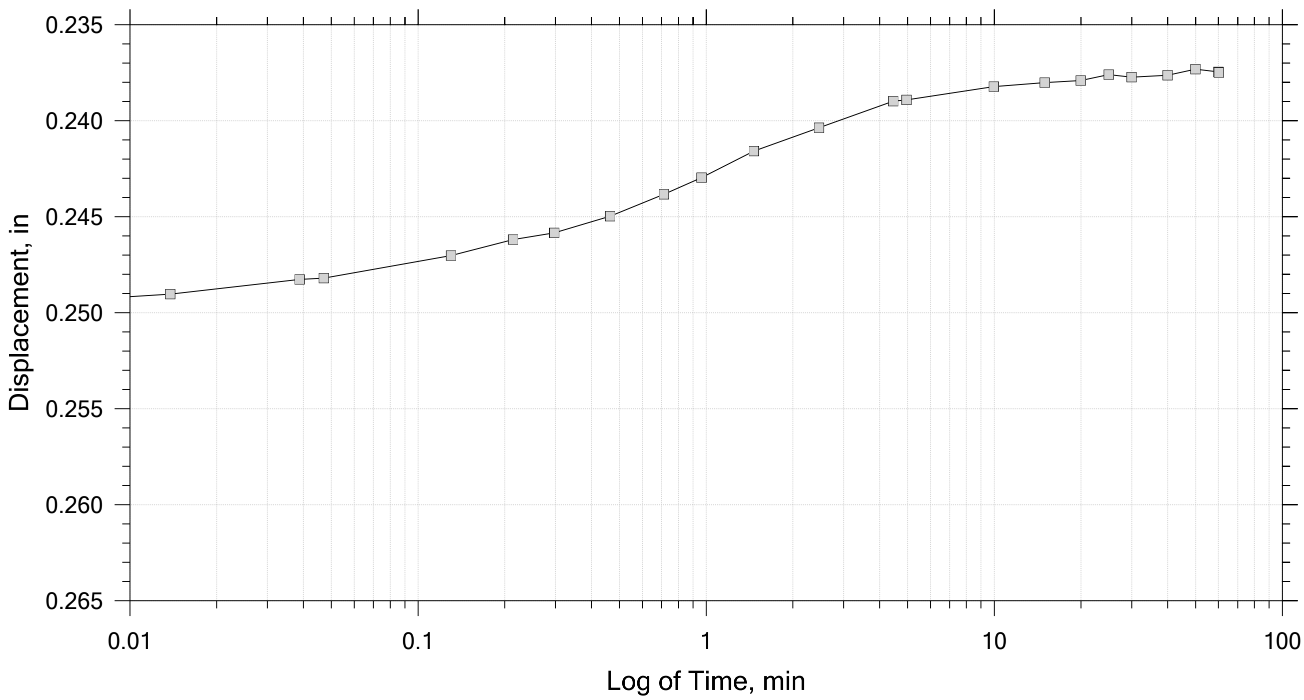


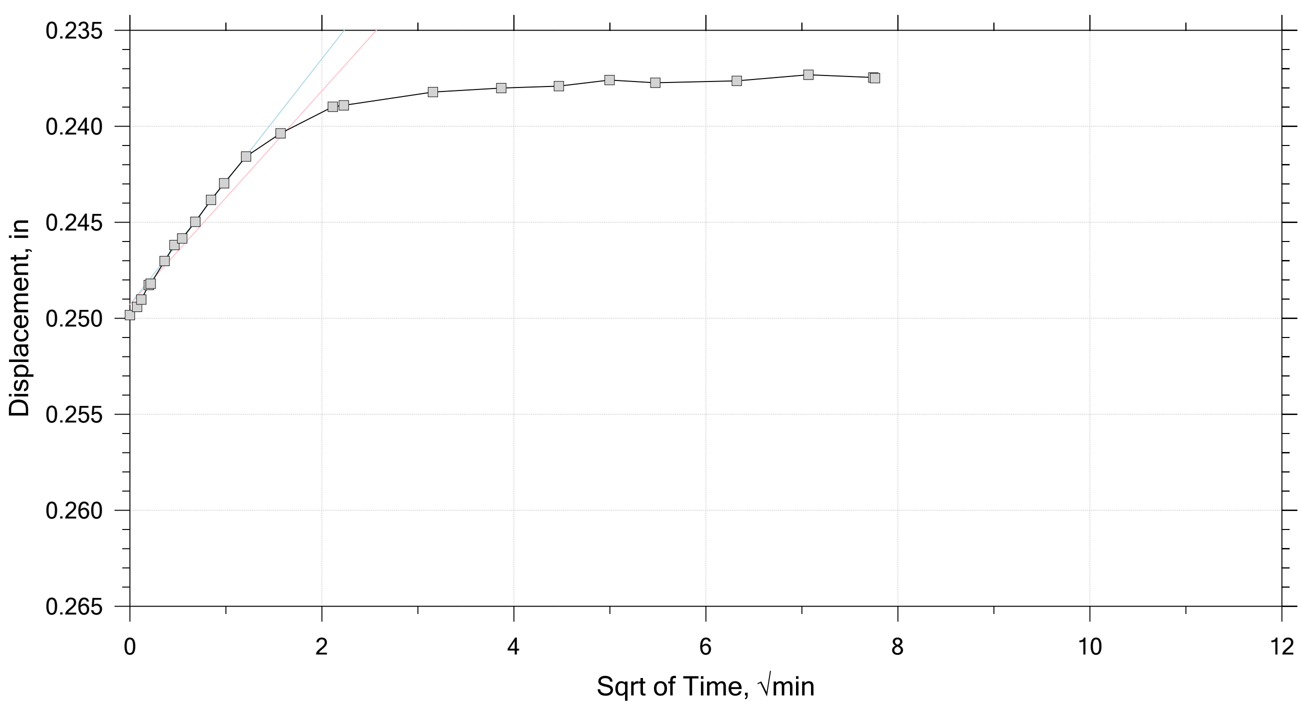


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 20 of 22
Constant Load Step
Stress: 5.13e+03 psf

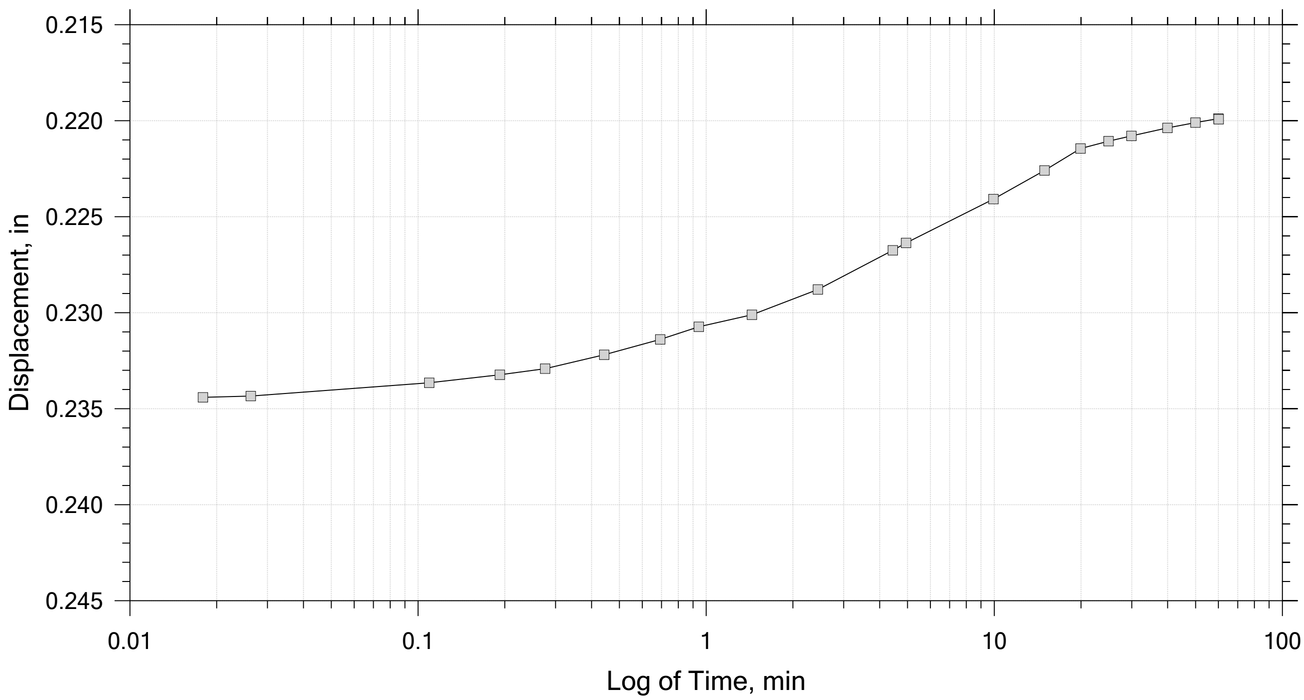


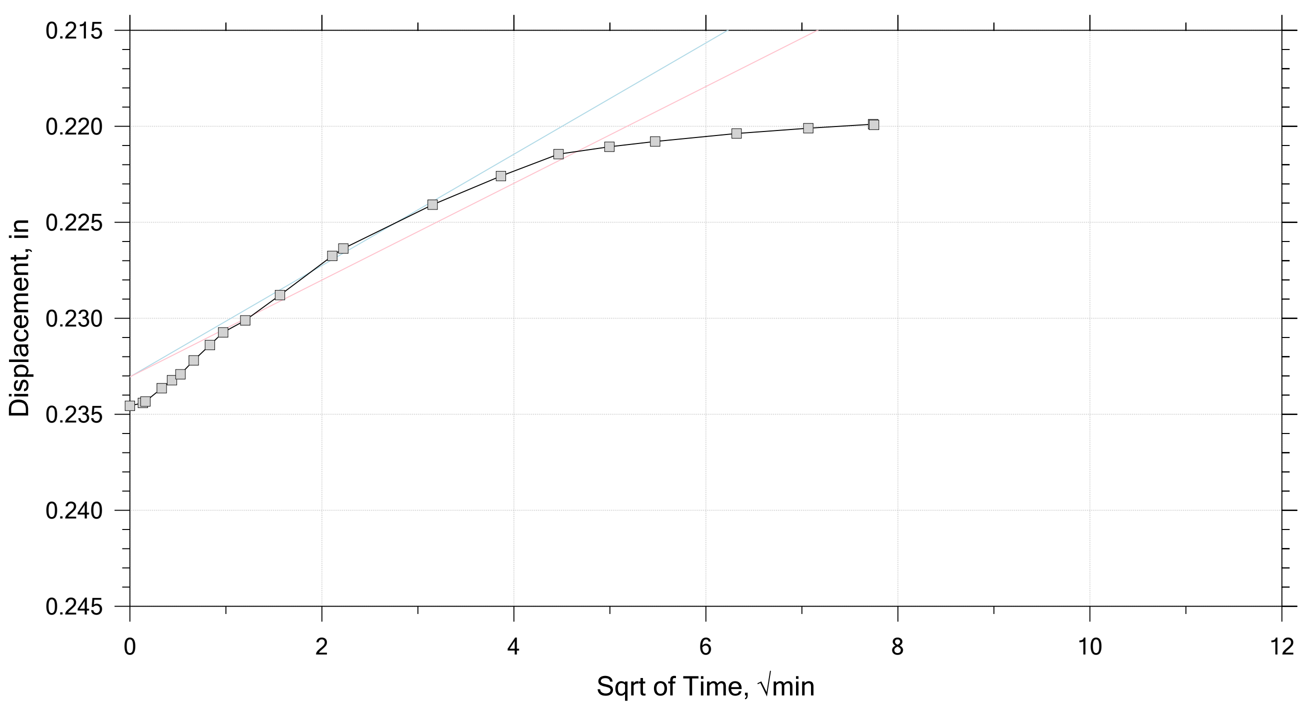


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 21 of 22
Constant Load Step
Stress: 1.52e+03 psf





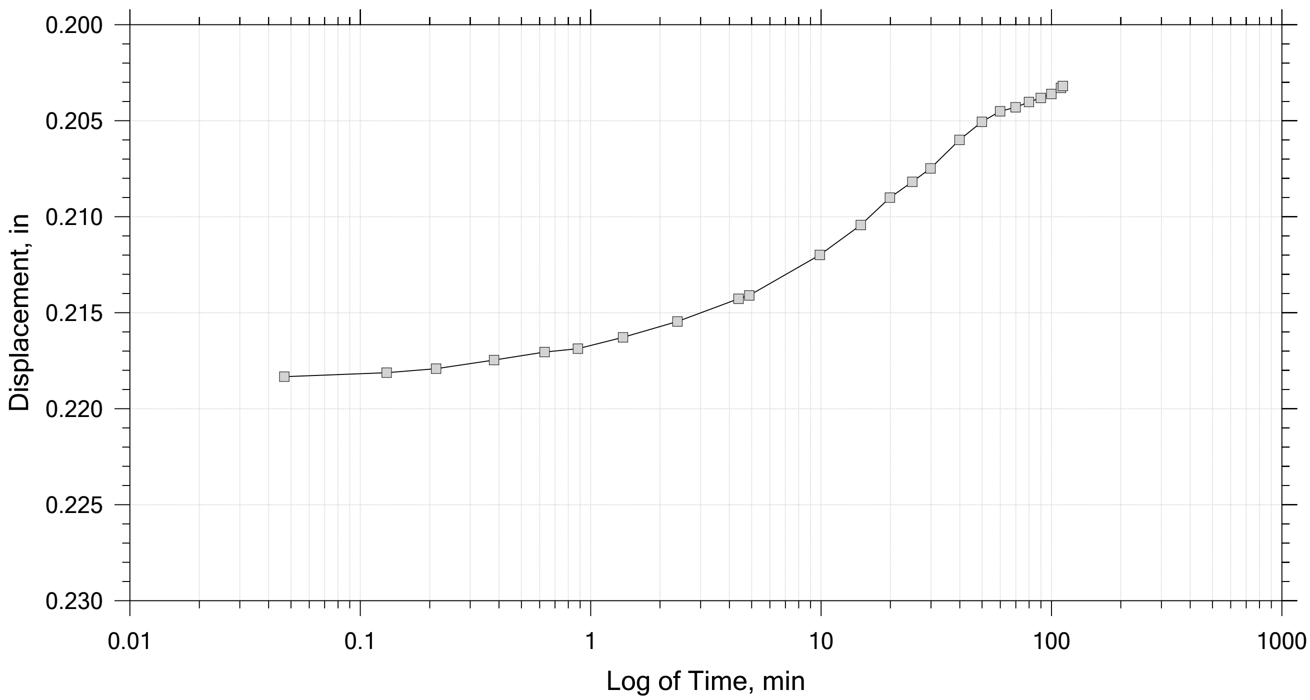
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

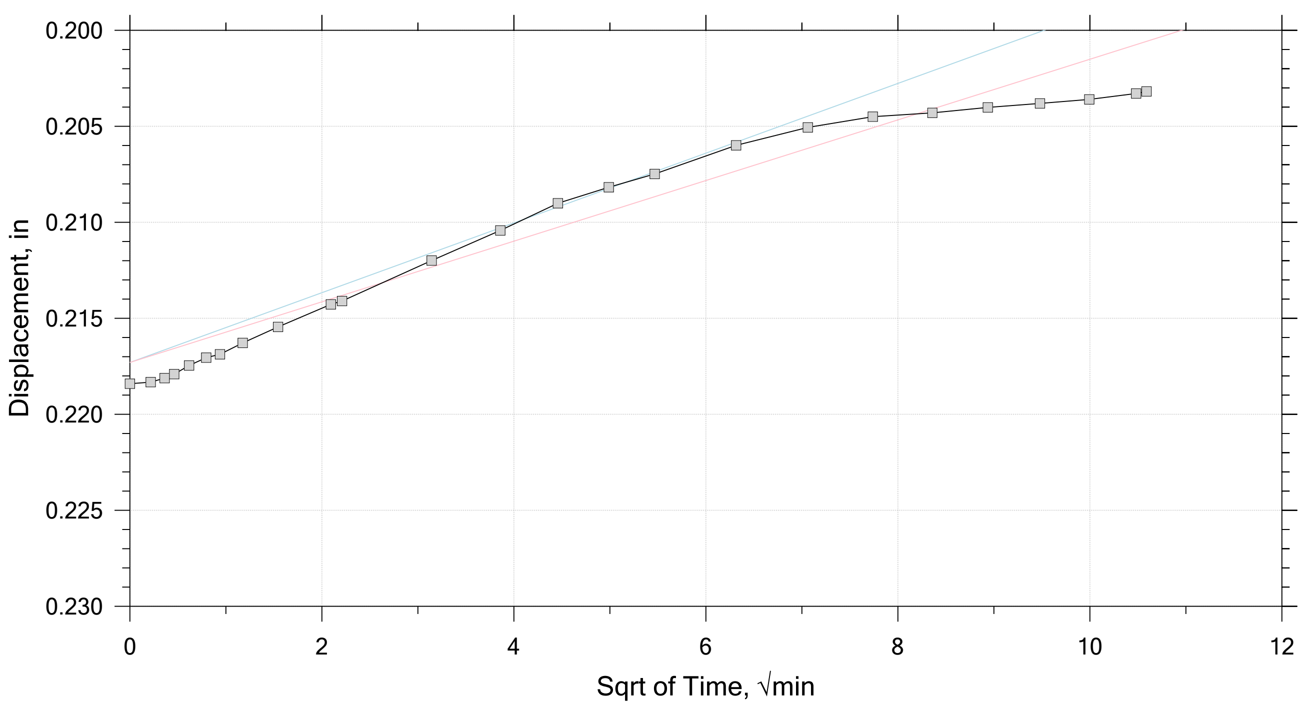
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 22 of 22

Constant Load Step

Stress: 450 psf





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
	Test Number: ICON305	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter, in: 2.50	Specific Gravity: 2.91 (Implied)	Liquid Limit: 42
Specimen Height, in: 1.00	Initial Void Ratio: 1.39	Plastic Limit: 23
Final Height, in: 0.80	Final Void Ratio: 0.903	Plasticity Index: 19

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	212	---	"ring"	305
Mass Container, gm	36.73	110.17	110.17	60.74
Mass Container + Wet Soil, gm	115.99	254.15	238.86	189.31
Mass Container + Dry Soil, gm	91.65	208.36	208.36	158.84
Mass Dry Soil, gm	54.92	98.192	98.192	98.1
Water Content, %	44.32	46.63	31.06	31.06
Void Ratio	---	1.39	0.90	---
Degree of Saturation, %	---	97.75	100.00	---
Dry Unit Weight, pcf	---	76.053	95.398	---

Preconsolidation Stress, psf	2100
Compression Ratio	0.24
Rebound Ratio	0.025
Compression Index	0.573
Rebound Index	0.05969

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.

Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
Test Number: ICON305	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients

Step	Applied Stress psf	EOP Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv ft²/day	Mv ft²/ton	k cm/s
1	200.	0.008559	1.37	0.854	47.164	4.47e-02	8.54e+04	4.21e-08
2	300.	0.01209	1.36	1.21	68.995	3.02e-02	7.05e+04	2.35e-08
3	450.	0.01646	1.35	1.64	106.645	1.94e-02	5.81e+04	1.24e-08
4	675.	0.02041	1.34	2.04	50.305	4.08e-02	3.50e+04	1.57e-08
5	1.01e+03	0.02623	1.33	2.62	27.044	7.51e-02	3.44e+04	2.84e-08
6	1.52e+03	0.03784	1.30	3.78	72.916	2.74e-02	4.58e+04	1.38e-08
7	2.28e+03	0.05395	1.26	5.38	45.944	4.22e-02	4.24e+04	1.97e-08
8	3.42e+03	0.09037	1.17	9.02	68.438	2.68e-02	6.38e+04	1.88e-08
9	5.13e+03	0.1324	1.07	13.2	56.795	2.96e-02	4.91e+04	1.60e-08
10	7.69e+03	0.1648	0.995	16.4	34.039	4.54e-02	2.52e+04	1.26e-08
11	1.16e+04	0.1940	0.925	19.4	22.431	6.40e-02	1.51e+04	1.06e-08
12	5.12e+03	0.1882	0.939	18.8	1.194	1.17e+00	1.81e+03	2.33e-08
13	2.28e+03	0.1804	0.958	18.0	13.041	1.09e-01	5.46e+03	6.54e-09
14	1.01e+03	0.1707	0.981	17.0	21.797	6.64e-02	1.54e+04	1.12e-08
15	2.28e+03	0.1739	0.973	17.4	8.421	1.73e-01	5.02e+03	9.59e-09
16	5.13e+03	0.1840	0.949	18.4	12.011	1.20e-01	7.14e+03	9.40e-09
17	1.15e+04	0.2021	0.906	20.2	7.845	1.77e-01	5.63e+03	1.10e-08
18	1.73e+04	0.2245	0.853	22.4	11.183	1.18e-01	7.74e+03	1.01e-08
19	2.60e+04	0.2526	0.786	25.2	11.995	1.03e-01	6.48e+03	7.35e-09
20	5.13e+03	0.2356	0.826	23.5	2.659	4.58e-01	1.62e+03	8.19e-09
21	1.52e+03	0.2201	0.863	22.0	21.717	5.85e-02	8.57e+03	5.52e-09
22	450.	0.2032	0.903	20.3	67.210	1.97e-02	3.17e+04	6.87e-09

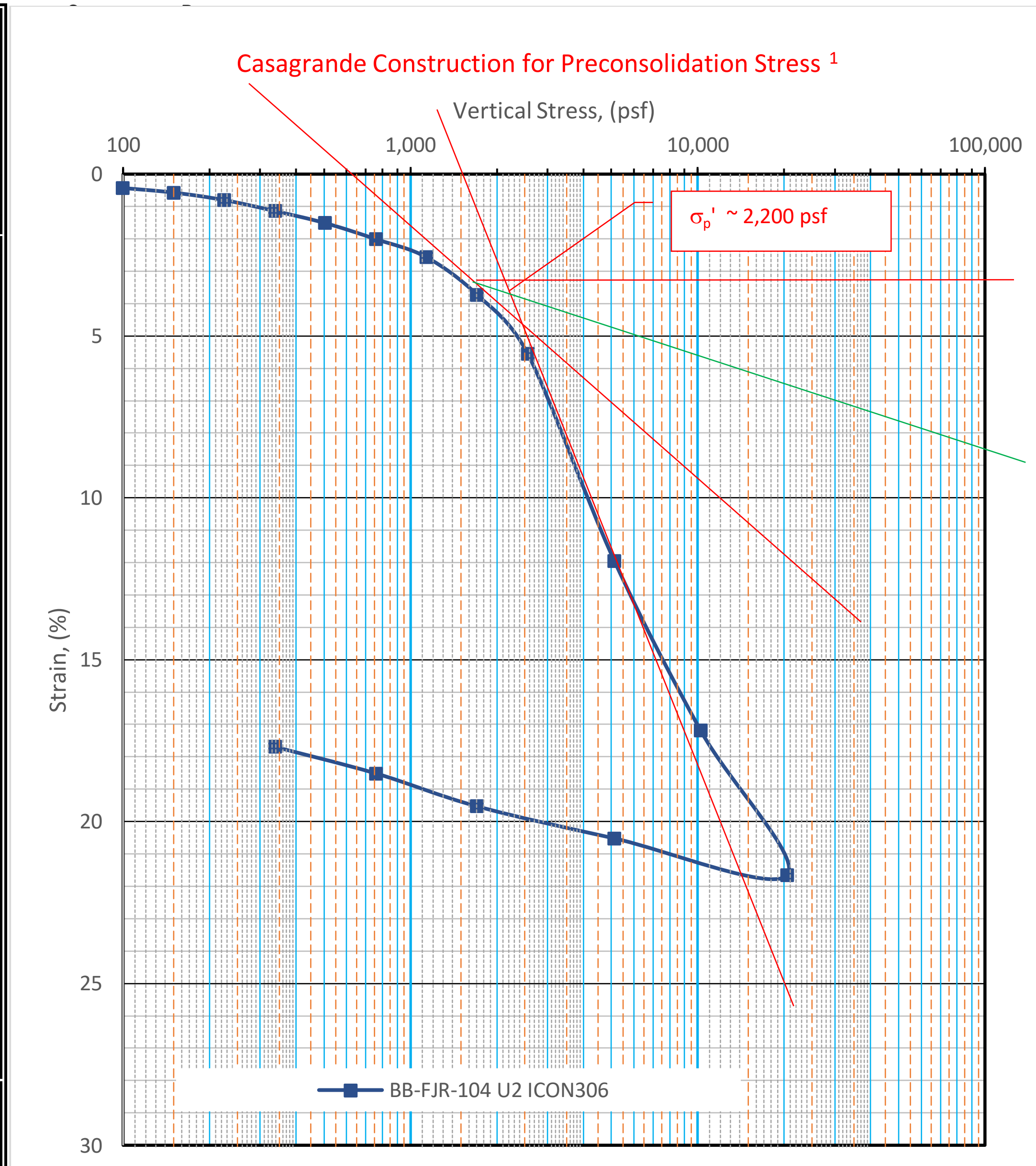
Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
Sample Number: U1	Test Date: 9/23/19	Depth: 31.05
Test Number: ICON305	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks:		
Displacement at End of Primary		

BB-FJR-104 2U

Consolidation Test Data

Project Name:	Johnson Road Bridge
Project Number:	166-11
Project Location:	Falmouth, Maine
Client:	GZA Proj. No. 09.0026024.00
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	ICON 306			
Boring No.	BB-FJR-104			
Sample No:	2U			
Boring Elevation (ft).	36.9			
Sample Depth (ft):	40 - 42			
Test Specimen Depth (Ft):	41.38			
Test Specimen Elevation:	-4.48			
Water Content (%):	41.57			
Dry Unit Weight (pcf):	80.69			
Wet Unit Weight (pcf):	114.23			
Saturation Before (%):	97.07			
Saturation After (%):	100			
Void Ratio Before:	1.23			
Void Ratio After:	0.84			
Overburden Pressure (psf):				
Max Previous stress (psf):	2,200			
Max Prev. stress (Work) (psf):	2,250			
OCR:				
Compression Index (C_{CE}):	0.22			
Recompression Index (C_{RE}):	0.024			
Liquid Limit:	40			
Plastic Limit:	23			
Plasticity Index:	17			
Liquidity Index:	1.1			
Specific Gravity (implied)	2.89			
Lab Vane Su at __ ft. (psf)				
Tested By:	sjr			
Date Tested:	9/24/2019			
Checked By:	sjr			

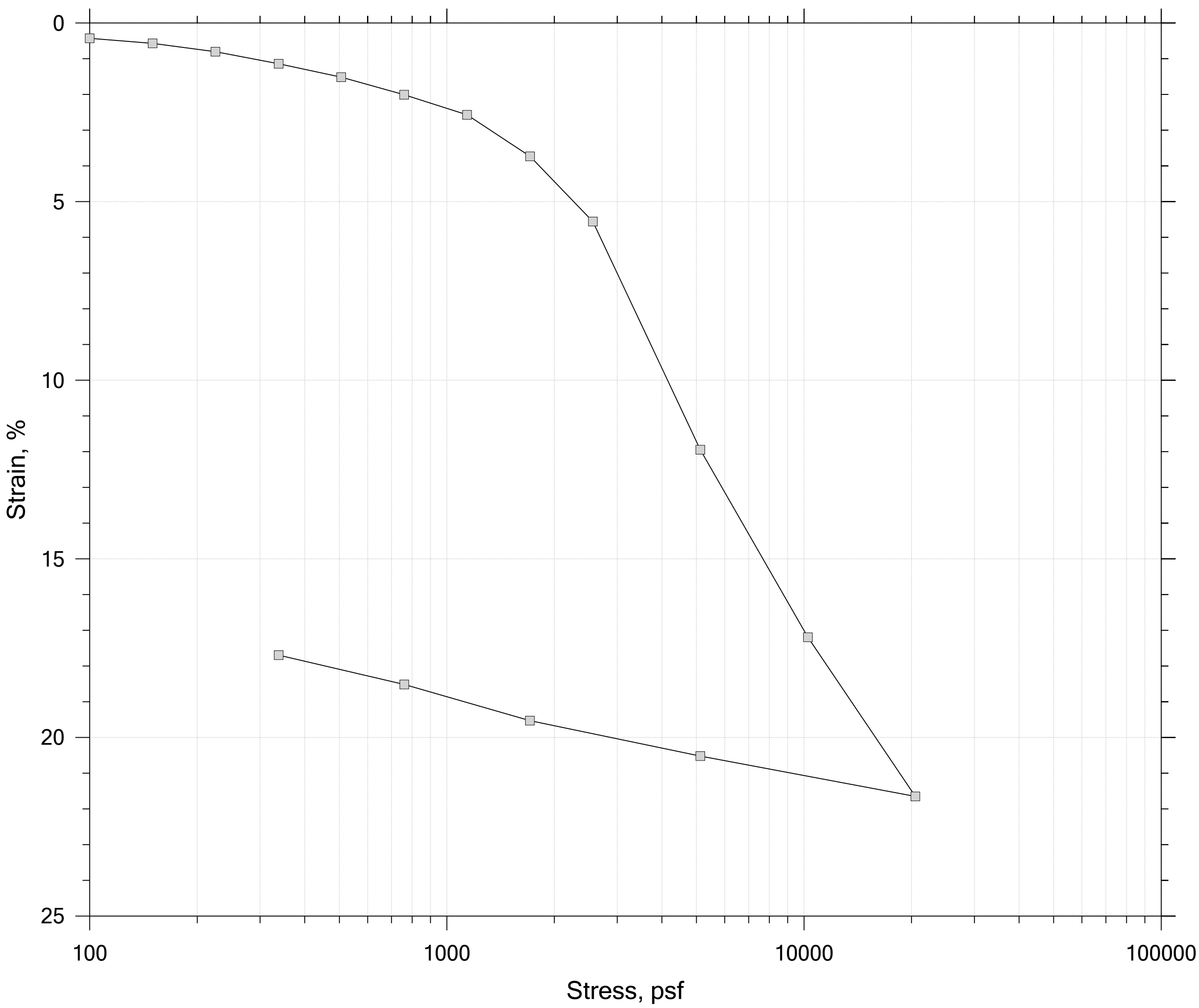


Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convienience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.

Soil Metrics LLC Geotechnical Engineering

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

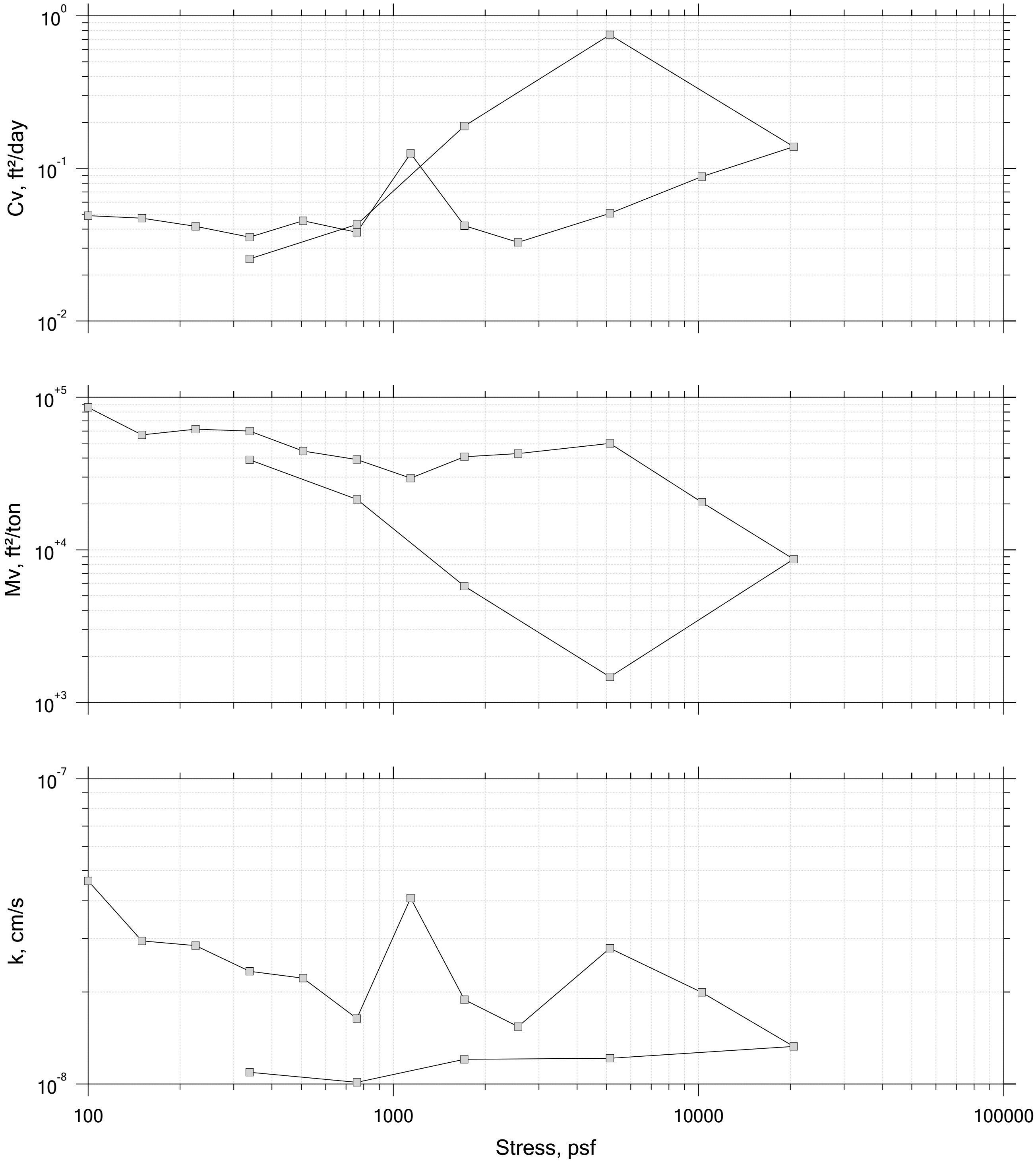


					Before Test	After Test
Current Vertical Effective Stress, psf: 0				Water Content, %	41.47	29.03
Preconsolidation Stress, psf: 2200				Dry Unit Weight, pcf	80.689	98.036
Compression Ratio: 0.22				Saturation, %	97.07	100.00
Specimen Diameter, in: 2.5		Specimen Height, in: 1.002		Void Ratio	1.23	0.84
LL: 0	PL: 0	PI: 0	GS: 2.89			

	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		
	Displacement at End of Primary		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients



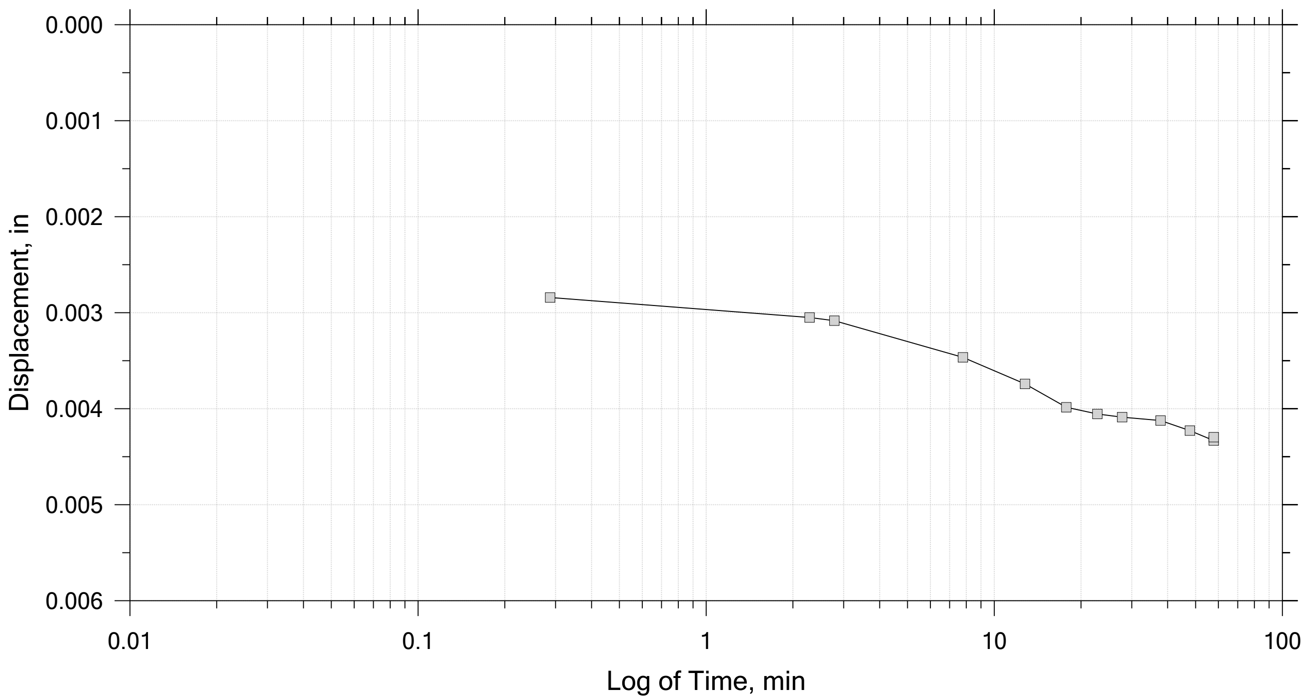
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

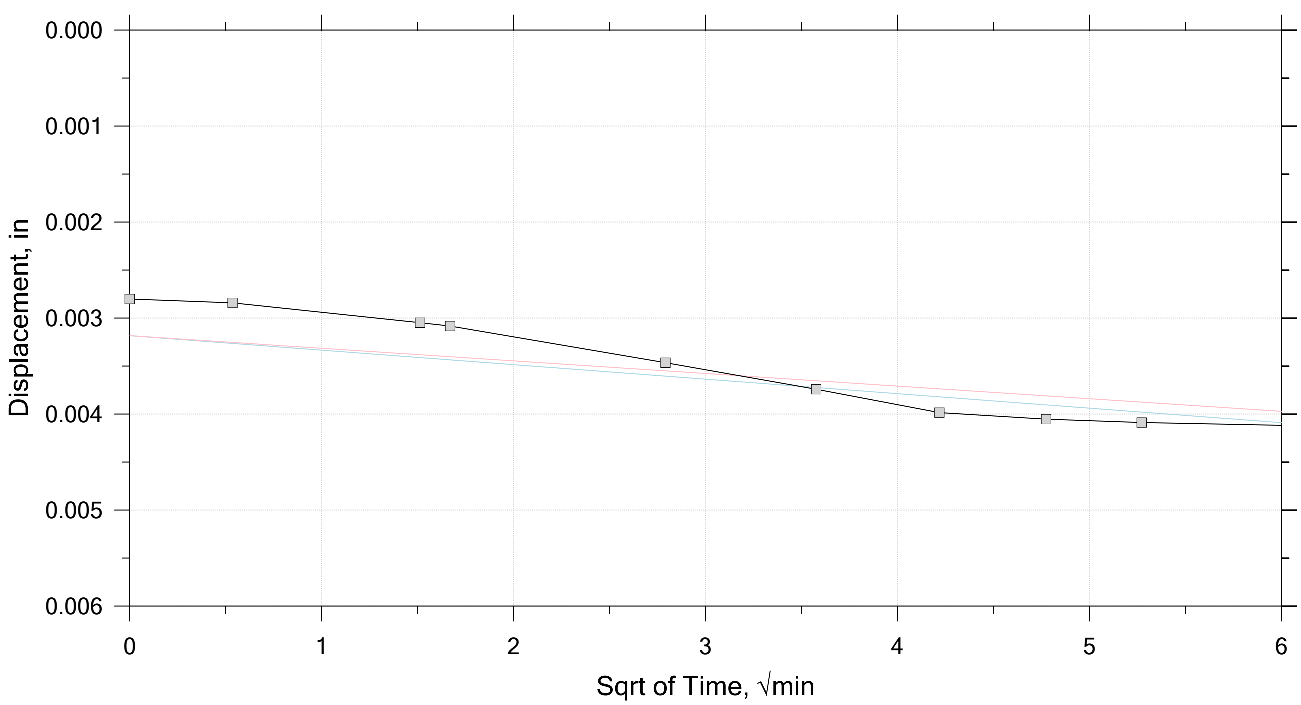
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 16

Constant Load Step

Stress: 100 psf





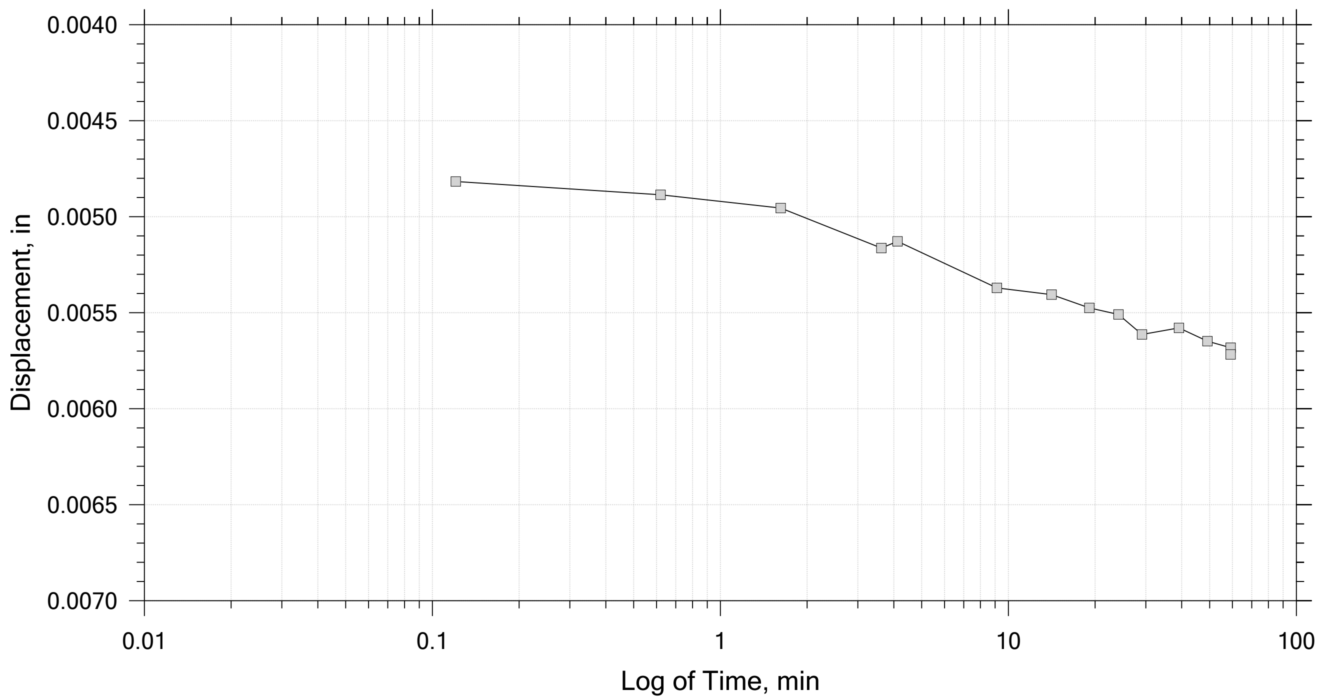
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

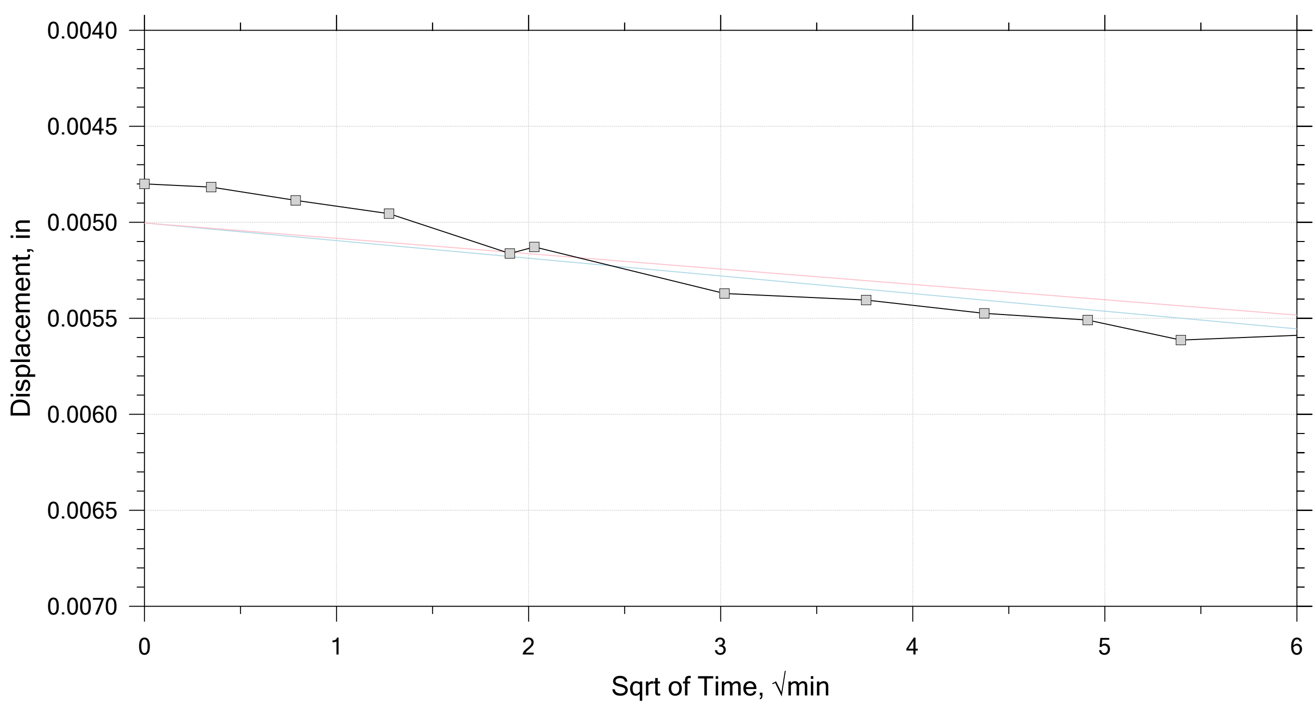
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 16

Constant Load Step

Stress: 150 psf





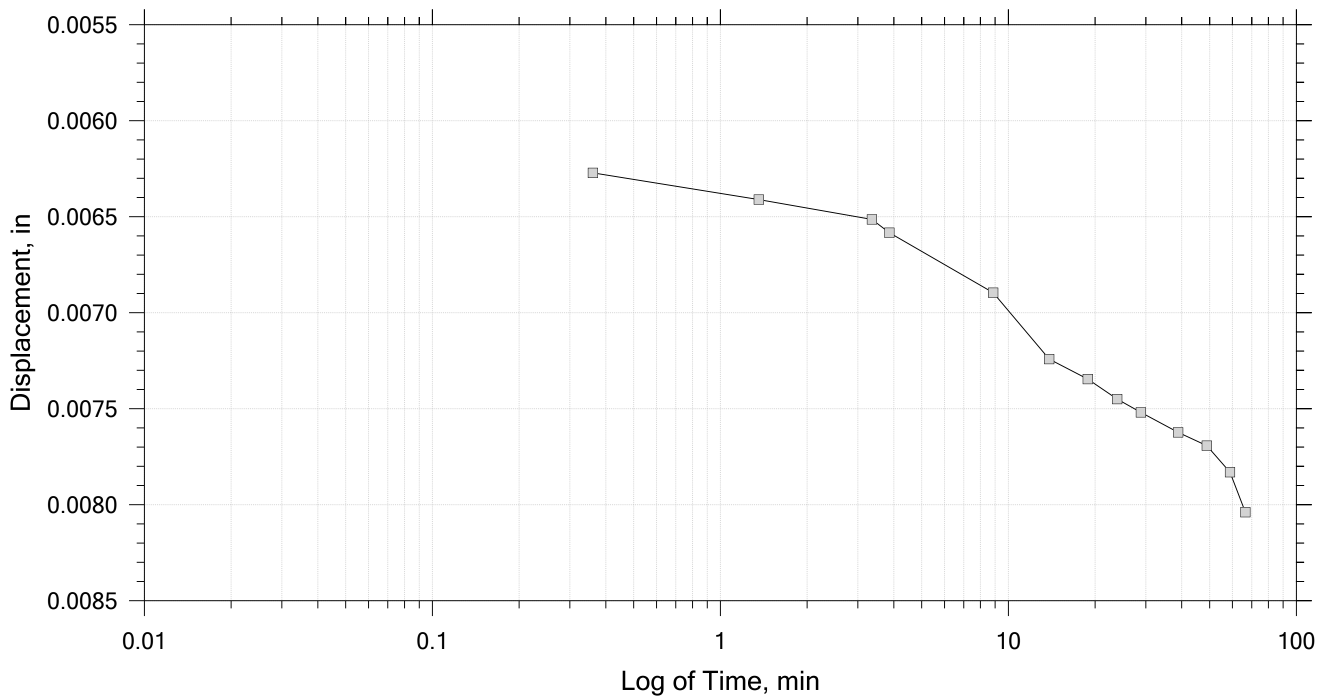
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

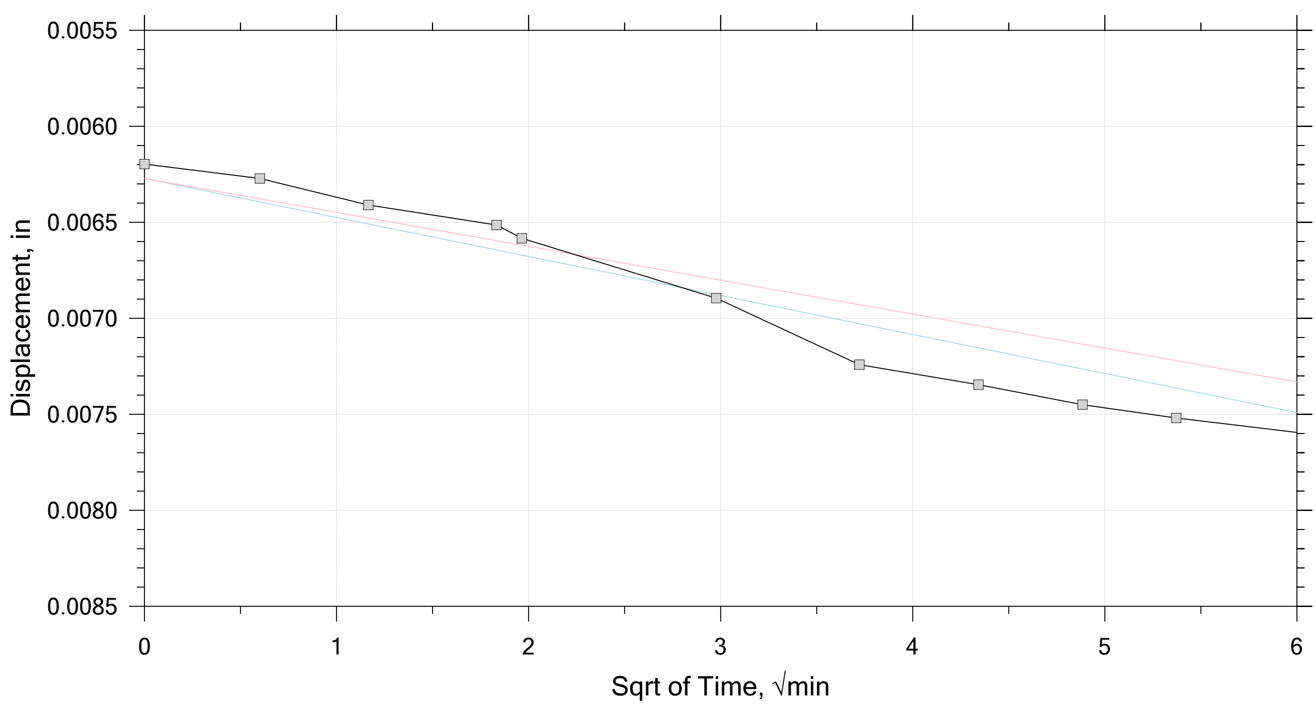
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 16

Constant Load Step

Stress: 225 psf





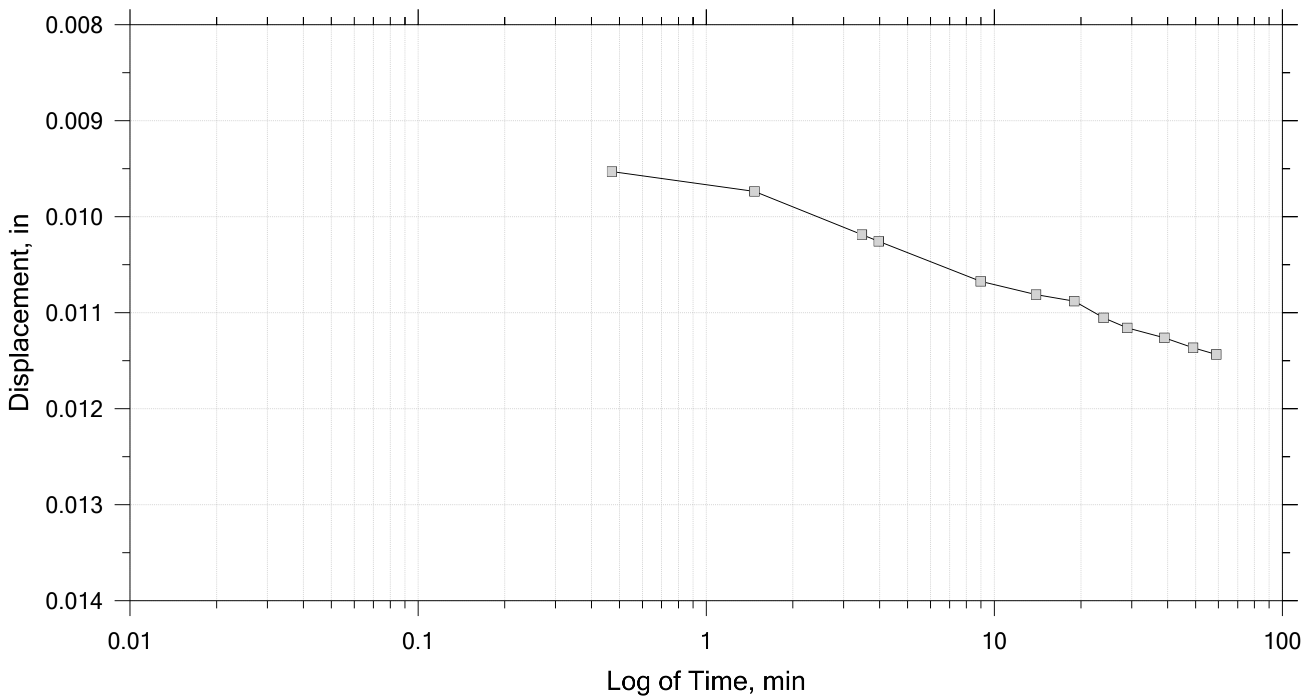
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

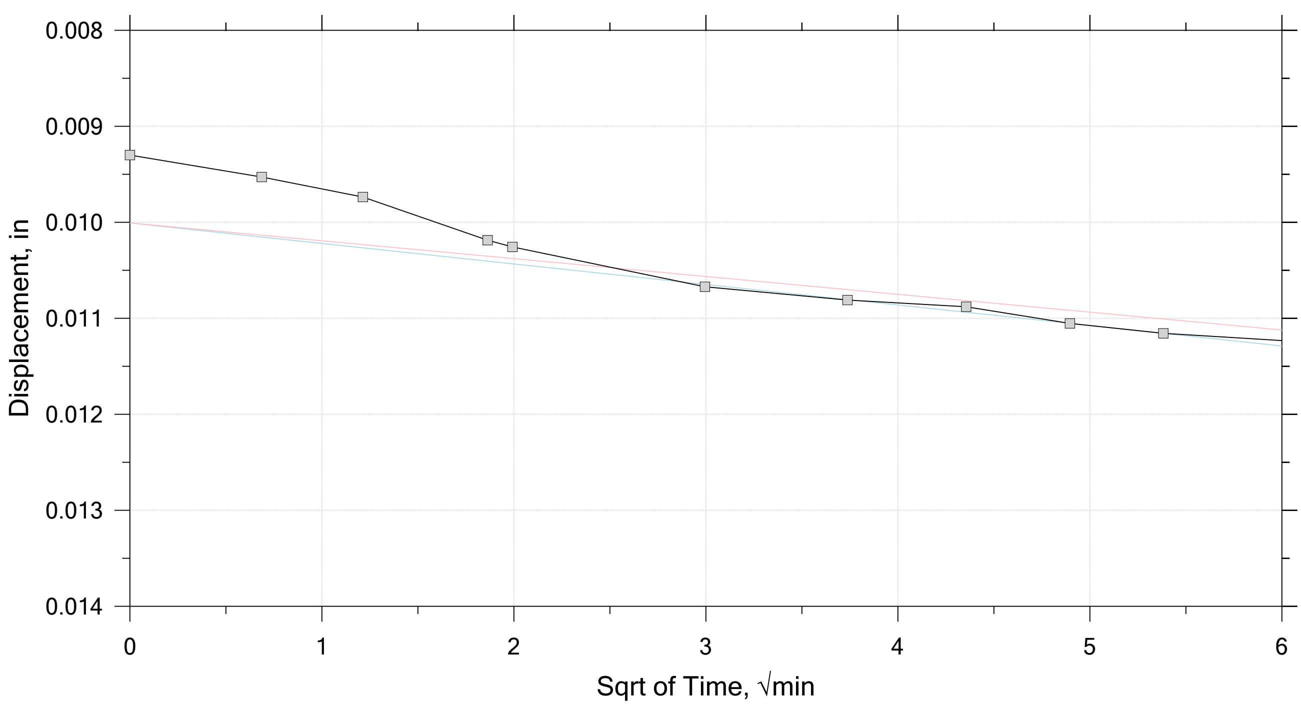
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 16

Constant Load Step

Stress: 338 psf

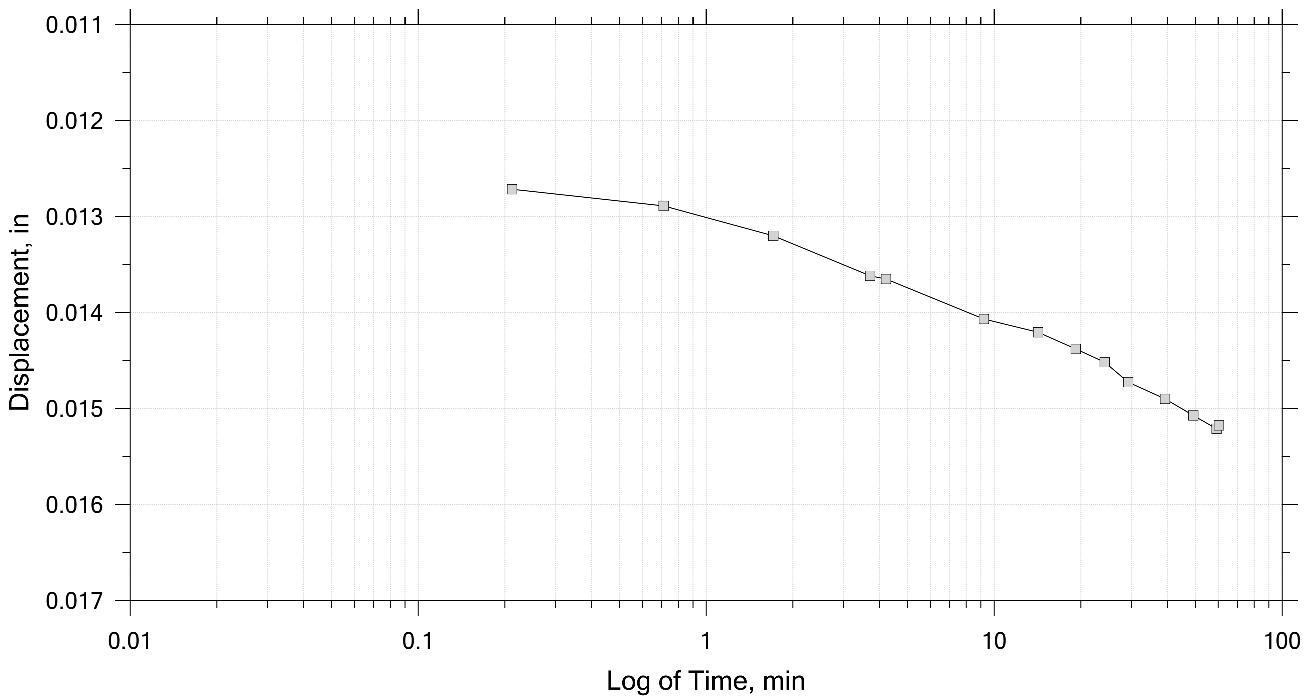


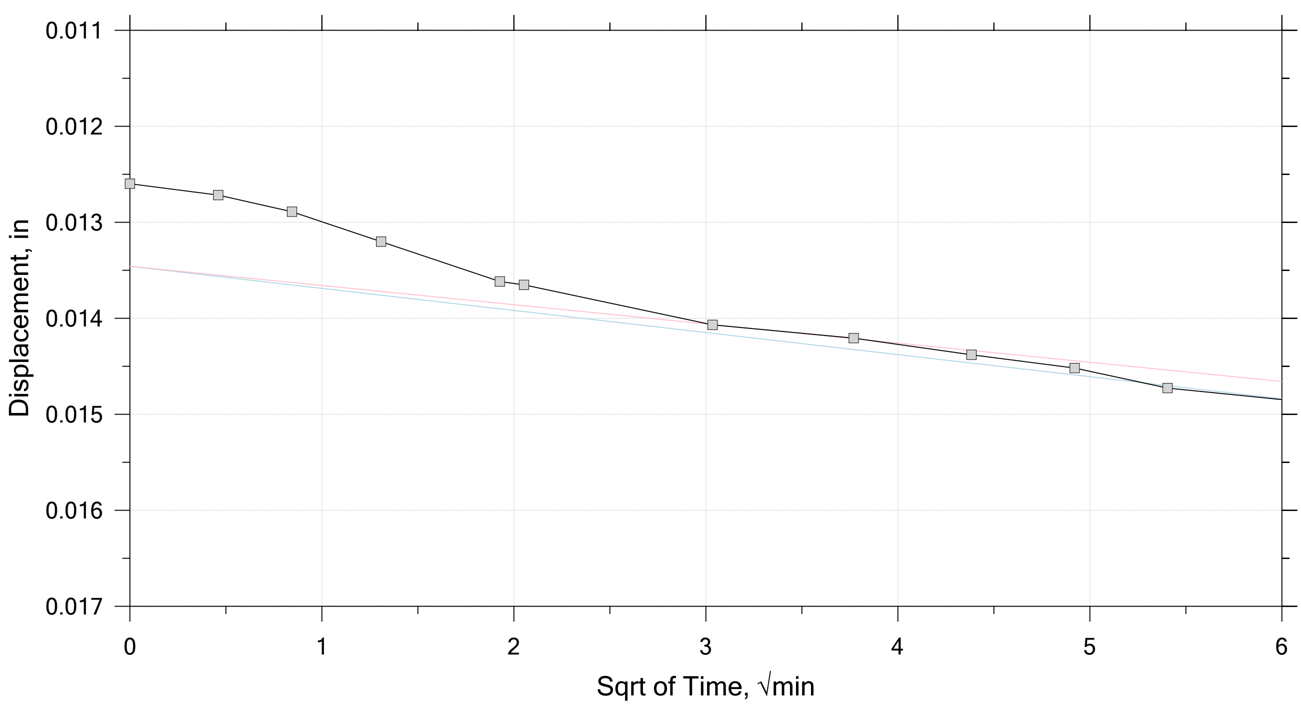


Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
Test Number: ICON306	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 16
Constant Load Step
Stress: 506 psf





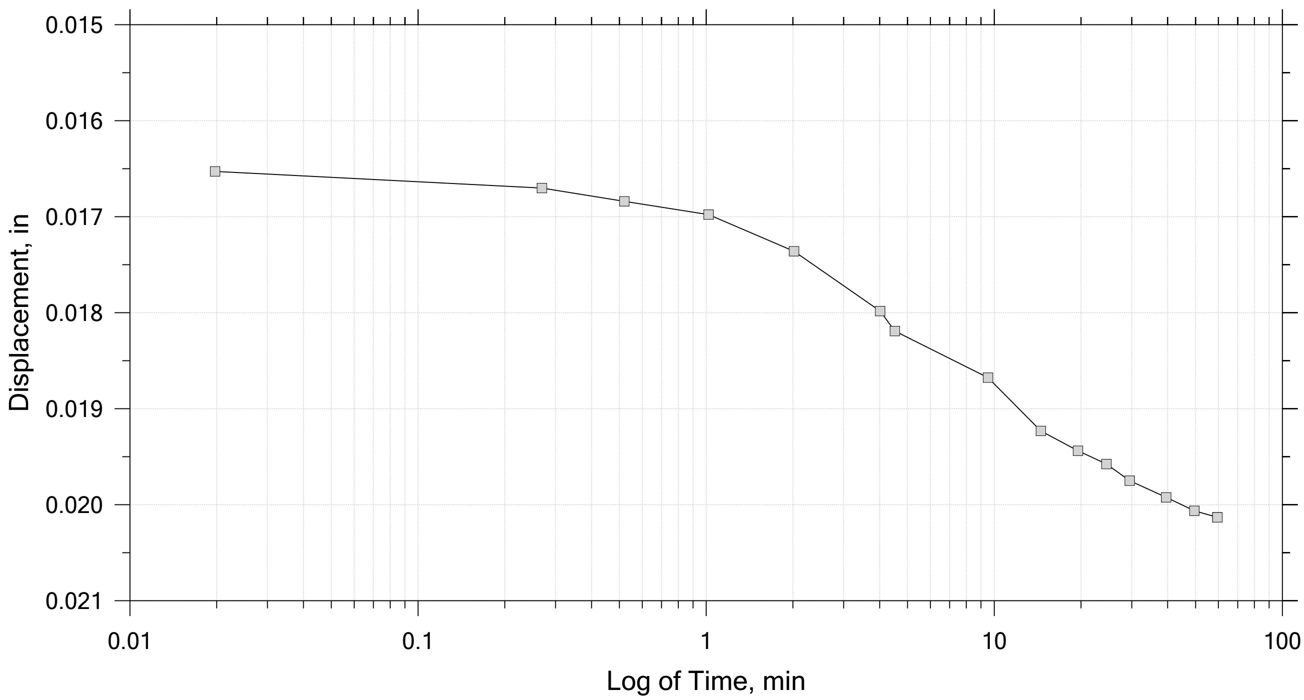
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

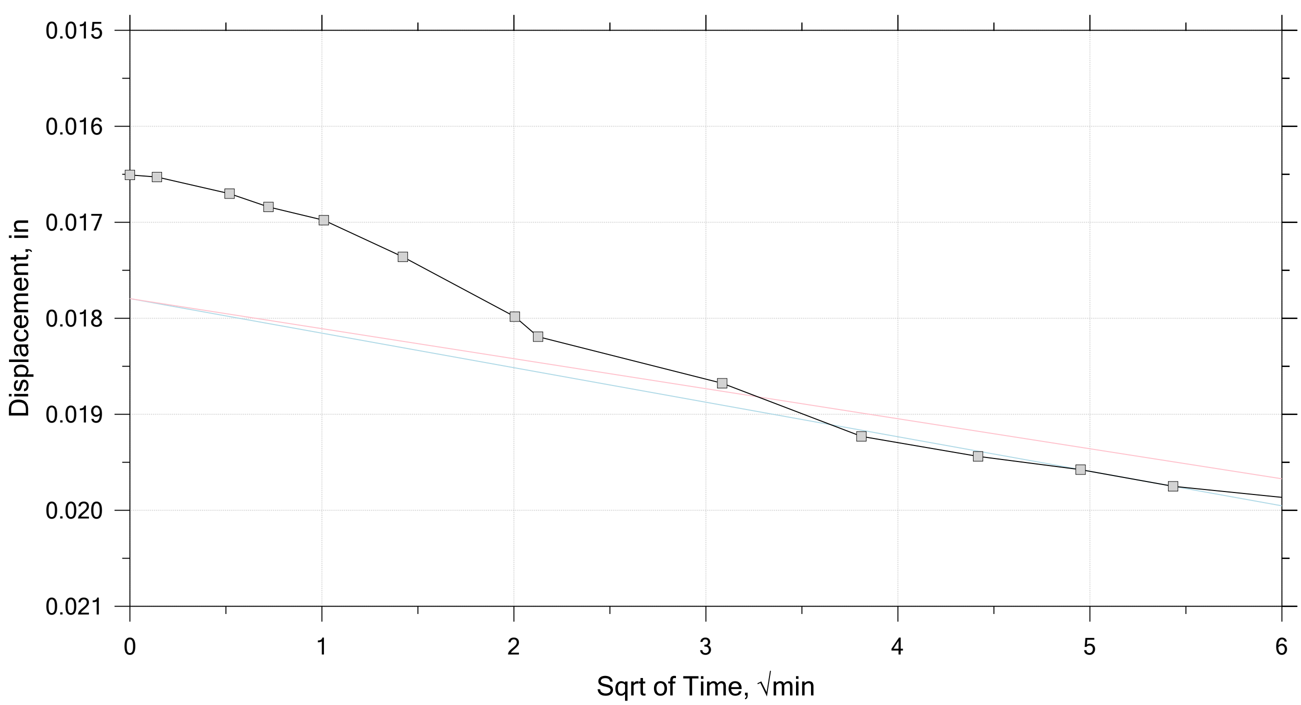
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 16

Constant Load Step

Stress: 759 psf

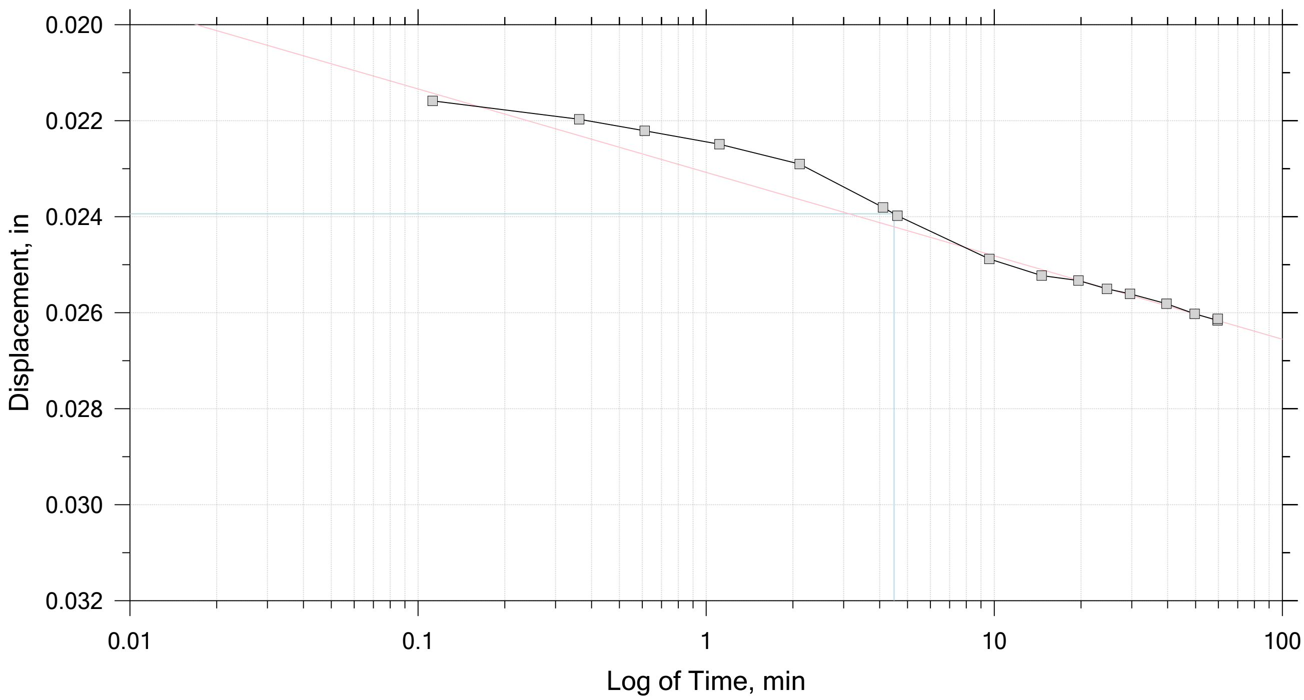


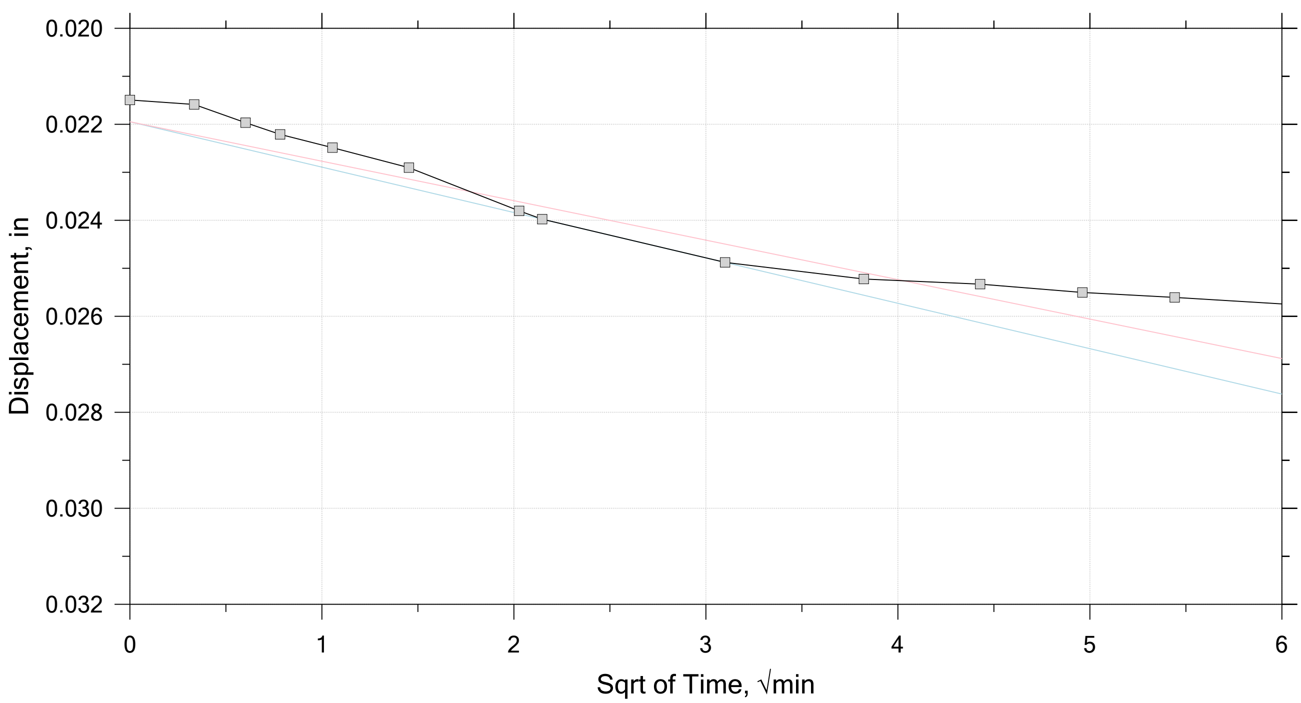


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 16
Constant Load Step
Stress: 1.14e+03 psf





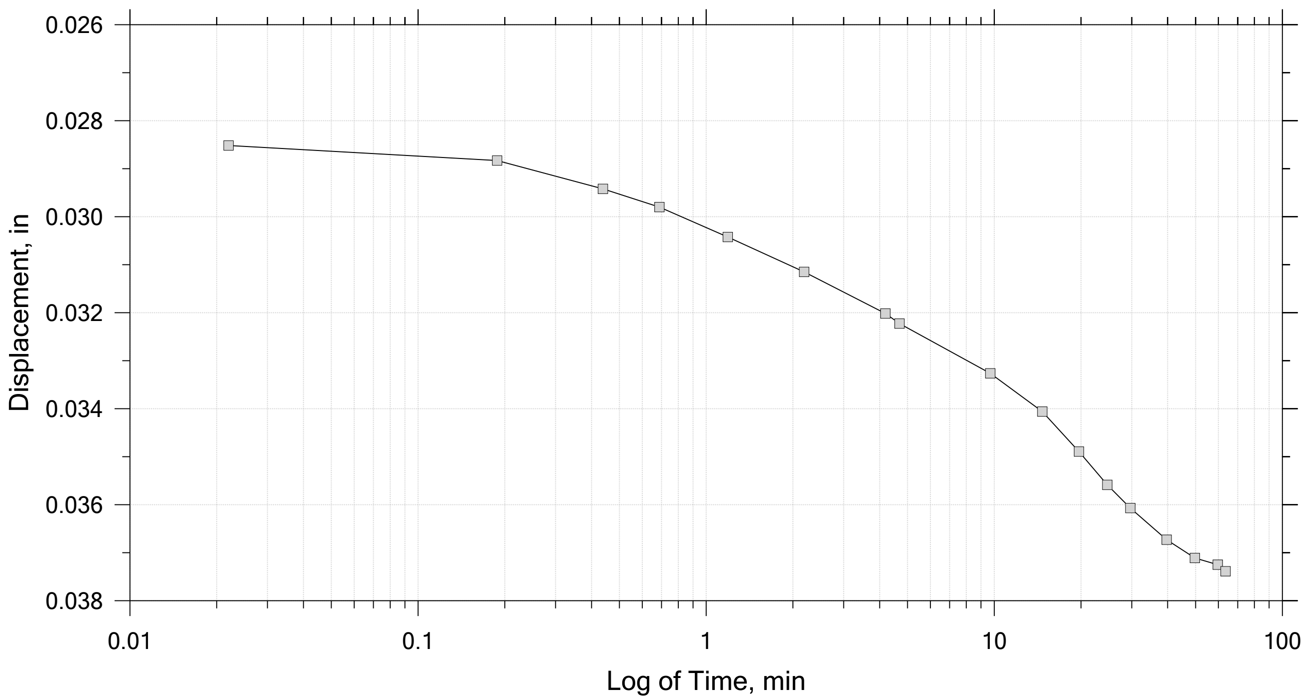
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

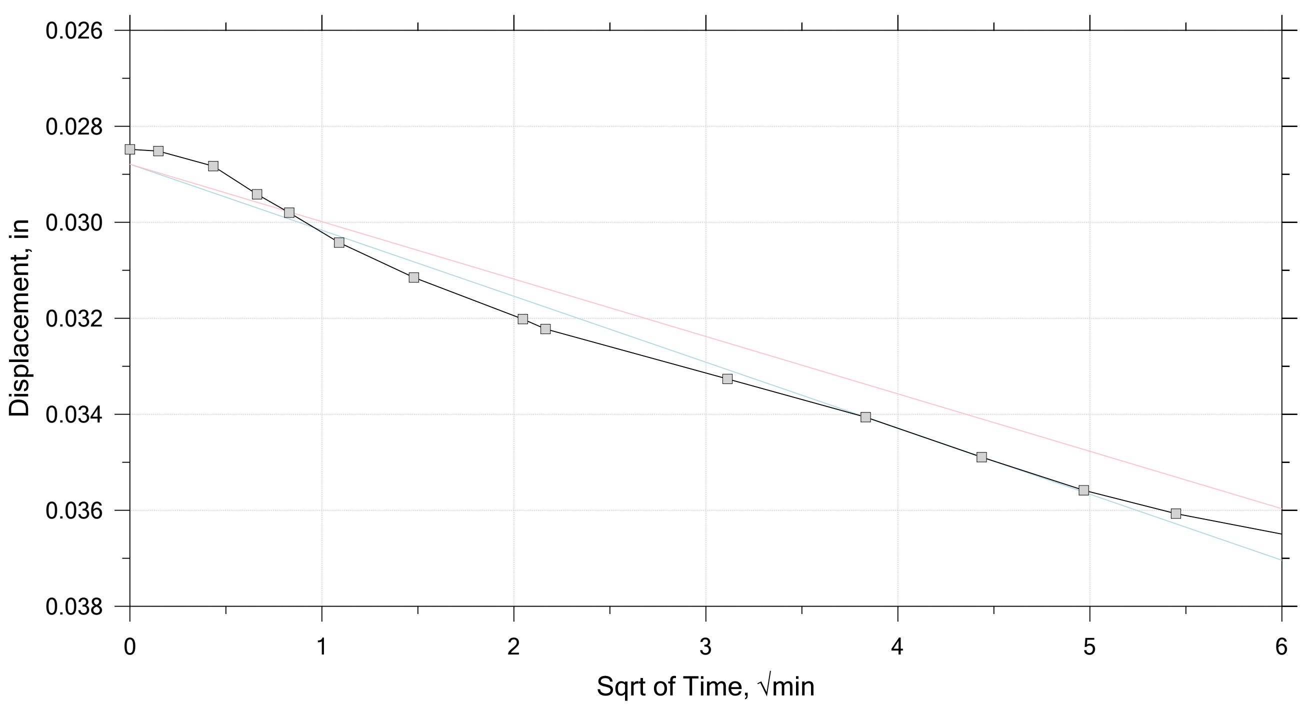
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 16

Constant Load Step

Stress: 1.71e+03 psf





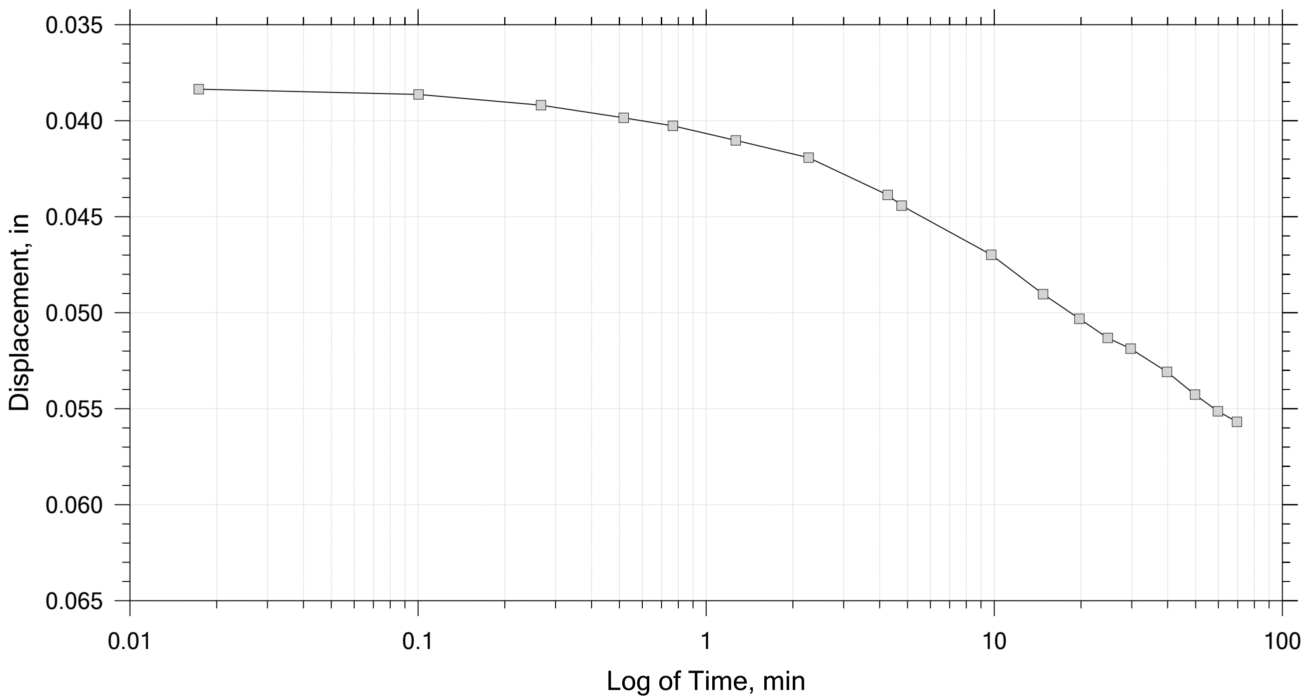
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

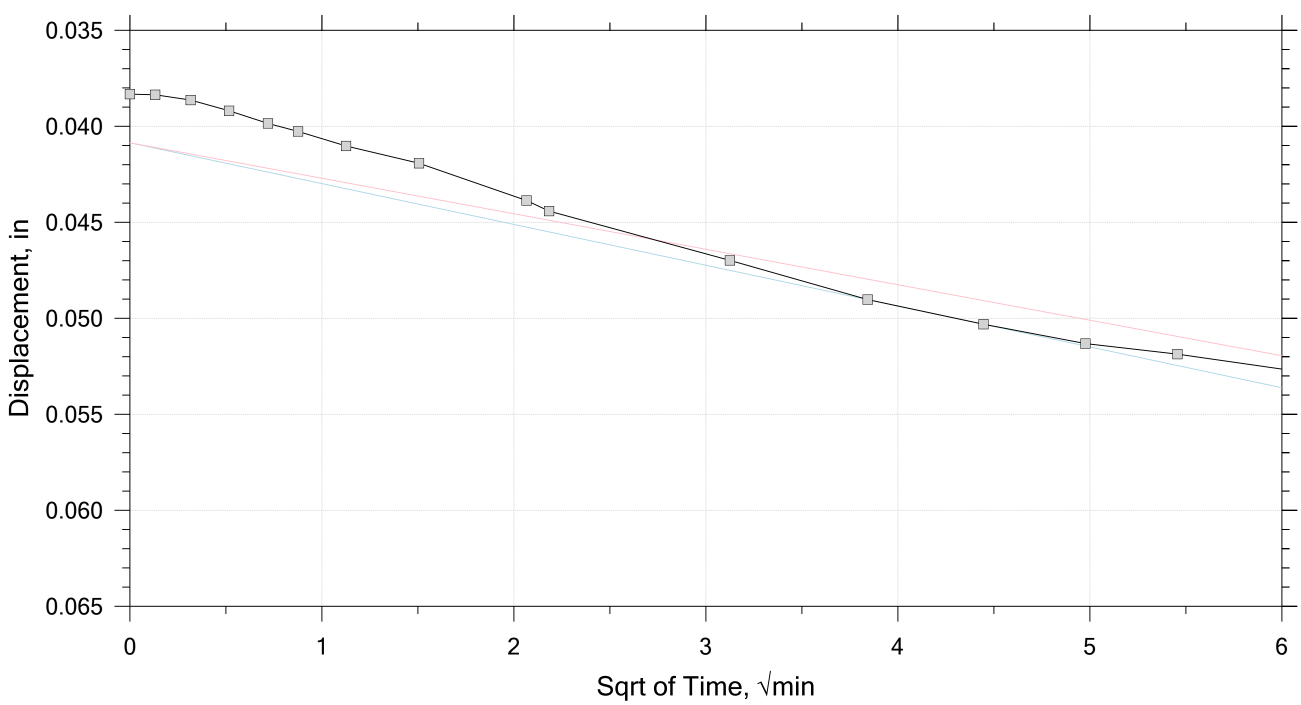
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 16

Constant Load Step

Stress: 2.56e+03 psf





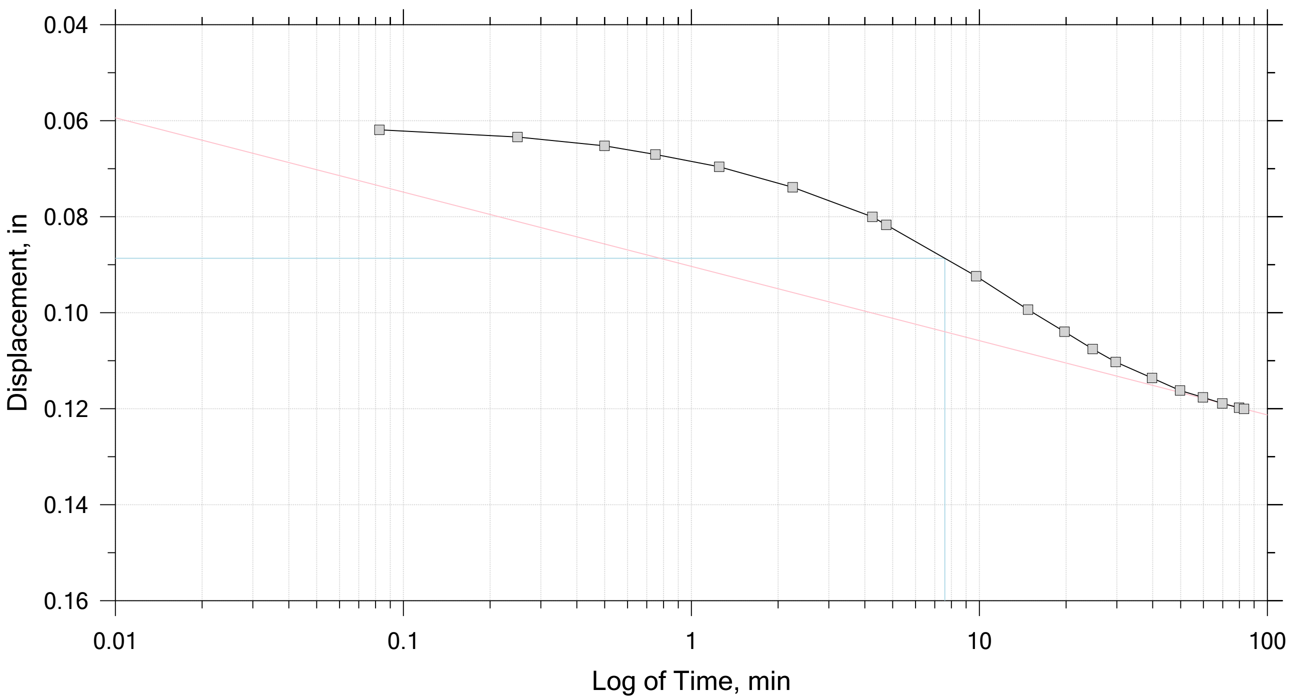
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

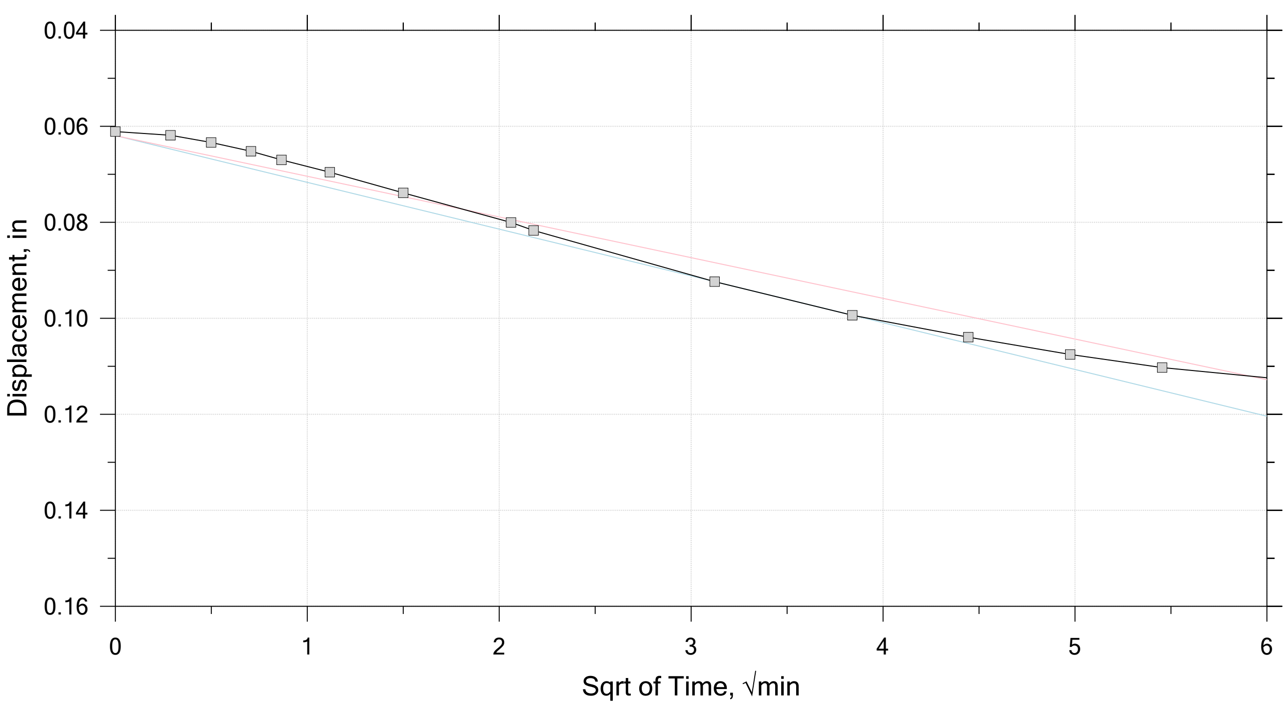
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 16

Constant Load Step

Stress: 5.13e+03 psf

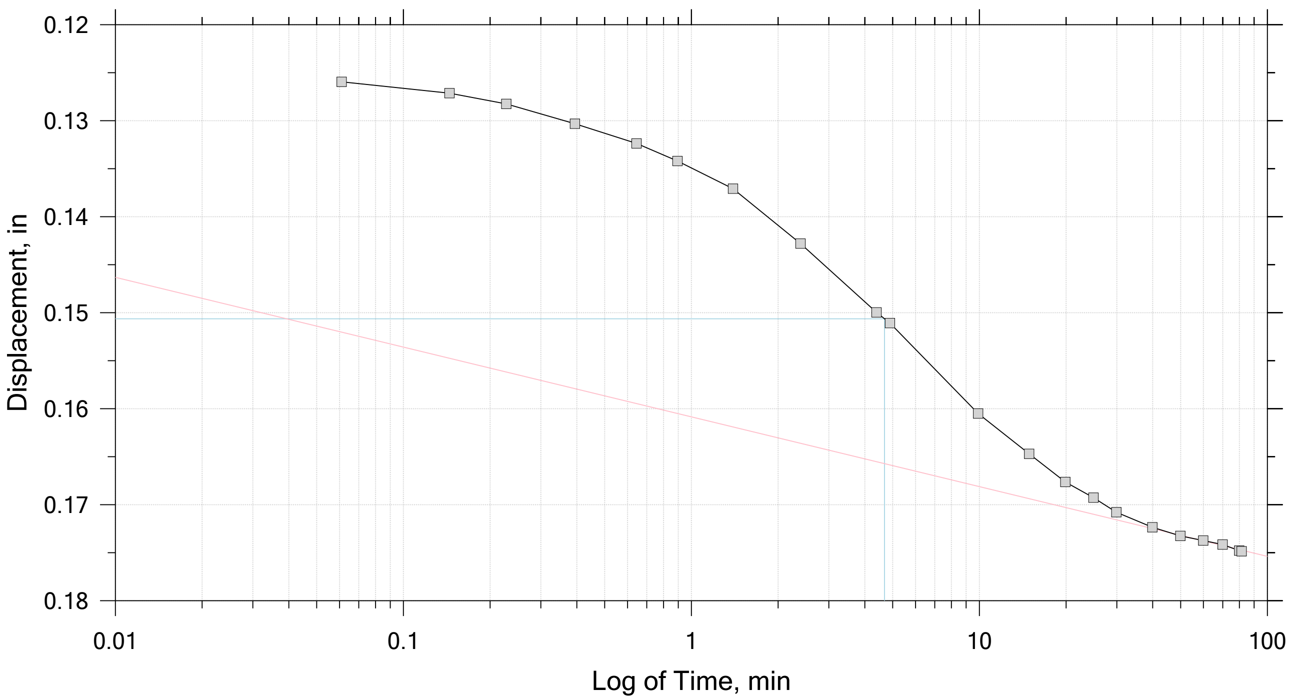


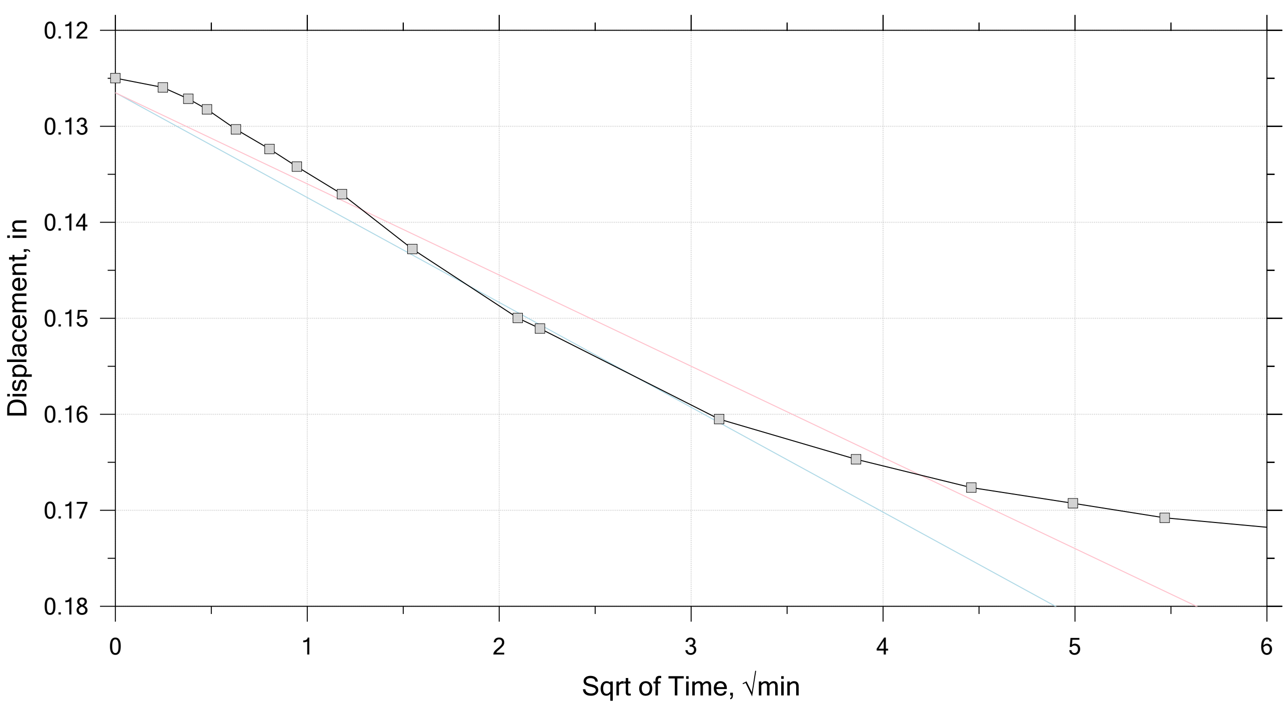


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 16
Constant Load Step
Stress: 1.03e+04 psf





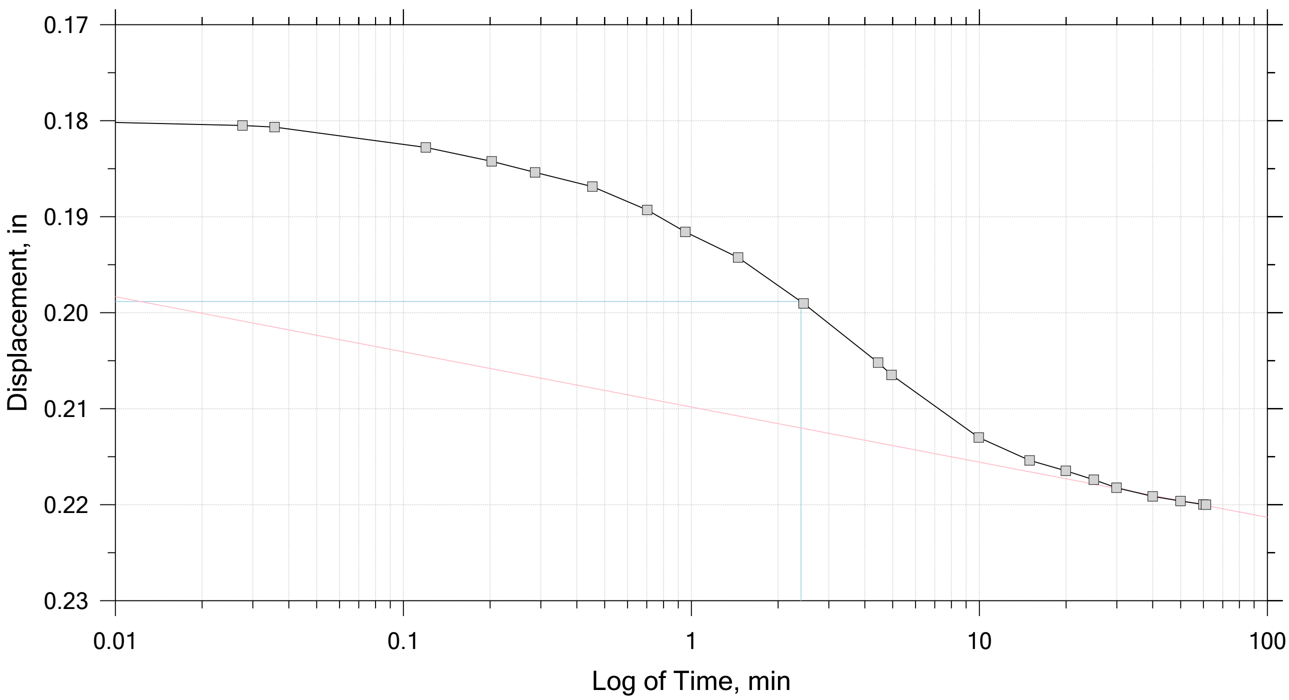
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

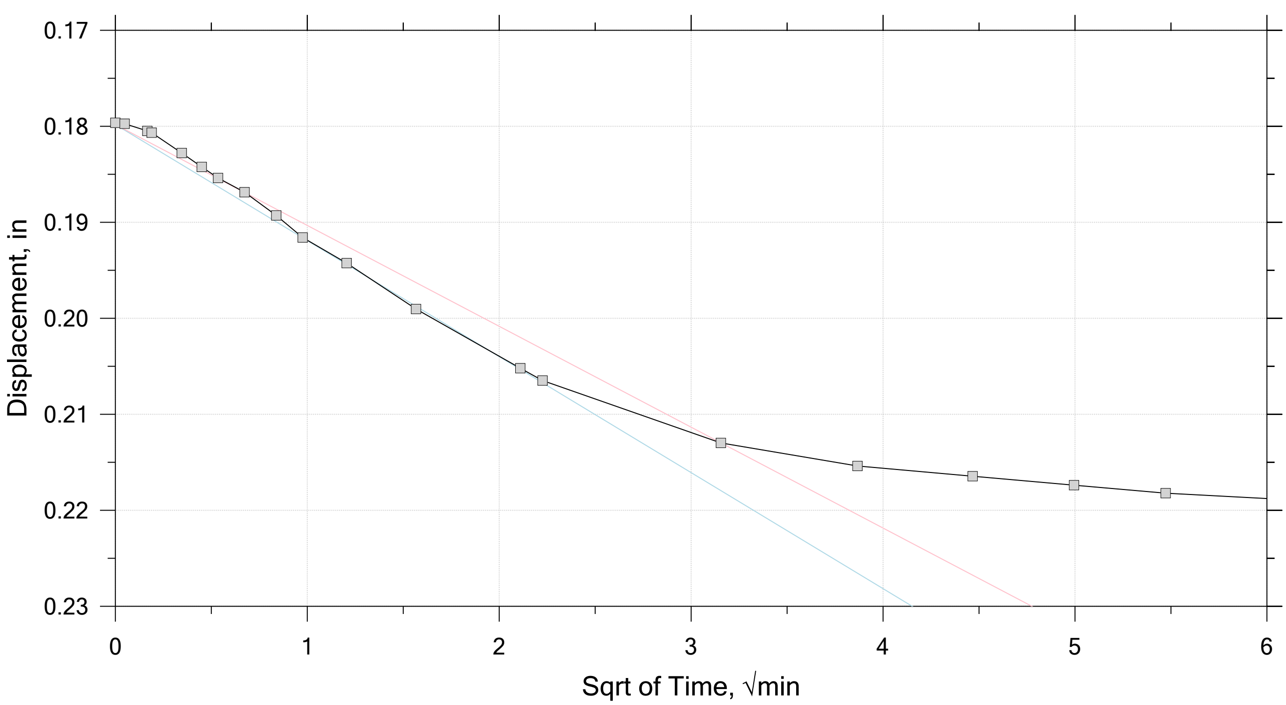
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 16

Constant Load Step

Stress: 2.05e+04 psf





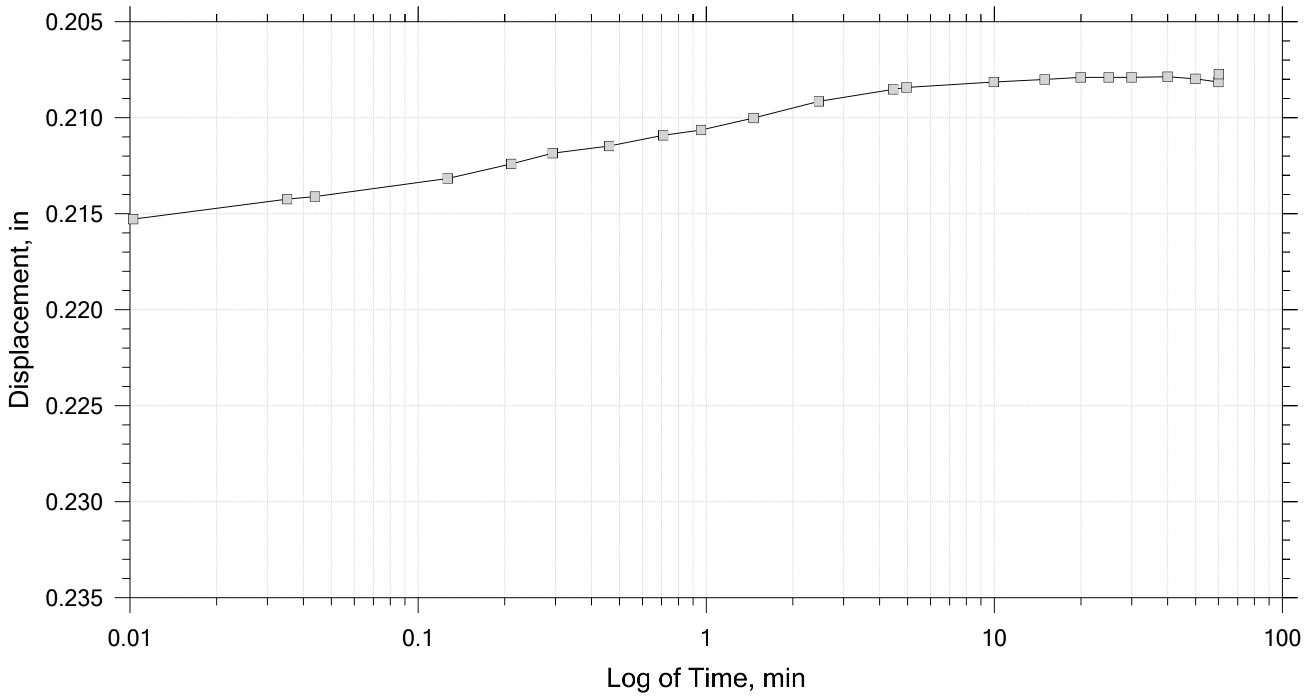
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

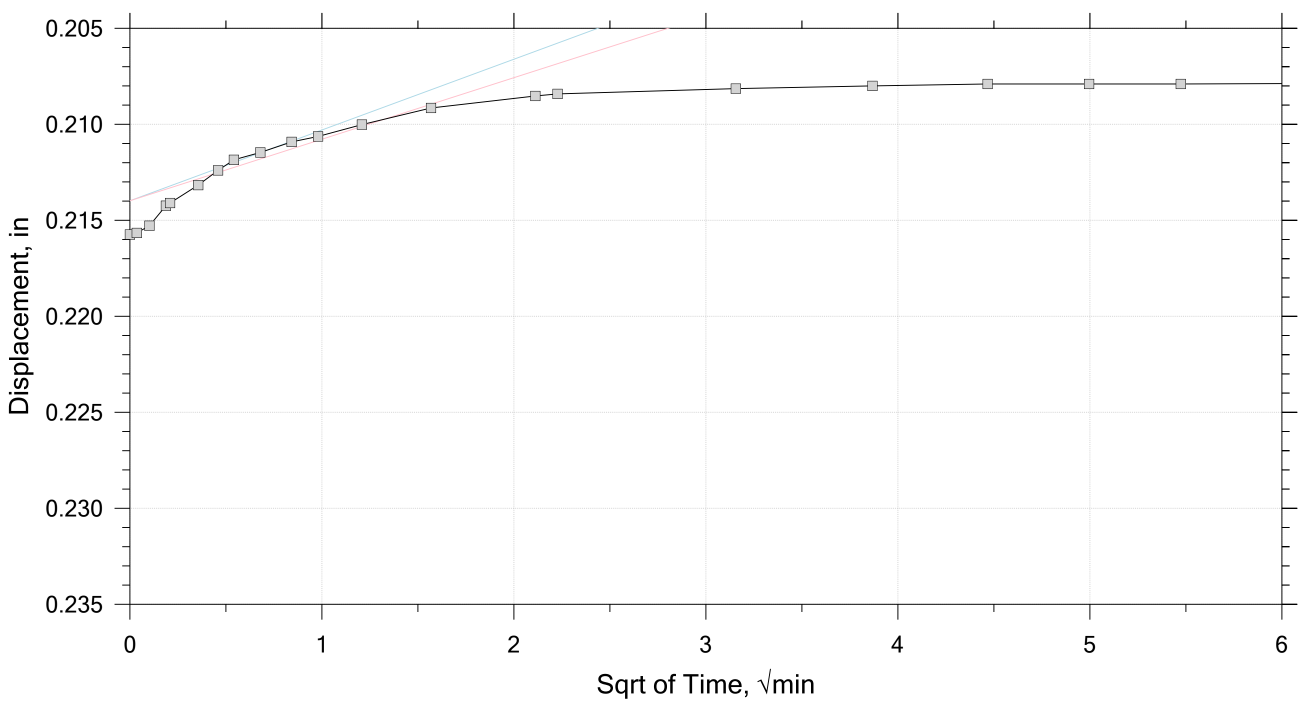
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 16

Constant Load Step

Stress: 5.13e+03 psf

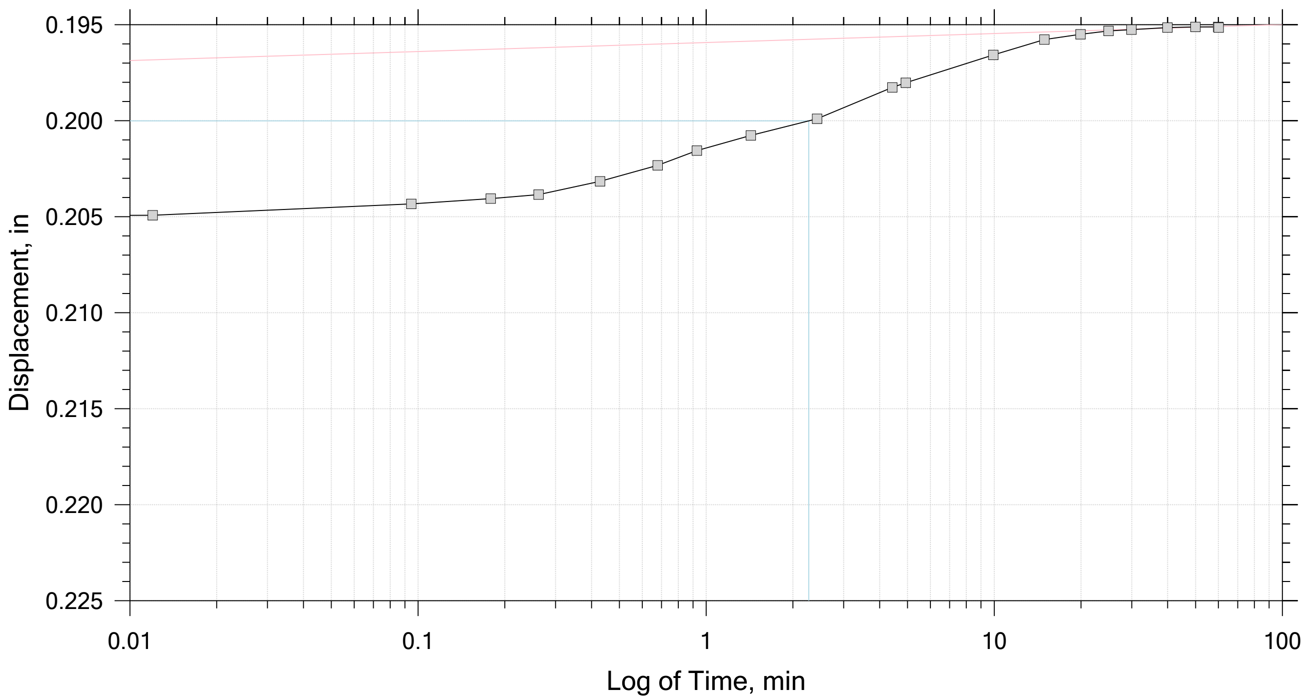


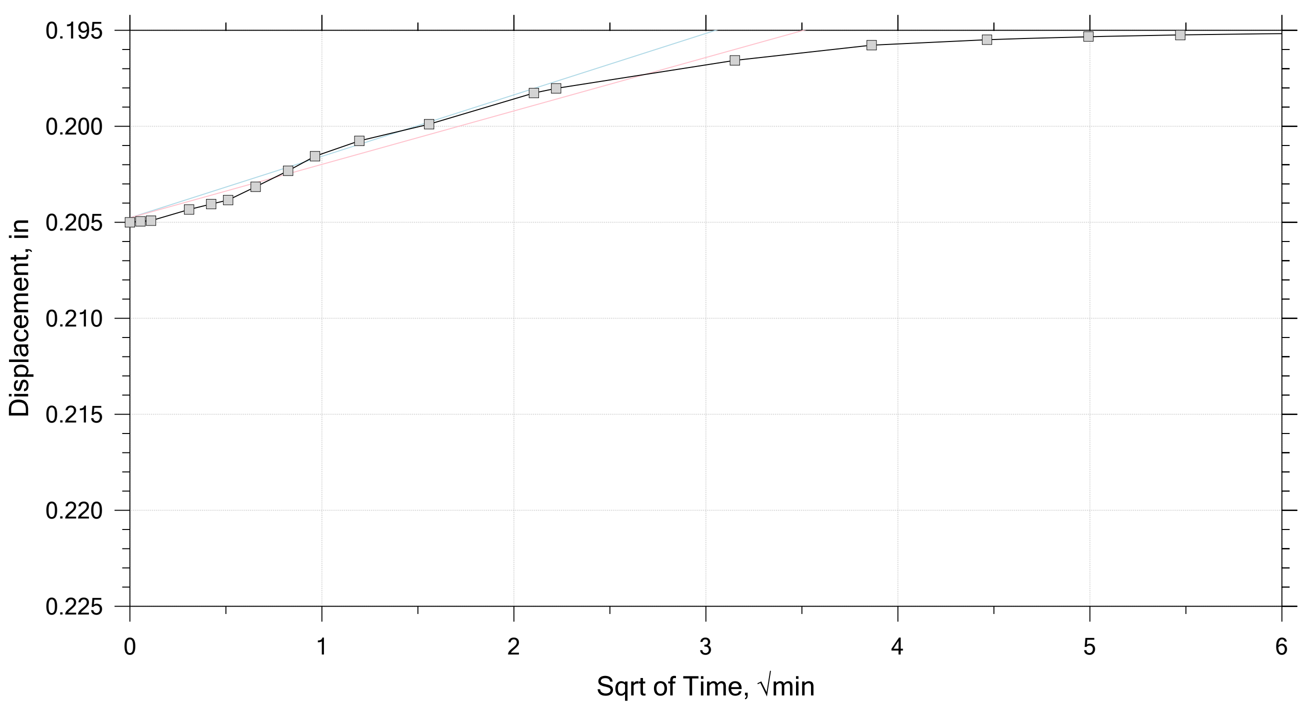


	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 16
Constant Load Step
Stress: 1.71e+03 psf





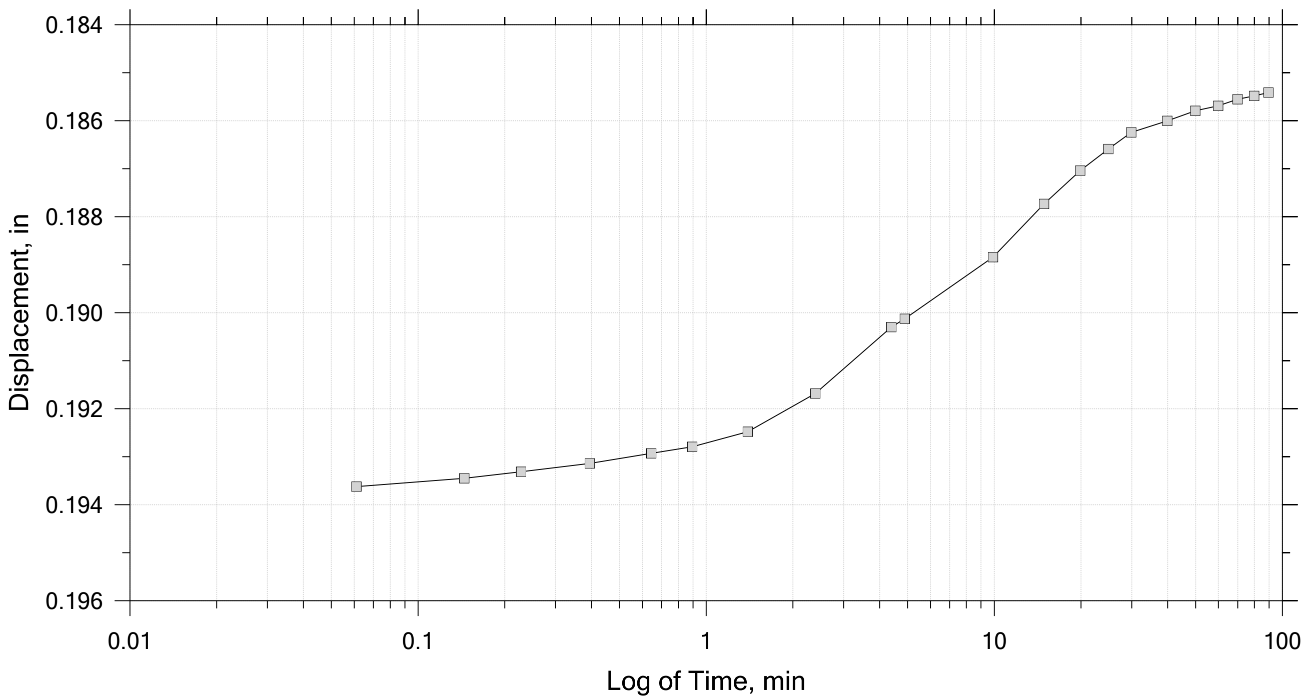
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

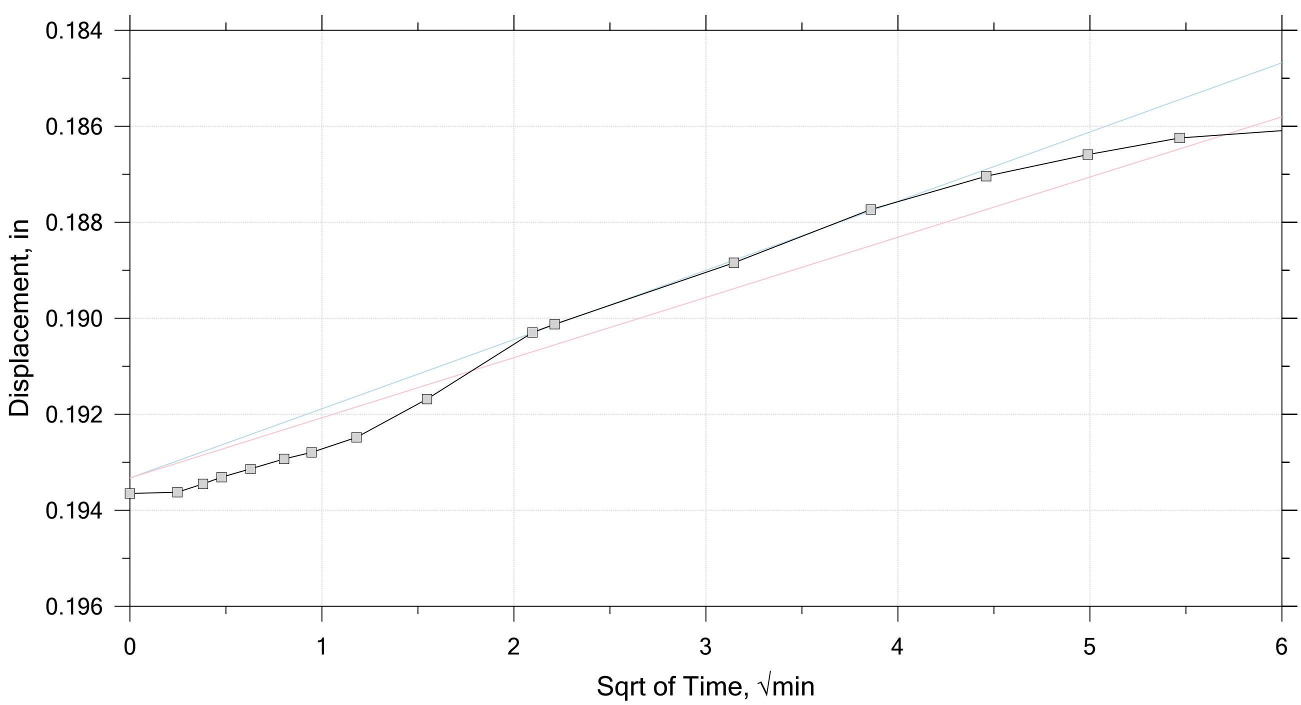
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 16

Constant Load Step

Stress: 760 psf





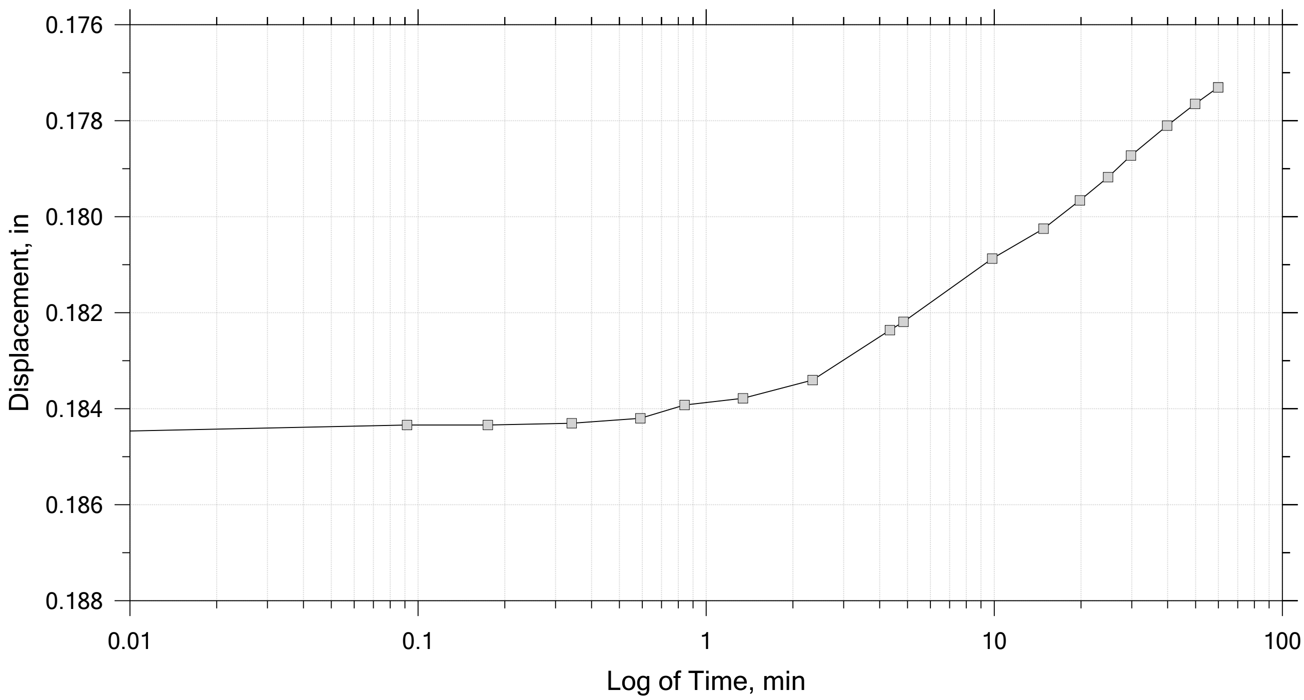
	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

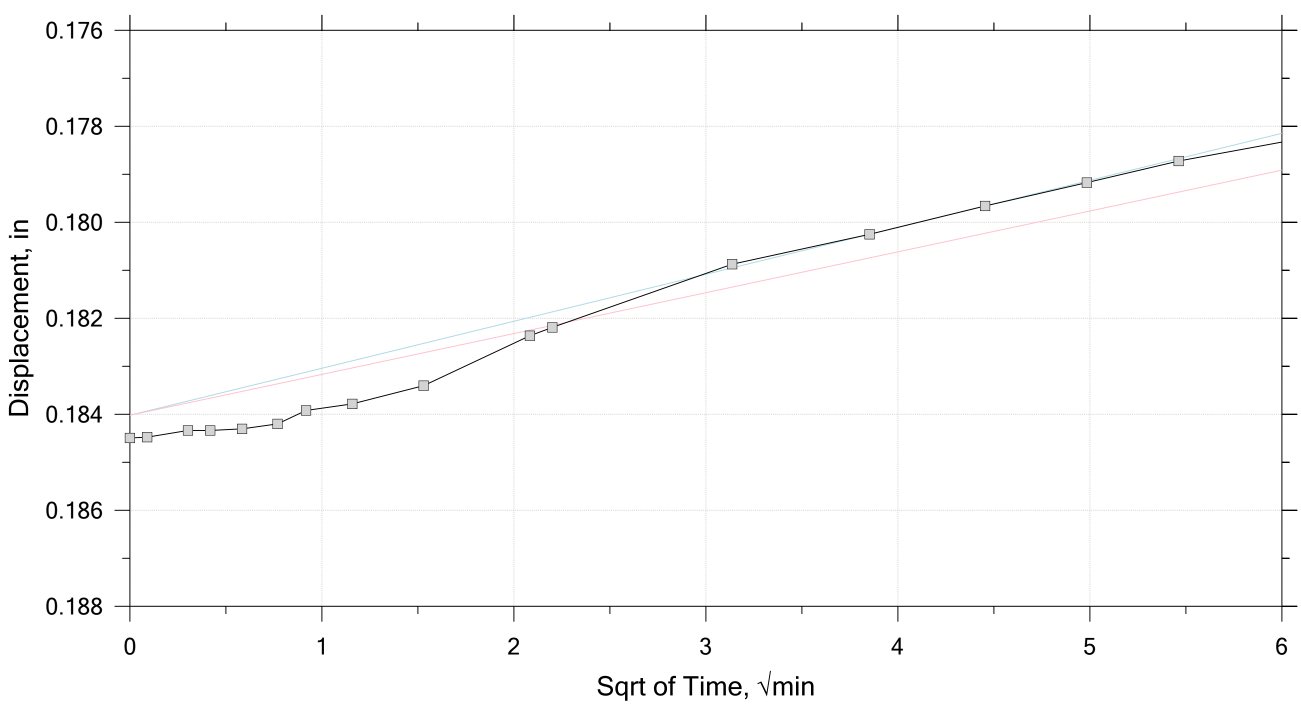
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 16

Constant Load Step

Stress: 338 psf





	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter, in: 2.50	Specific Gravity: 2.89 (Implied)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 1.23	Plastic Limit: 0
Final Height, in: 0.82	Final Void Ratio: 0.838	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	202	---	"ring"	307
Mass Container, gm	36.85	110.17	110.17	60.45
Mass Container + Wet Soil, gm	134.55	257.55	244.59	194.68
Mass Container + Dry Soil, gm	105.86	214.35	214.35	164.48
Mass Dry Soil, gm	69.01	104.18	104.18	104.03
Water Content, %	41.57	41.47	29.03	29.03
Void Ratio	---	1.23	0.84	---
Degree of Saturation, %	---	97.07	100.00	---
Dry Unit Weight, pcf	---	80.689	98.036	---

Preconsolidation Stress, psf	2200
Compression Ratio	0.22
Rebound Ratio	0.024
Compression Index	0.4913
Rebound Index	0.05359

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.

	Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
	Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
	Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
	Test Number: ICON306	Preparation: Shelby Tube	Elevation:
	Description: Gray Silty Clay		
	Remarks: Thin sand layer< 0.5 mm noted		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients

Step	Applied Stress psf	EOP Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv ft²/day	Mv ft²/ton	k cm/s
1	100.	0.004297	1.22	0.429	43.257	4.90e-02	8.58e+04	4.63e-08
2	150.	0.005717	1.22	0.571	44.709	4.71e-02	5.67e+04	2.94e-08
3	225.	0.008039	1.22	0.802	50.321	4.17e-02	6.18e+04	2.84e-08
4	338.	0.01143	1.21	1.14	58.914	3.54e-02	6.00e+04	2.34e-08
5	506.	0.01518	1.20	1.51	45.663	4.54e-02	4.45e+04	2.22e-08
6	759.	0.02013	1.19	2.01	53.955	3.81e-02	3.91e+04	1.64e-08
7	1.14e+03	0.02575	1.18	2.57	16.241	1.25e-01	2.95e+04	4.07e-08
8	1.71e+03	0.03739	1.15	3.73	47.450	4.21e-02	4.07e+04	1.89e-08
9	2.56e+03	0.05568	1.11	5.56	59.152	3.27e-02	4.28e+04	1.54e-08
10	5.13e+03	0.1197	0.966	11.9	34.943	5.07e-02	4.98e+04	2.78e-08
11	1.03e+04	0.1723	0.849	17.2	17.574	8.84e-02	2.05e+04	1.99e-08
12	2.05e+04	0.2169	0.750	21.6	9.972	1.39e-01	8.69e+03	1.33e-08
13	5.13e+03	0.2056	0.775	20.5	1.767	7.50e-01	1.47e+03	1.21e-08
14	1.71e+03	0.1957	0.797	19.5	7.195	1.89e-01	5.78e+03	1.20e-08
15	760.	0.1855	0.820	18.5	32.528	4.29e-02	2.14e+04	1.01e-08
16	338.	0.1773	0.838	17.7	55.986	2.55e-02	3.89e+04	1.09e-08

Project Name: Johnson Road Bridge	Location: Falmouth, ME	Project Number: 166-11
Boring Number: BB-FJR-104	Tester: SJR	Checker: SJR
Sample Number: U2	Test Date: 9/24/19	Depth: 41.38
Test Number: ICON306	Preparation: Shelby Tube	Elevation:
Description: Gray Silty Clay		
Remarks: Thin sand layer< 0.5 mm noted		
Displacement at End of Primary		

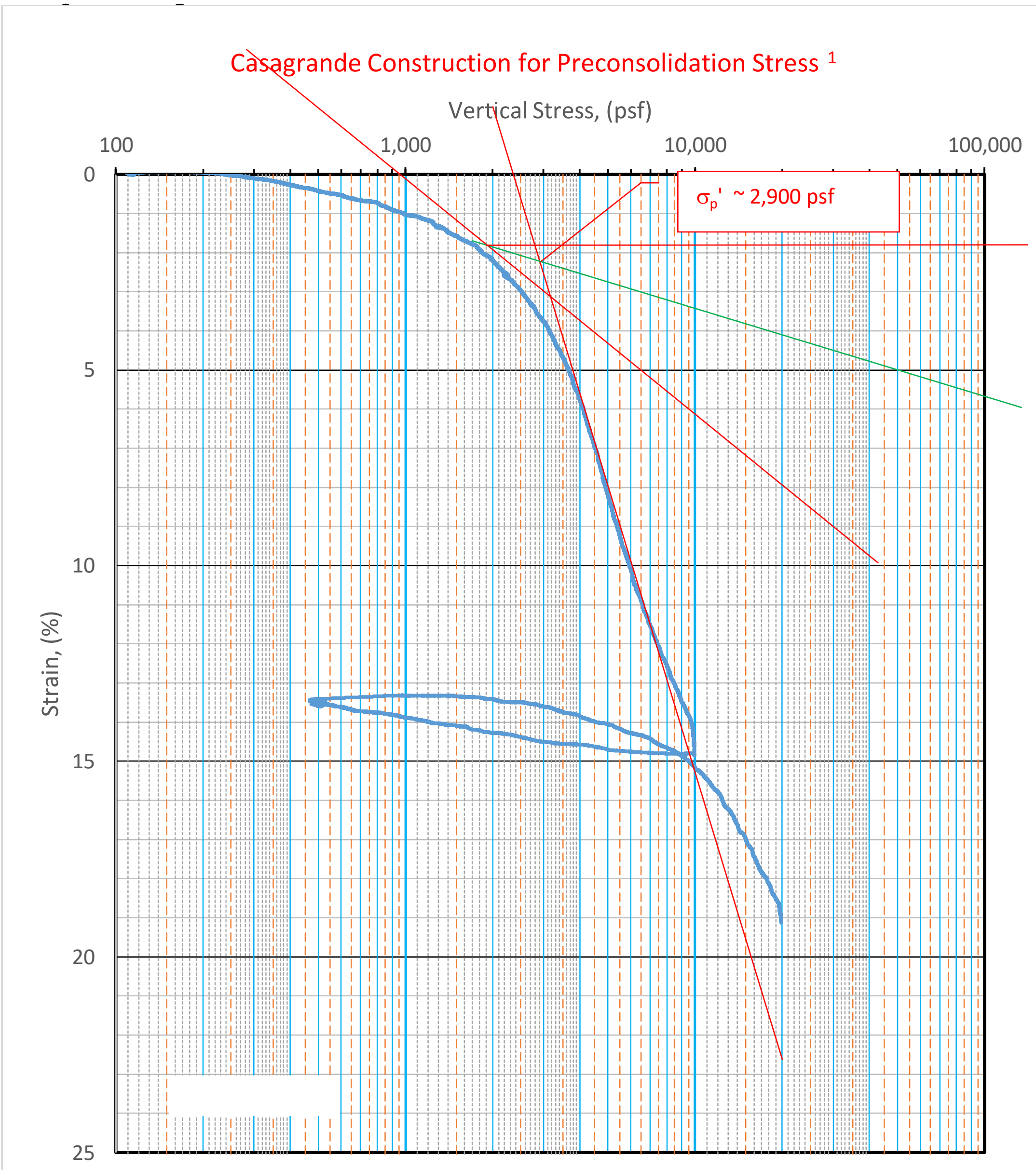
CONSTANT RATE OF STRAIN CONSOLIDATION

BB-FJR-102A 3U

Consolidation Test Data

Project Name:	Johnson Road Bridge
Project Number:	166-11
Project Location:	Falmouth, Maine
Client:	GZA Proj. No. 09.0026024.00
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	CRS 304			
Boring No.	BB-FJR-102A			
Sample No:	3U			
Boring Elevation (ft).	36.9			
Sample Depth (ft):	50 - 52			
Test Specimen Depth (Ft):	51.35			
Test Specimen Elevation:	-14.45			
Water Content (%):	37.86			
Dry Unit Weight (pcf):	83.27			
Wet Unit Weight (pcf):	114.80			
Saturation Before (%):	95.95			
Saturation After (%):	100			
Void Ratio Before:	1.18			
Void Ratio After:	0.76			
Overburden Pressure (psf):				
Max Previous stress (psf):	2,900			
Max Prev. stress (Work) (psf):				
OCR:				
Compression Index (C_{CE}):	0.24			
Recompression Index (C_{RE}):	0.018			
Liquid Limit:	39			
Plastic Limit:	21			
Plasticity Index:	18			
Liquidity Index:	0.9			
Specific Gravity (implied)	2.9			
Lab Vane Su at 21 ft. (psf)				
Tested By:	sjr			
Date Tested:	9/27/2019			
Checked By:	sjr			



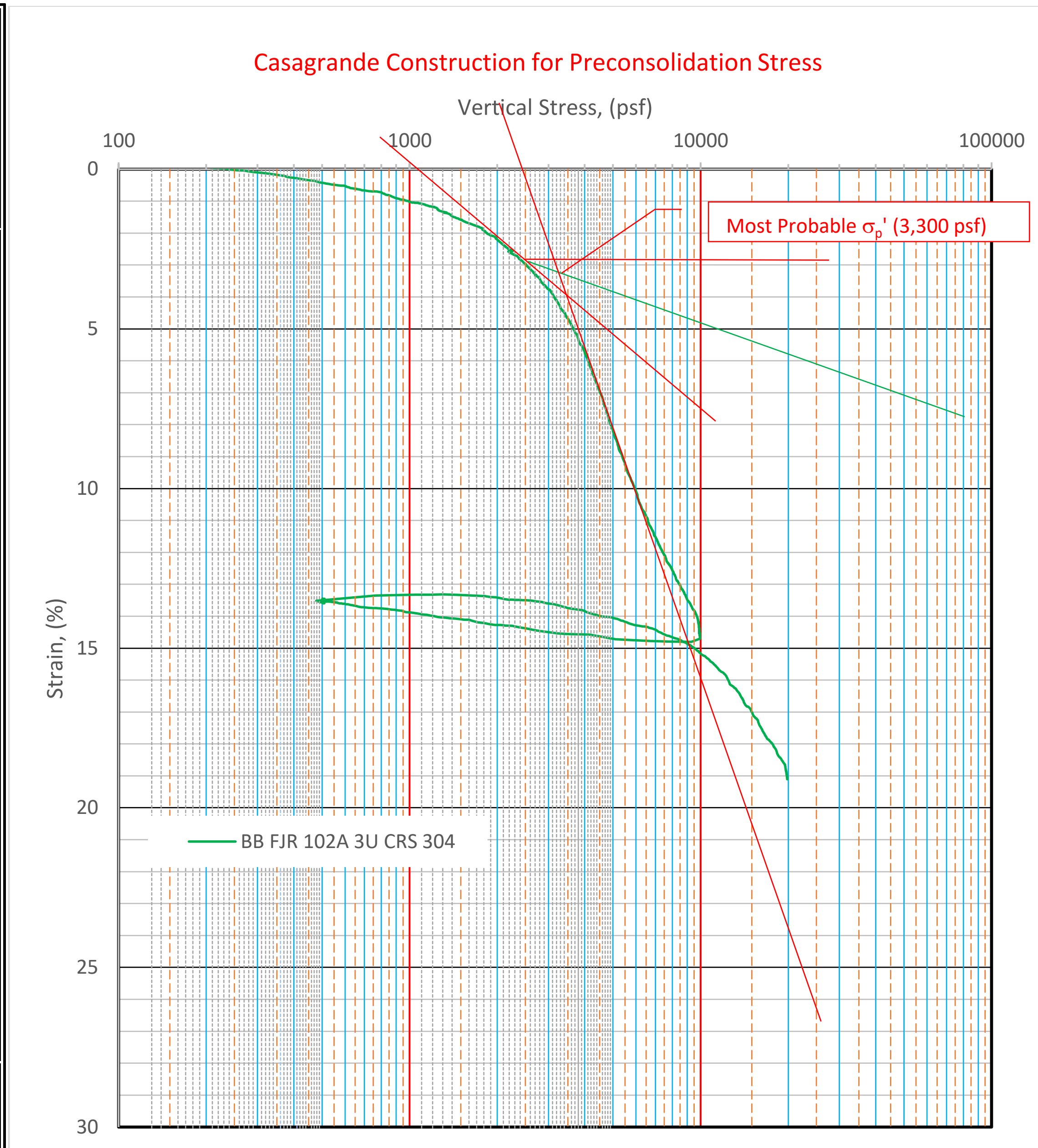
Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convienience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.

Soil Metrics LLC Geotechnical Engineering

Consolidation Test Data
Summary Report

Project Name:	Johnson Road Bridge
Project Number:	166-12
Project Location:	Falmouth Maine
Client:	GZA
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	CRS 304			
Boring No.	BB FJR 102A			
Sample No:	3U			
Boring Elevation (ft).	36.90			
Sample Depth (ft):	50-52			
Test Specimin Depth (Ft):	51.35			
Test Specimin Elevation:	11.75			
Water Content (%):	37.86			
Dry Unit Weight (pcf):	83.27			
Wet Unit Weight (pcf):	114.80			
Saturation Before (%):	95.95			
Saturation After (%):	100			
Void Ratio Before:	1.18			
Void Ratio After:	0.76			
Overburden Pressure (psf):				
Max Previous stress (psf):	3,300			
Max Prev. stress (PP Ratio) (psf):	3,500			
OCR:				
Compression Index (C_{CE}):	0.26			
Recompression Index (C_{RE}):	0.019			
Liquid Limit:	39			
Plastic Limit:	21			
Plasticity Index:	18			
Liquidity Index:	0.9			
Specific Gravity (estimated)	2.9			
Tested By:	sjr			
Date Tested:	9/27/2019			
Checked By:	sjr			

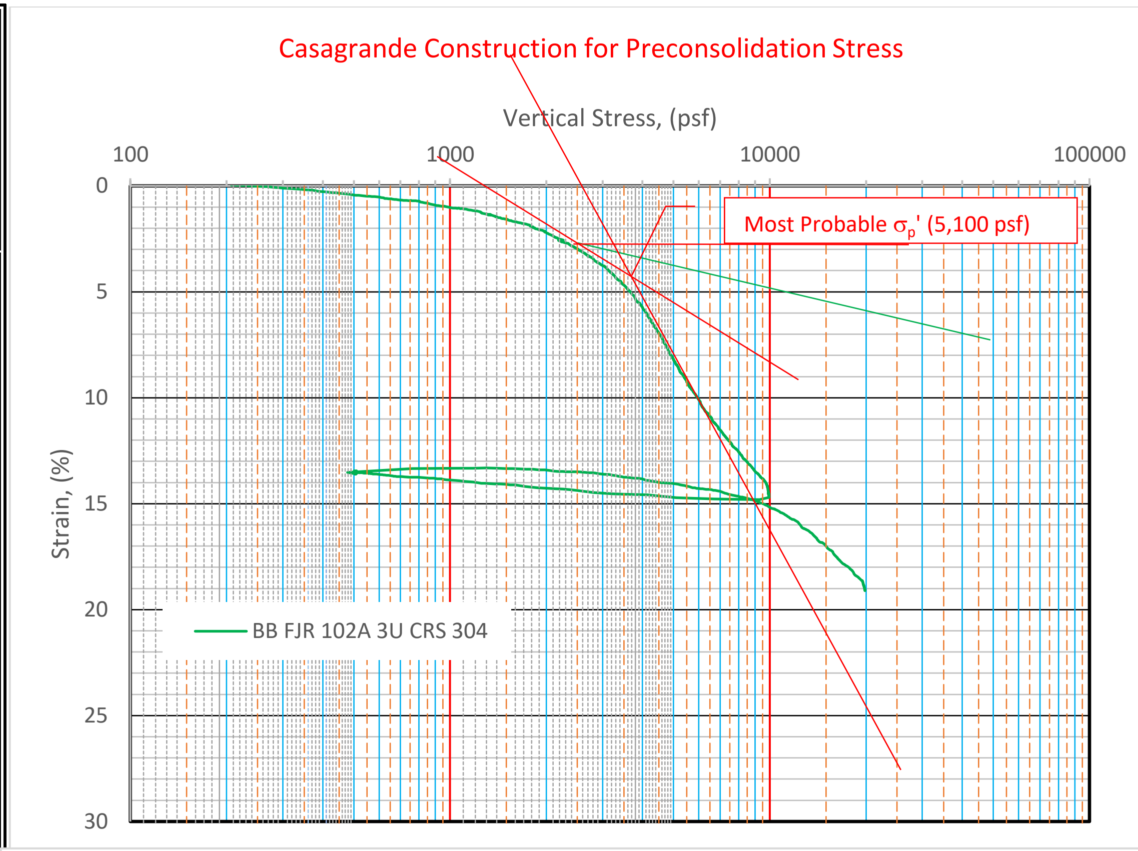


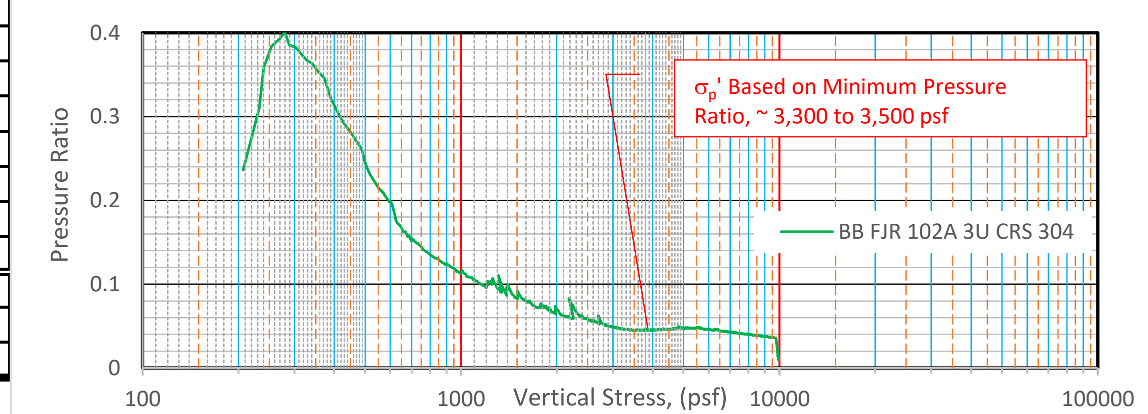
Soil Metrics LLC Geotechnical Engineering

Consolidation Test Data
Summary Report

Project Name:	Johnson Road Bridge
Project Number:	166-12
Project Location:	Falmouth Maine
Client:	GZA
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	CRS 304			
Boring No.	BB FJR 102A			
Sample No:	3U			
Boring Elevation (ft).	36.90			
Sample Depth (ft):	50-52			
Test Specimin Depth (Ft):	51.35			
Test Specimin Elevation:	11.75			
Water Content (%):	37.86			
Dry Unit Weight (pcf):	83.27			
Wet Unit Weight (pcf):	114.80			
Saturation Before (%):	95.95			
Saturation After (%):	100			
Void Ratio Before:	1.18			
Void Ratio After:	0.76			
Overburden Pressure (psf):				
Max Previous stress (psf):	3,300			
Max Prev. stress (PP Ratio) (psf):	3,500			
OCR:				
Compression Index (C_{CE}):	0.26			
Recompression Index (C_{RE}):	0.019			
Liquid Limit:	39			
Plastic Limit:	21			
Plasticity Index:	18			
Liquidity Index:	0.9			
Specific Gravity (estimated)	2.9			
Tested By:	sjr			
Date Tested:	9/27/2019			
Checked By:	sjr			



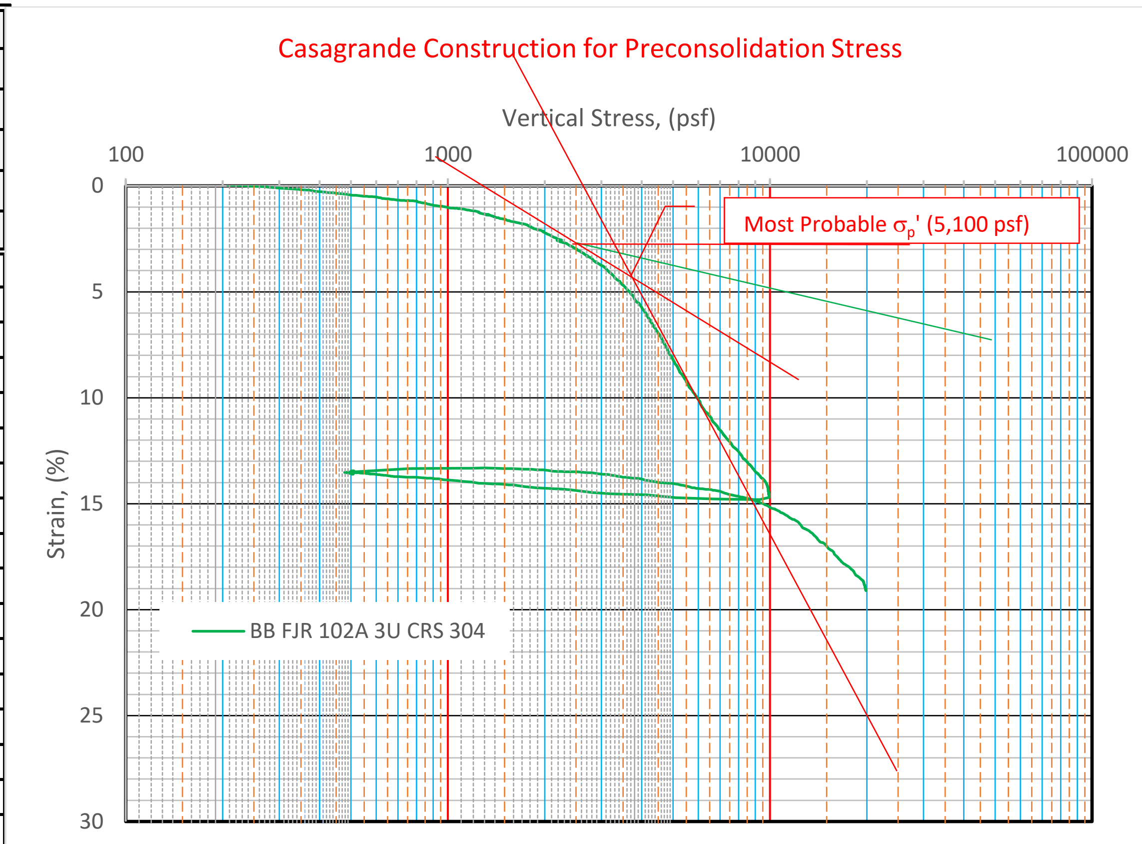


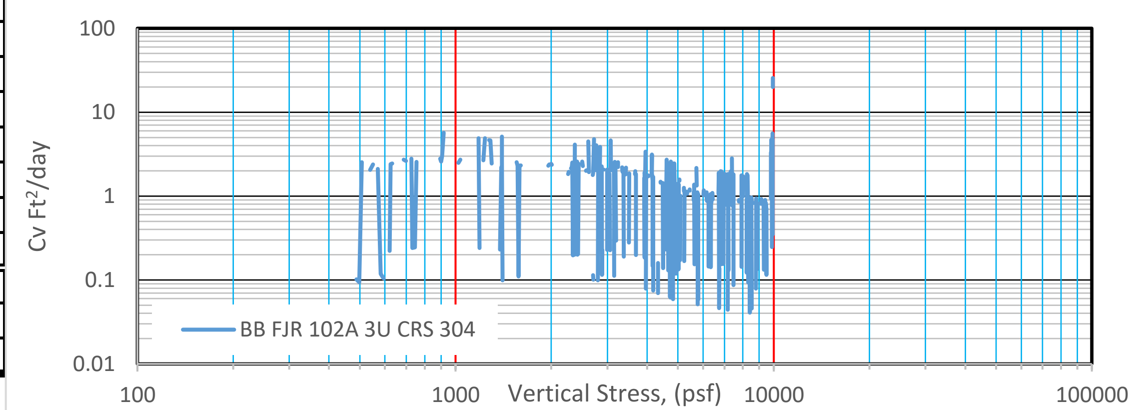
Soil Metrics LLC Geotechnical Engineering

Consolidation Test Data
Summary Report

Project Name:	Johnson Road Bridge
Project Number:	166-12
Project Location:	Falmouth Maine
Client:	GZA
Sample Description:	Gray Silty Clay
Preparation:	Trimmed Shelby Tube

Lab Test No:	CRS 304			
Boring No.	BB FJR 102A			
Sample No:	3U			
Boring Elevation (ft).	36.90			
Sample Depth (ft):	50-52			
Test Specimin Depth (Ft):	51.35			
Test Specimin Elevation:	11.75			
Water Content (%):	37.86			
Dry Unit Weight (pcf):	83.27			
Wet Unit Weight (pcf):	114.80			
Saturation Before (%):	95.95			
Saturation After (%):	100			
Void Ratio Before:	1.18			
Void Ratio After:	0.76			
Overburden Pressure (psf):				
Max Previous stress (psf):	3,300			
Max Prev. stress (PP Ratio) (psf):	3,500			
OCR:				
Compression Index (C_{CE}):	0.26			
Recompression Index (C_{RE}):	0.019			
Liquid Limit:	39			
Plastic Limit:	21			
Plasticity Index:	18			
Liquidity Index:	0.9			
Specific Gravity (estimated)	2.9			
Tested By:	sjr			
Date Tested:	9/27/2019			
Checked By:	sjr			

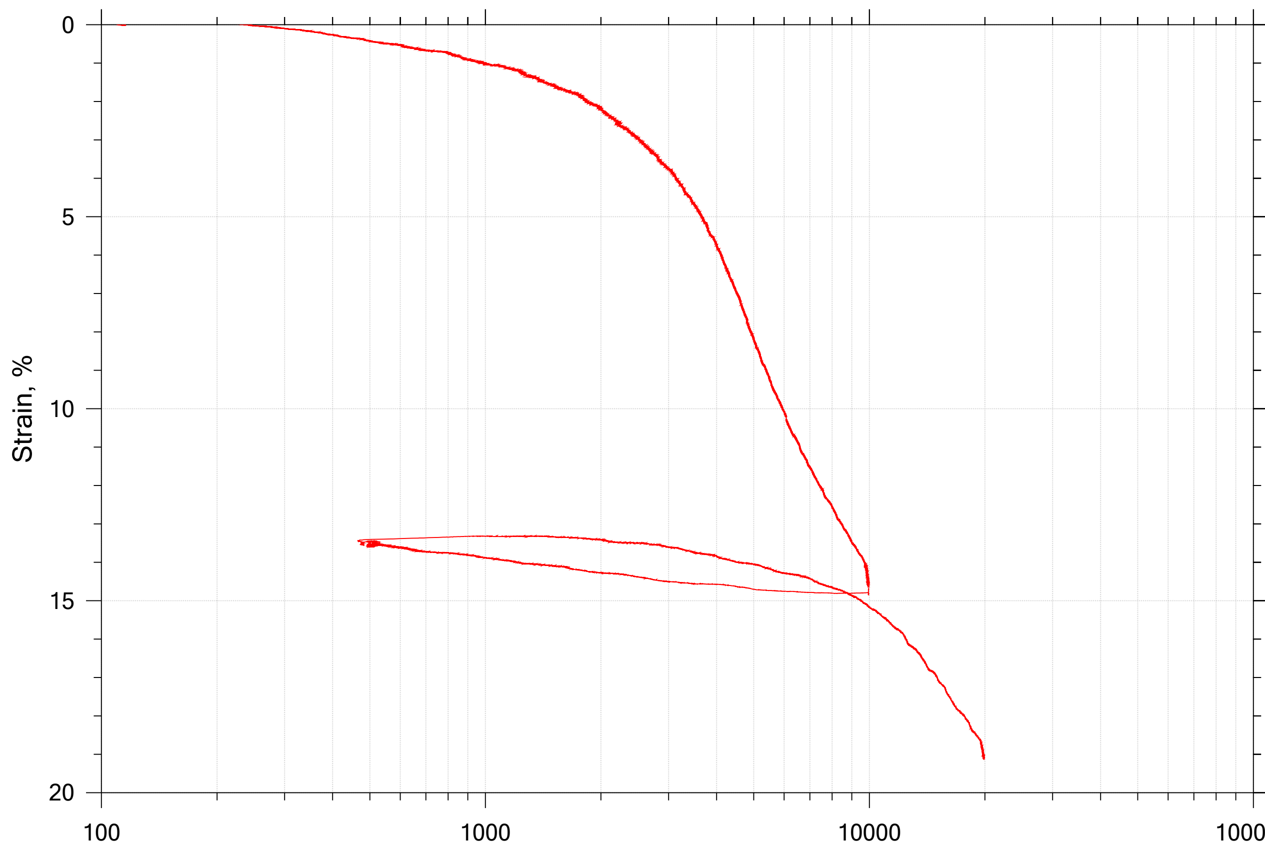


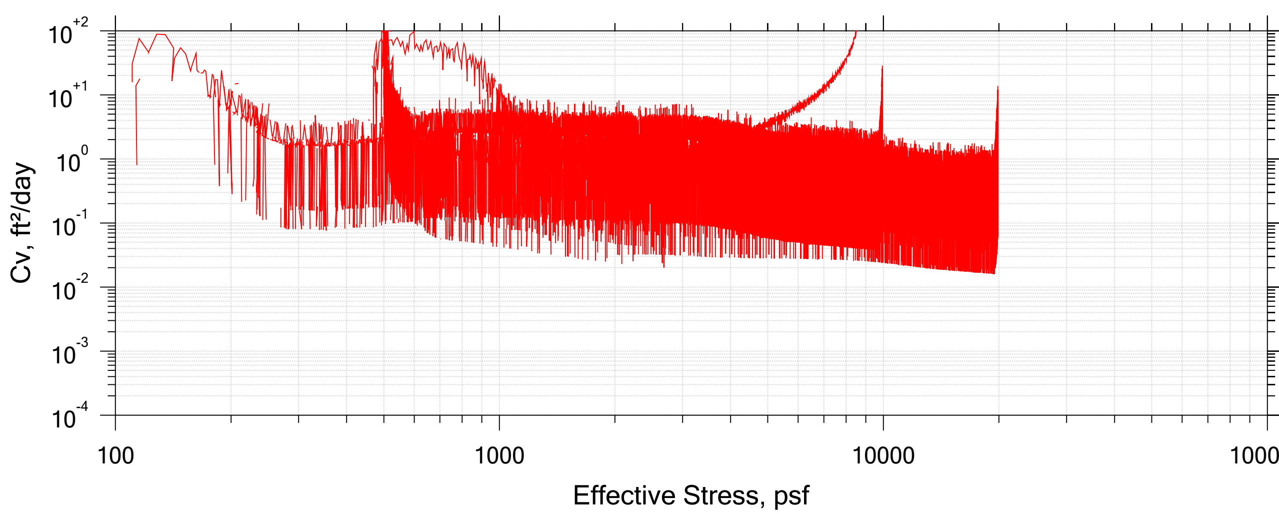


Soil Metrics LLC Geotechnical Engineering

CRC TEST

Summary Curves

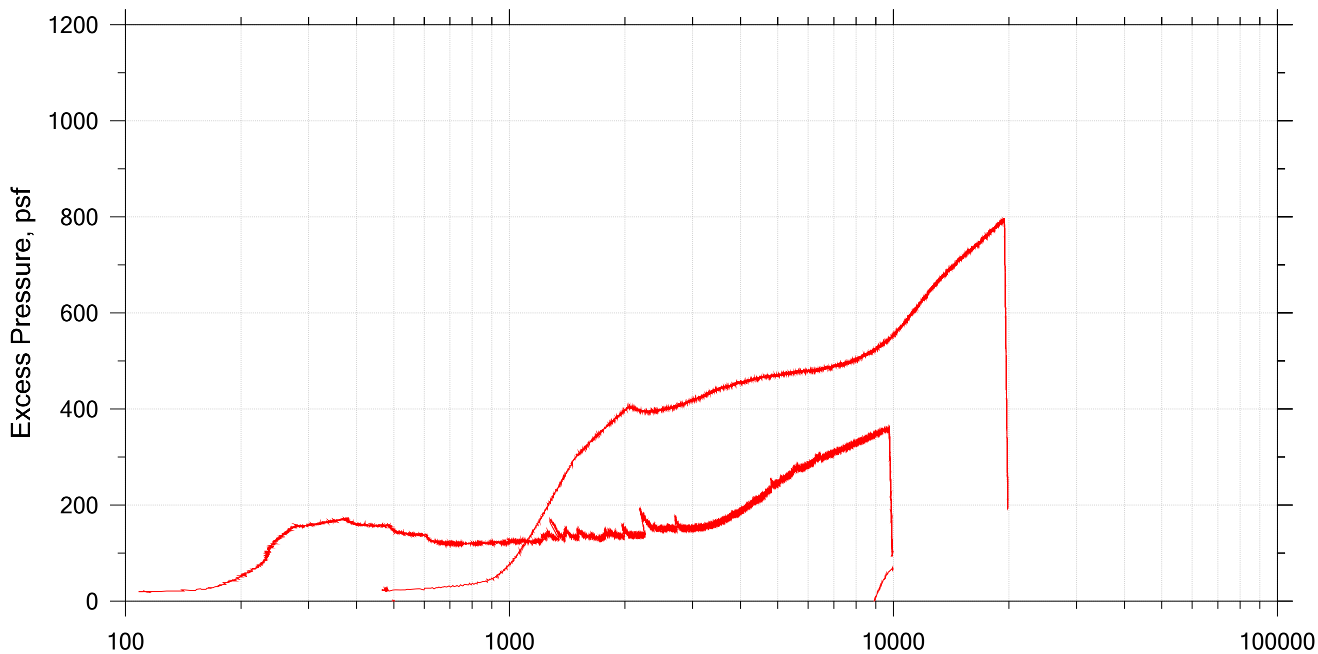


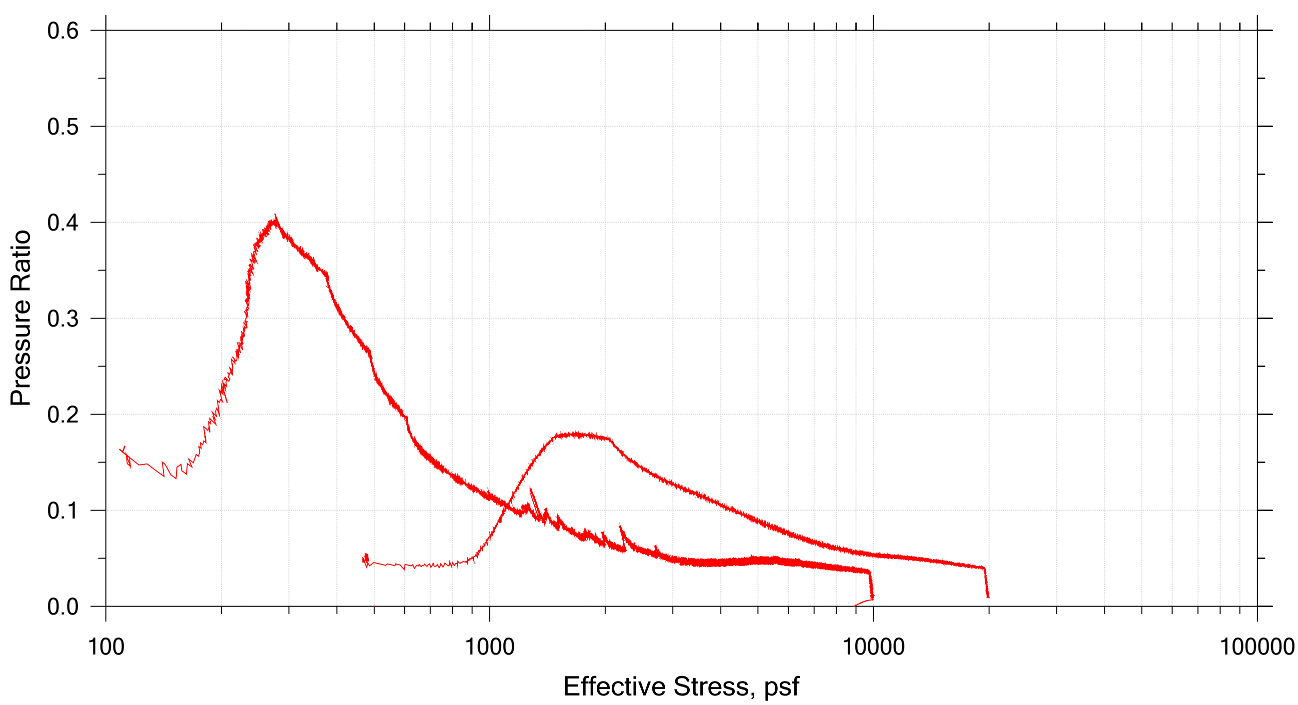


Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, ME	Project No.: 166.11
	Boring No.: BB FJR 102A	Tested By: SJR	Checked By: SJR
	Sample No.: 3U	Test Date: 9/28/19	Depth: 51.4
	Test No.: CRS 304	Sample Type: SJR	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

CRC TEST

Pressure Curves





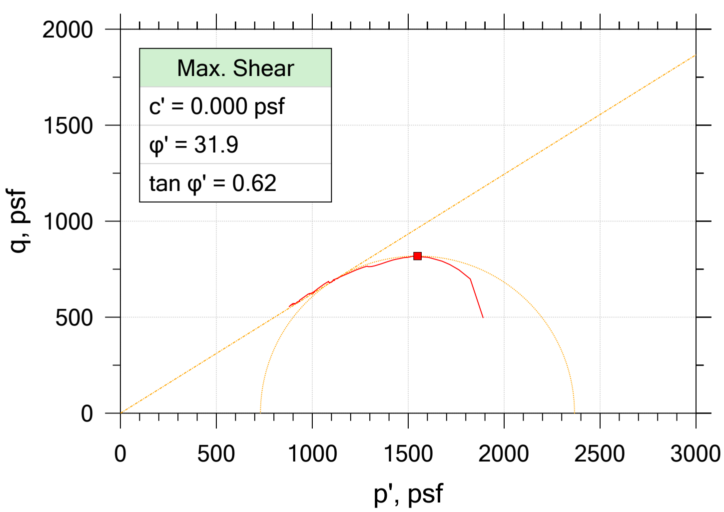
Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, ME	Project No.: 166.11
	Boring No.: BB FJR 102A	Tested By: SJR	Checked By: SJR
	Sample No.: 3U	Test Date: 9/28/19	Depth: 51.4
	Test No.: CRS 304	Sample Type: SJR	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

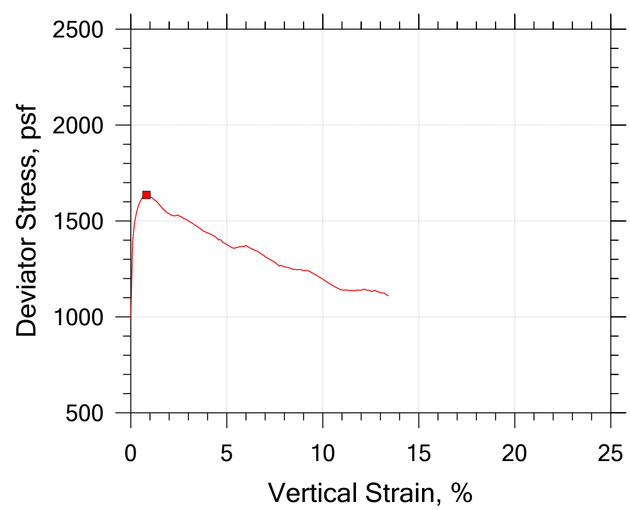
TRIAXIAL COMPRESSION

CK_OUC

BB-FJR-102A 1U

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



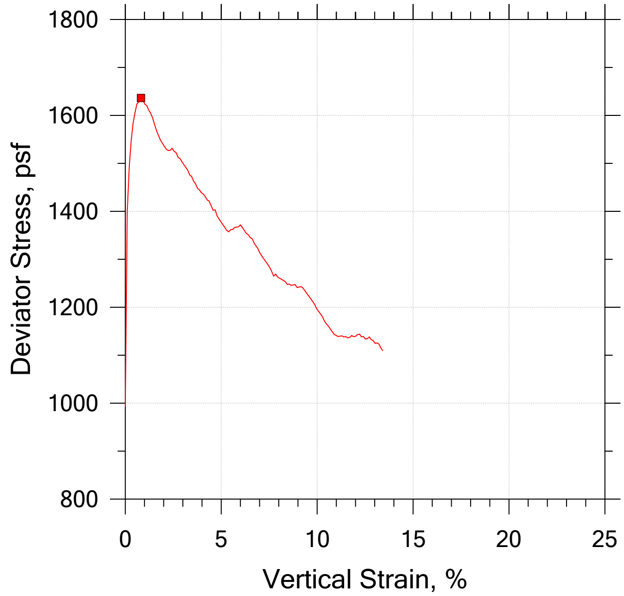


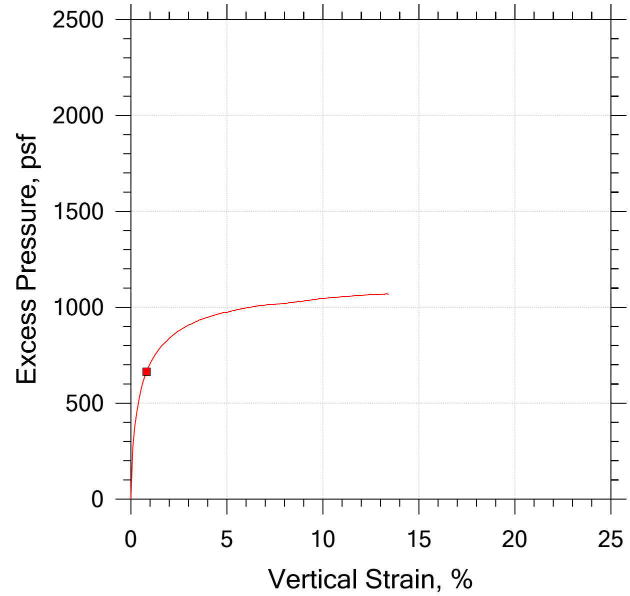
Symbol		■				
Sample ID		1U				
Depth, ft		31.75				
Test Number		CKoU 303				
Initial	Height, in	4.000				
	Diameter, in	2.000				
	Moisture Content (from Cuttings), %	37.4				
	Dry Density, pcf	84.5				
	Saturation (Wet Method), %	98.1				
	Void Ratio	1.07				
Final	Moisture Content, %	37.3				
	Dry Density, pcf	85.5				
	Cross-Sectional Area (Method A), in²	3.158				
	Saturation, %	100.0				
	Void Ratio	1.04				
	Back Pressure, %	1.165e+04				
Vertical Effective Consolidation Stress, psf		2365.				
Horizontal Effective Consolidation Stress, psf		1395.				
Vertical Strain after Consolidation, %		1.860				
Volumetric Strain after Consolidation, %		1.698				
Time to 50% Consolidation, min		0.0000				
Shear Strength, psf		818.2				
Strain at Failure, %		0.814				
Strain Rate, %/min		0.01500				
Deviator Stress at Failure, psf		1636.				
Effective Minor Principal Stress at Failure, psf		730.2				
Effective Major Principal Stress at Failure, psf		2367.				
B-Value		0.93				

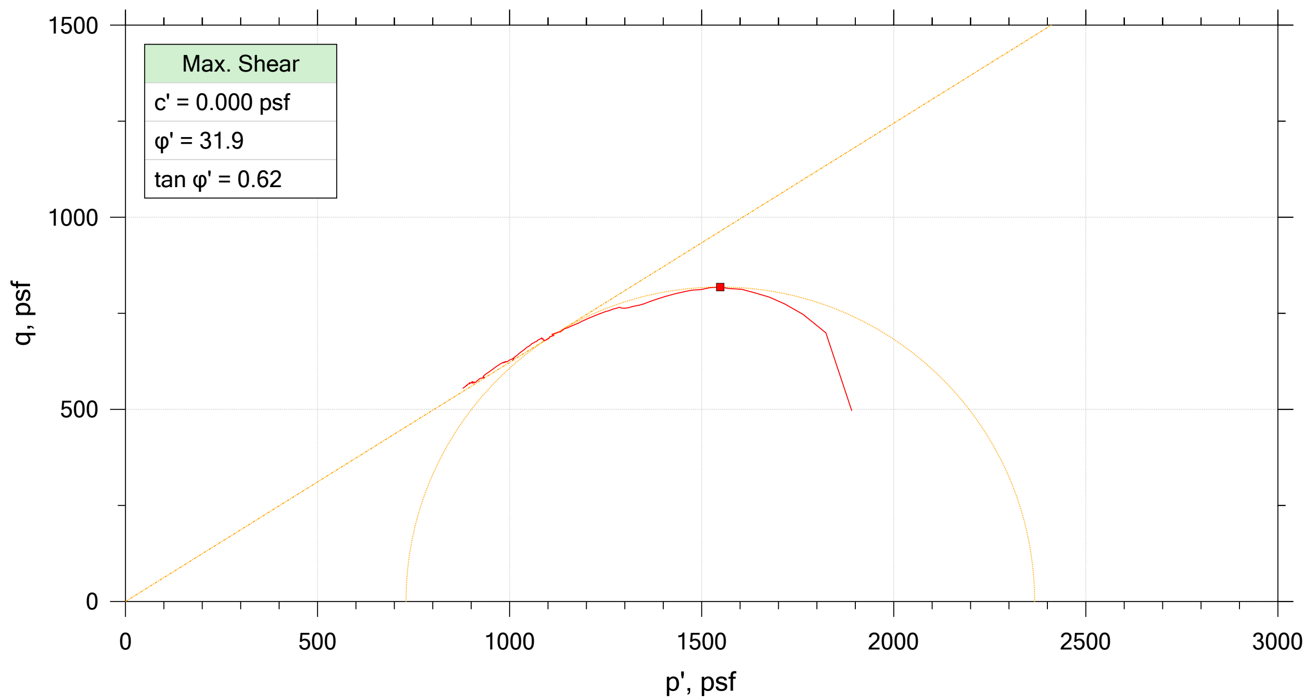
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Atterberg Limits determined by ASTM D4318.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, ME	Project No.: 166-11
	Boring No.: BB JRB 102A	Tested By: SJR	Checked By: sjr
	Sample No.: 1U	Test Date: 10/1/19	Depth: 31.75
	Test No.: CKoU 303	Sample Type: sjr	Elevation:
	Description: Gray Silty Clay with black streaks		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



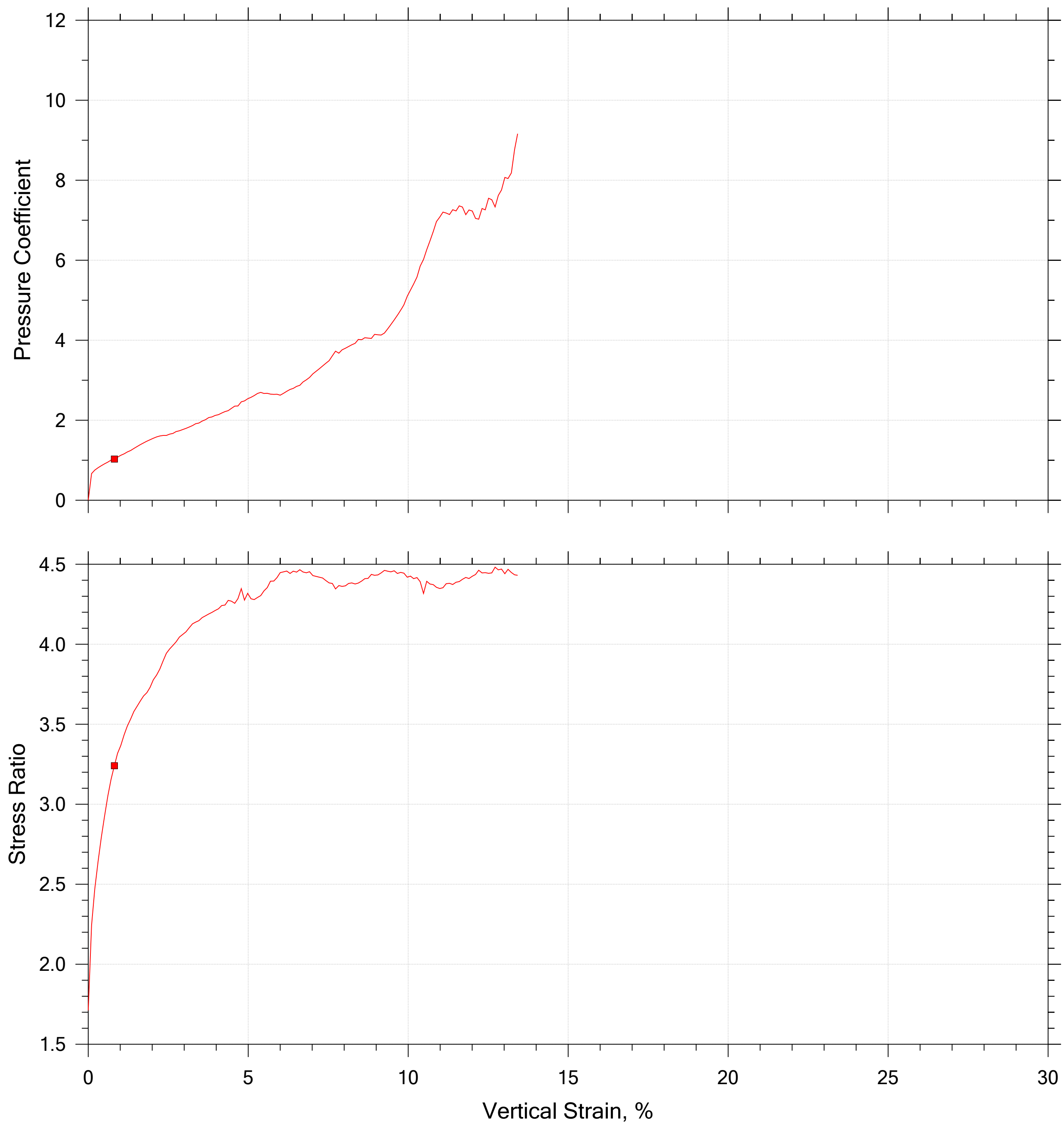




	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1U	CKoU 303	31.75	SJR	10/1/19	sjr	10/5/19	CKoU 303.dat

Project: Johnson Road Bridge	Location: Falmouth, ME	Project No.: 166-11
Boring No.: BB JRB 102A	Tested By: SJR	Checked By: sjr
Sample No.: 1U	Test Date: 10/1/19	Depth: 31.75
Test No.: CKoU 303	Sample Type: sjr	Elevation:
Description: Gray Silty Clay with black streaks		
Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

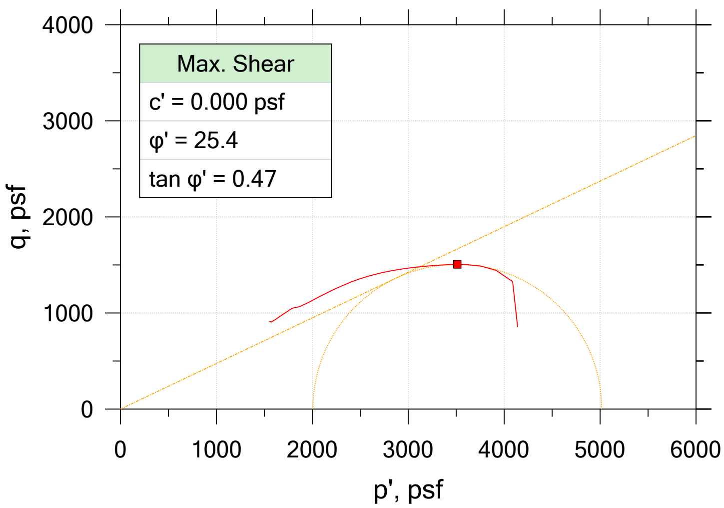


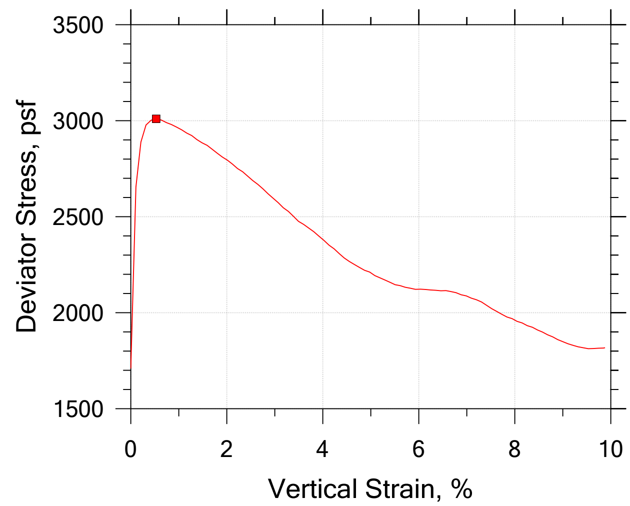
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1U	CKoU 303	31.75	SJR	10/1/19	sjr	10/5/19	CKoU 303.dat

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, ME	Project No.: 166-11
	Boring No.: BB JRB 102A	Tested By: SJR	Checked By: sjr
	Sample No.: 1U	Test Date: 10/1/19	Depth: 31.75
	Test No.: CKoU 303	Sample Type: sjr	Elevation:
	Description: Gray Silty Clay with black streaks		
	Remarks:		

BB-FJR-102A 3U

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



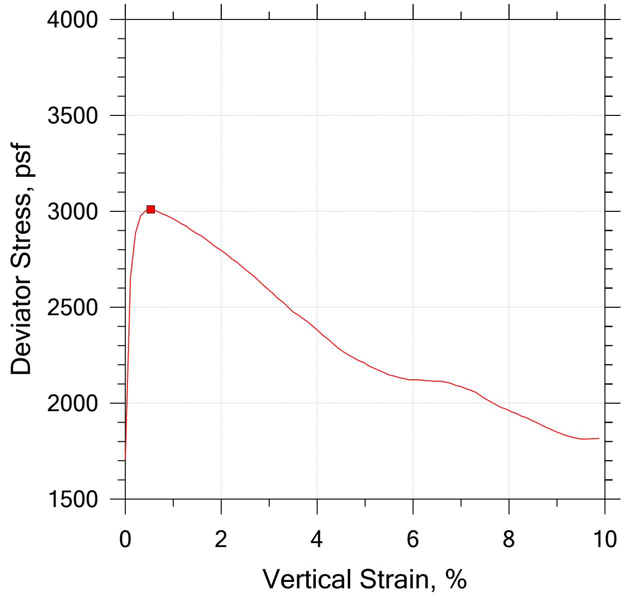


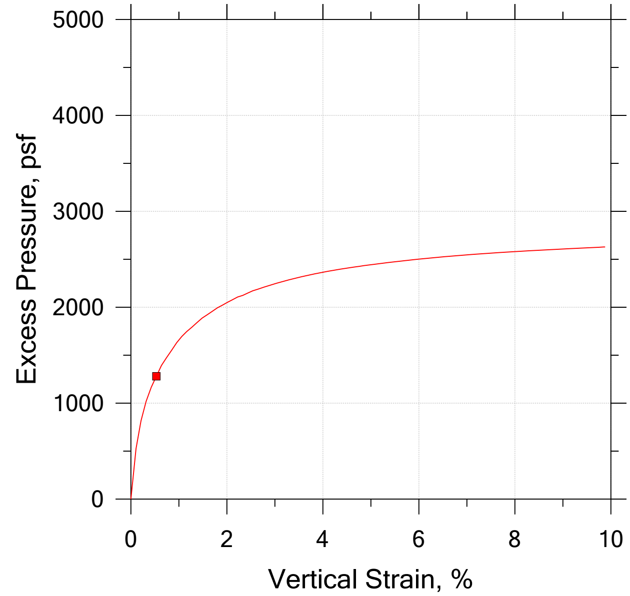
Symbol		■				
Sample ID		3U				
Depth, ft		51.75				
Test Number		CkoU 306				
Initial	Height, in	4.000				
	Diameter, in	2.000				
	Moisture Content (from Cuttings), %	39.7				
	Dry Density, pcf	84.5				
	Saturation (Wet Method), %	105.8				
	Void Ratio	1.03				
Final	Moisture Content, %	33.7				
	Dry Density, pcf	89.0				
	Cross-Sectional Area (Method A), in²	3.158				
	Saturation, %	100.0				
	Void Ratio	0.928				
	Back Pressure, %	1.123e+04				
Vertical Effective Consolidation Stress, psf		4926.				
Horizontal Effective Consolidation Stress, psf		3285.				
Vertical Strain after Consolidation, %		5.697				
Volumetric Strain after Consolidation, %		5.462				
Time to 50% Consolidation, min		0.0000				
Shear Strength, psf		1505.				
Strain at Failure, %		0.530				
Strain Rate, %/min		0.01500				
Deviator Stress at Failure, psf		3010.				
Effective Minor Principal Stress at Failure, psf		2006.				
Effective Major Principal Stress at Failure, psf		5017.				
B-Value		0.95				

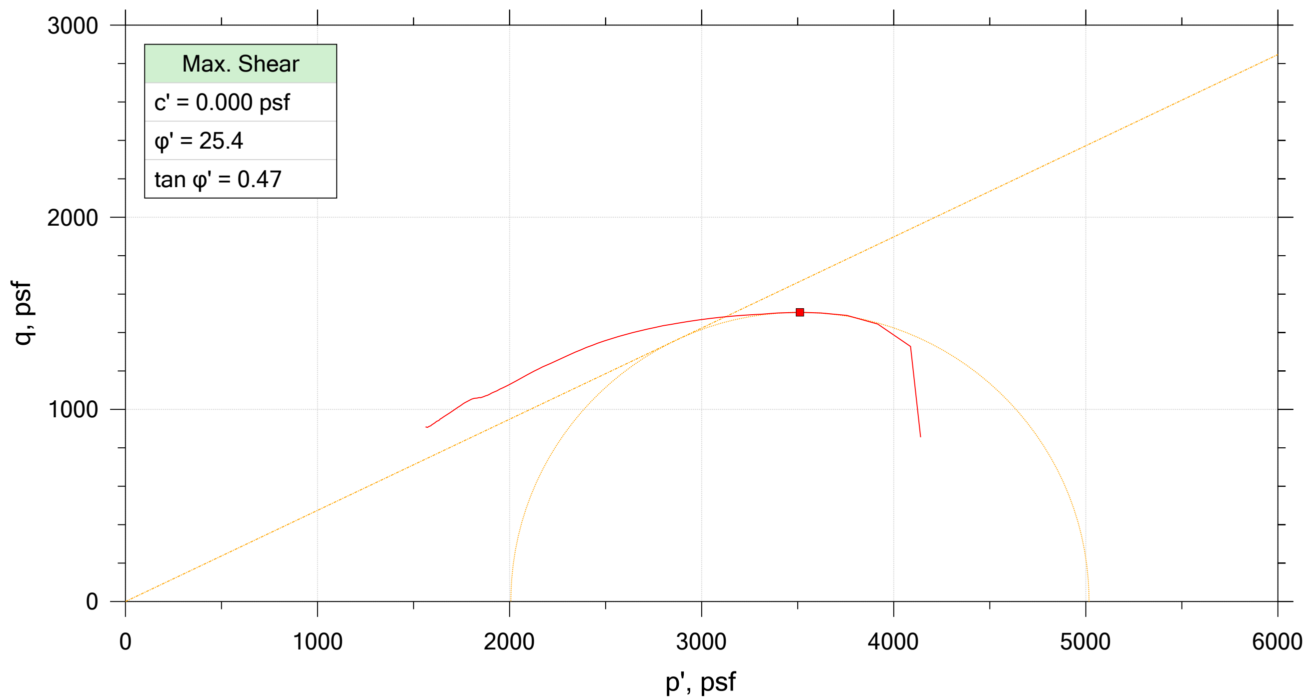
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Atterberg Limits determined by ASTM D4318.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Soil Metrics LLC			
	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 102a	Tested By: SJR	Checked By: sjr
	Sample No.: 3U	Test Date: 10/14/19	Depth: 51.75
	Test No.: CkoU 306	Sample Type:	Elevation:
	Description:		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



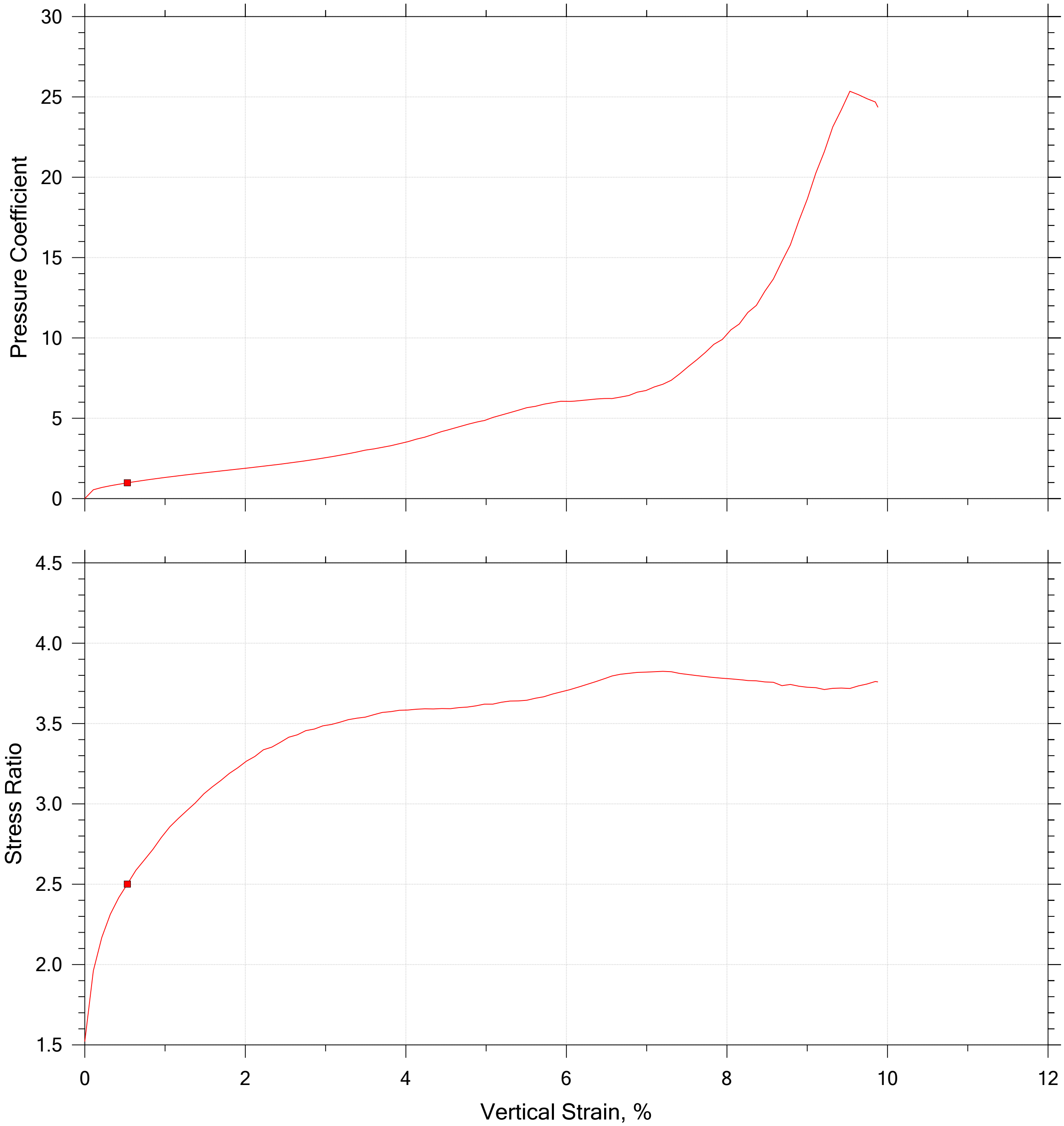




	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	3U	CkoU 306	51.75	SJR	10/14/19	sjr		CKoU 306.dat

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 102a	Tested By: SJR	Checked By: sjr
	Sample No.: 3U	Test Date: 10/14/19	Depth: 51.75
	Test No.: CkoU 306	Sample Type:	Elevation:
	Description:		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767

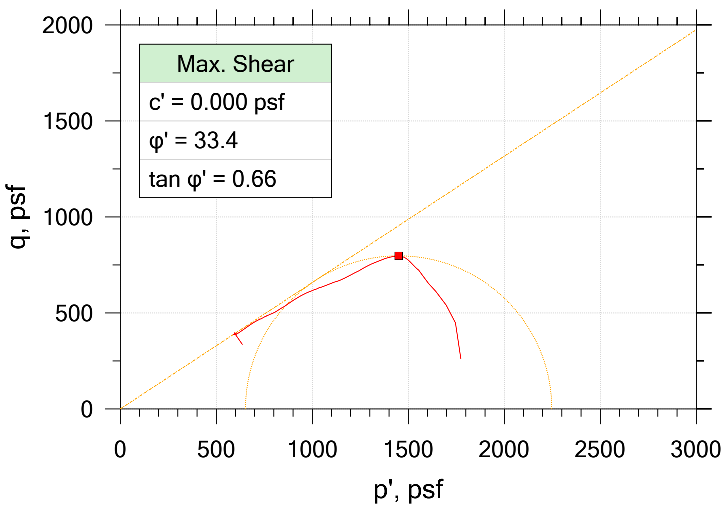


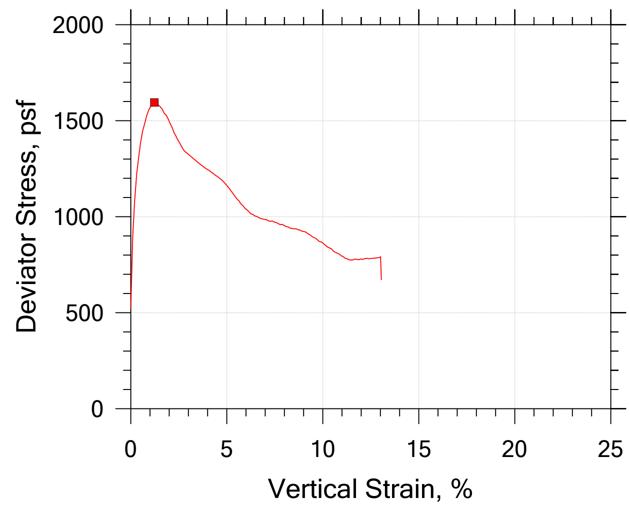
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	3U	CkoU 306	51.75	SJR	10/14/19	sjr		CKoU 306.dat

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 102a	Tested By: SJR	Checked By: sjr
	Sample No.: 3U	Test Date: 10/14/19	Depth: 51.75
	Test No.: CkoU 306	Sample Type:	Elevation:
	Description:		
	Remarks:		

BB-FJR-104 1U

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



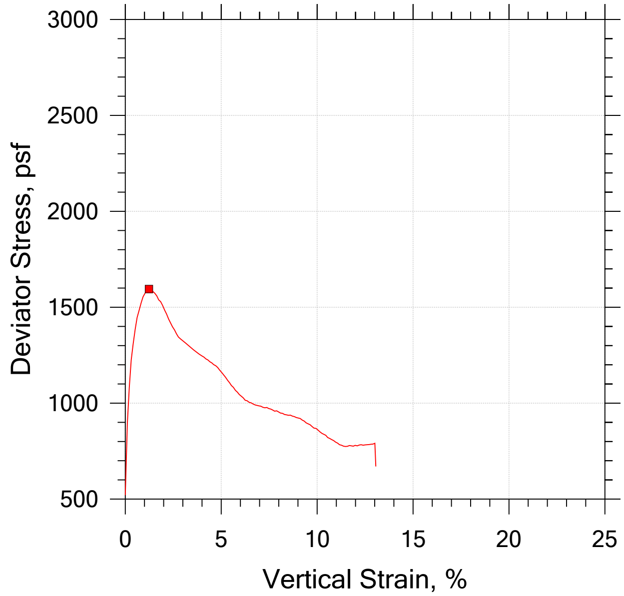


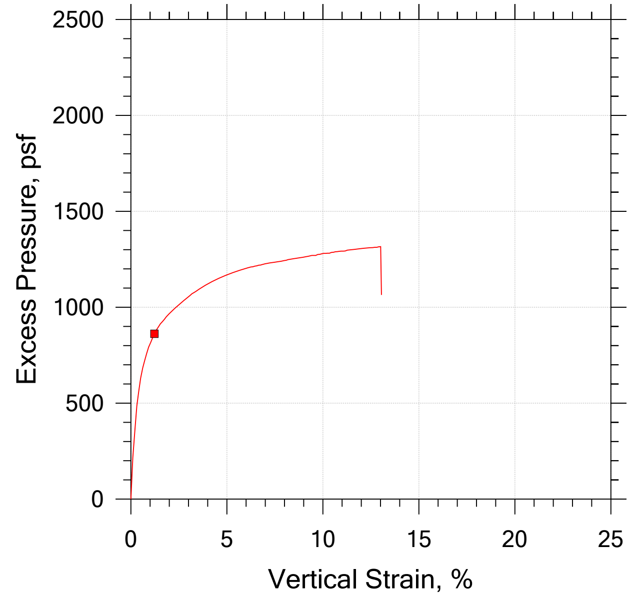
Symbol		■			
Sample ID		1U			
Depth, ft		31.43			
Test Number		CkoU 304			
Initial	Height, in	4.000			
Initial	Diameter, in	2.000			
Initial	Moisture Content (from Cuttings), %	41.8			
Initial	Dry Density, pcf	79.5			
Initial	Saturation (Wet Method), %	97.5			
Initial	Void Ratio	1.20			
Final	Moisture Content, %	41.5			
Final	Dry Density, pcf	80.9			
Final	Cross-Sectional Area (Method A), in²	3.162			
Final	Saturation, %	100.0			
Final	Void Ratio	1.16			
Final	Back Pressure, %	1.209e+04			
Vertical Effective Consolidation Stress, psf		2004.			
Horizontal Effective Consolidation Stress, psf		1514.			
Vertical Strain after Consolidation, %		2.526			
Volumetric Strain after Consolidation, %		2.188			
Time to 50% Consolidation, min		0.0000			
Shear Strength, psf		797.5			
Strain at Failure, %		1.23			
Strain Rate, %/min		0.01500			
Deviator Stress at Failure, psf		1595.			
Effective Minor Principal Stress at Failure, psf		652.6			
Effective Major Principal Stress at Failure, psf		2248.			
B-Value		0.95			

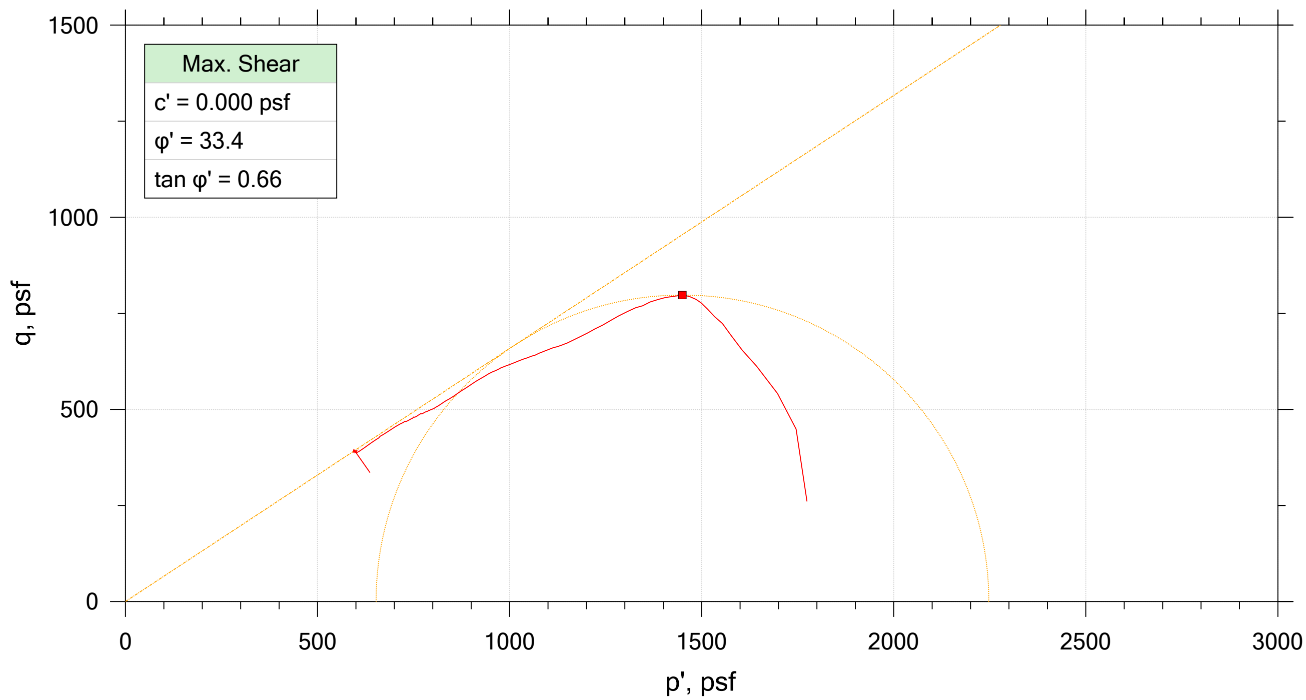
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Atterberg Limits determined by ASTM D4318.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
	Sample No.: 1U	Test Date: 10/5/19	Depth: 31.43
	Test No.: CkoU 304	Sample Type:	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



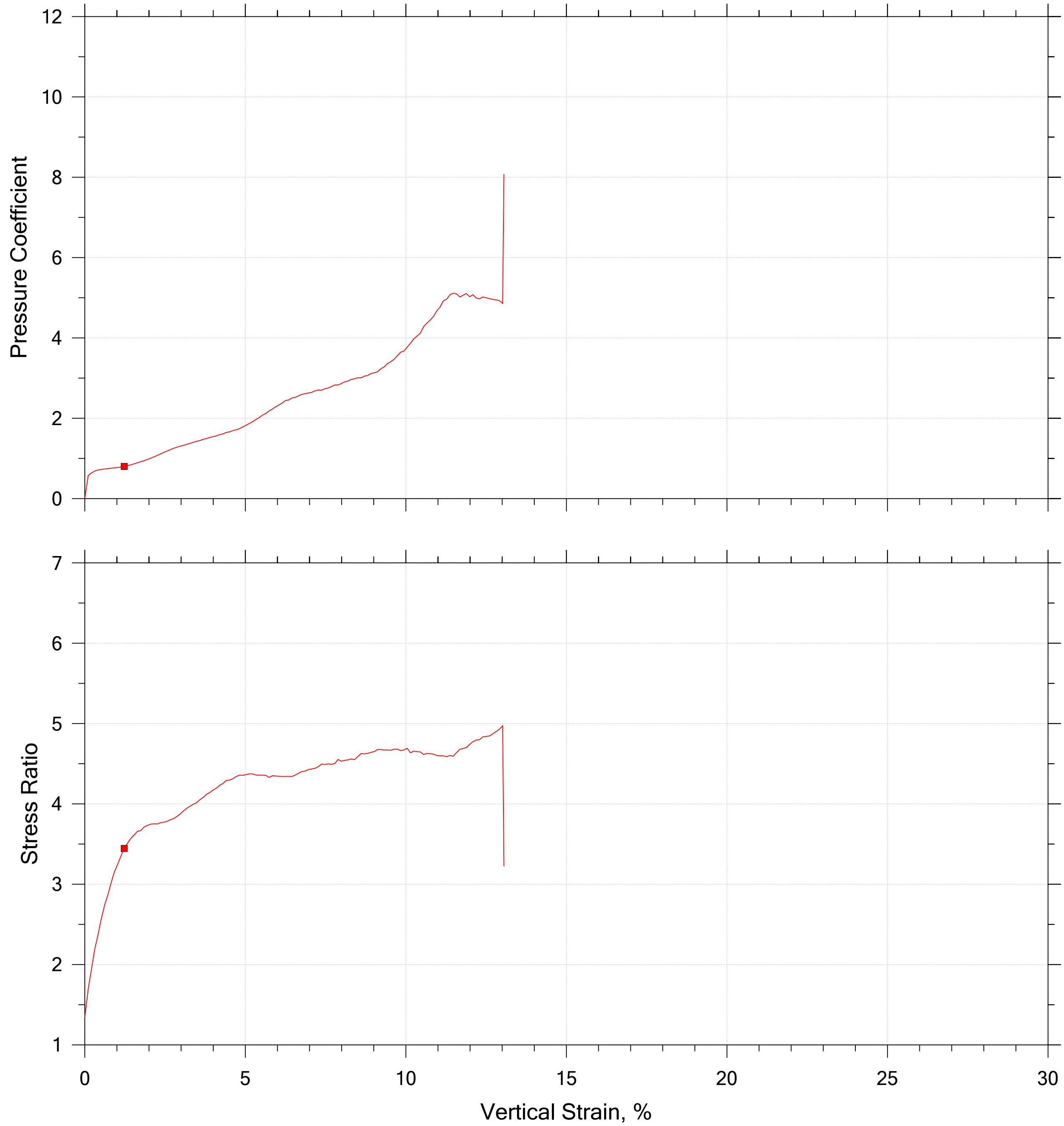




	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1U	CkoU 304	31.43	SJR	10/5/19	sjr	10/21/19	CKoU 304.dat

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
	Sample No.: 1U	Test Date: 10/5/19	Depth: 31.43
	Test No.: CkoU 304	Sample Type:	Elevation:
	Description: Gray Silty Clay		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



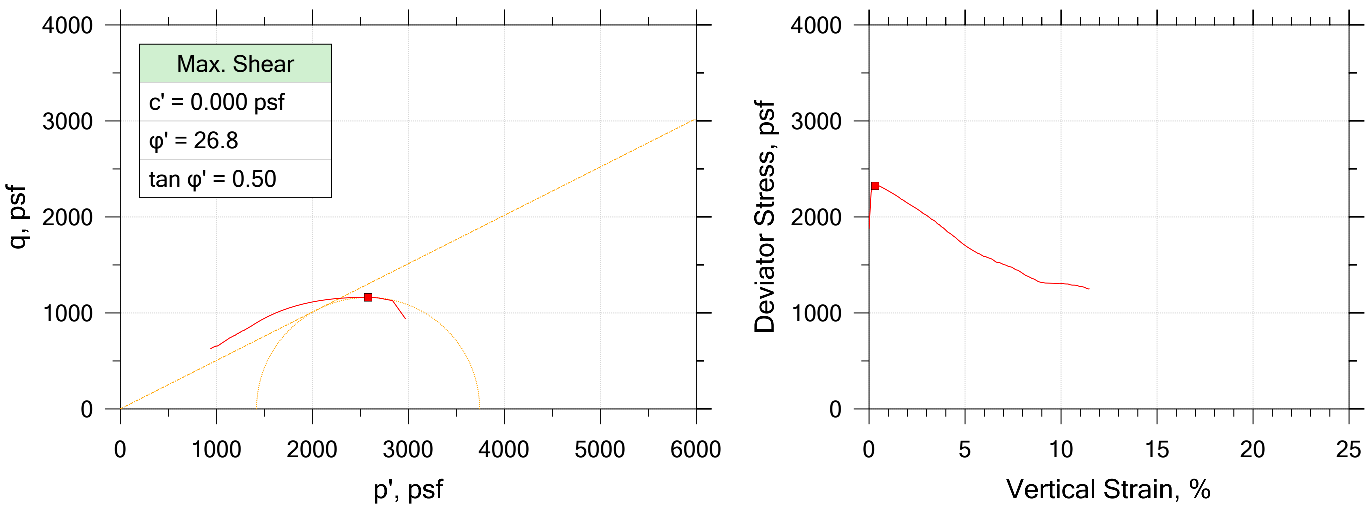
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	1U	CkoU 304	31.43	SJR	10/5/19	sjr	10/21/19	CKoU 304.dat

Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
Sample No.: 1U	Test Date: 10/5/19	Depth: 31.43
Test No.: CkoU 304	Sample Type:	Elevation:
Description: Gray Silty Clay		
Remarks:		

Soil Metrics LLC

BB-FJR-104 2U

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



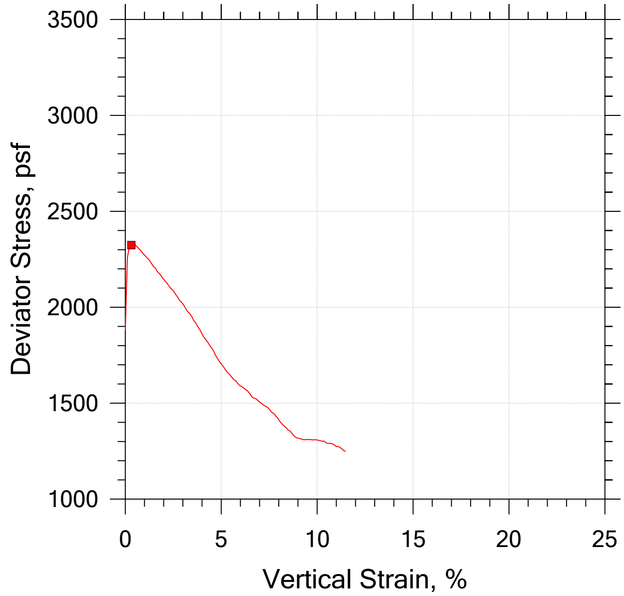
Symbol		■			
Sample ID		2U			
Depth, ft		41.75			
Test Number		CkoU 305			
Initial	Height, in	4.000			
	Diameter, in	2.000			
	Moisture Content (from Cuttings), %	44.3			
	Dry Density, pcf	78.7			
	Saturation (Wet Method), %	101.6			
	Void Ratio	1.22			
Final	Moisture Content, %	40.2			
	Dry Density, pcf	82.2			
	Cross-Sectional Area (Method A), in²	3.150			
	Saturation, %	100.0			
	Void Ratio	1.13			
	Back Pressure, %	1.102e+04			
Vertical Effective Consolidation Stress, psf		3855.			
Horizontal Effective Consolidation Stress, psf		2034.			
Vertical Strain after Consolidation, %		4.621			
Volumetric Strain after Consolidation, %		4.547			
Time to 50% Consolidation, min		0.0000			
Shear Strength, psf		1162.			
Strain at Failure, %		0.315			
Strain Rate, %/min		0.01500			
Deviator Stress at Failure, psf		2325.			
Effective Minor Principal Stress at Failure, psf		1419.			
Effective Major Principal Stress at Failure, psf		3744.			
B-Value		0.95			

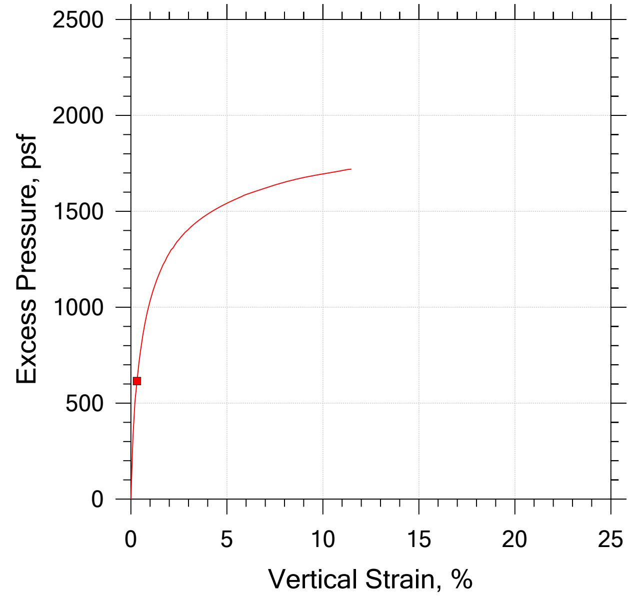
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Atterberg Limits determined by ASTM D4318.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

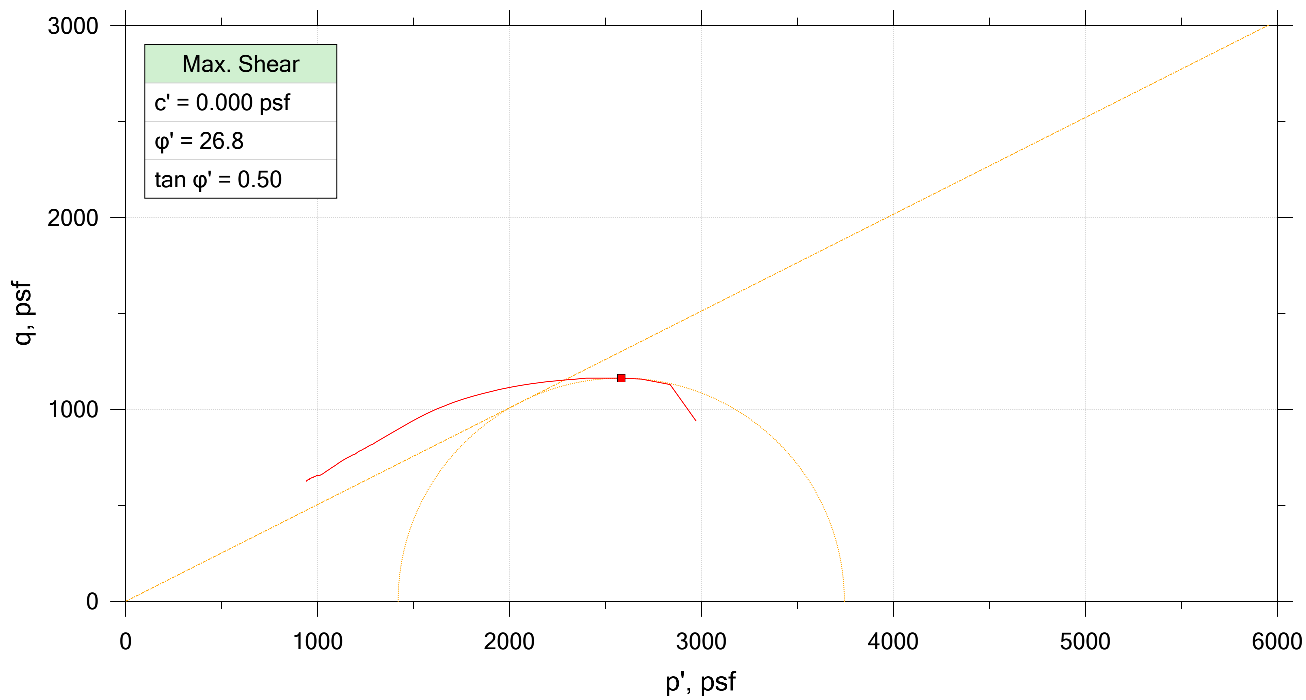
Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
Sample No.: 2U	Test Date: 10/5/19	Depth: 41.75
Test No.: CkoU 305	Sample Type:	Elevation:
Description:		
Remarks:		

Soil Metrics LLC

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



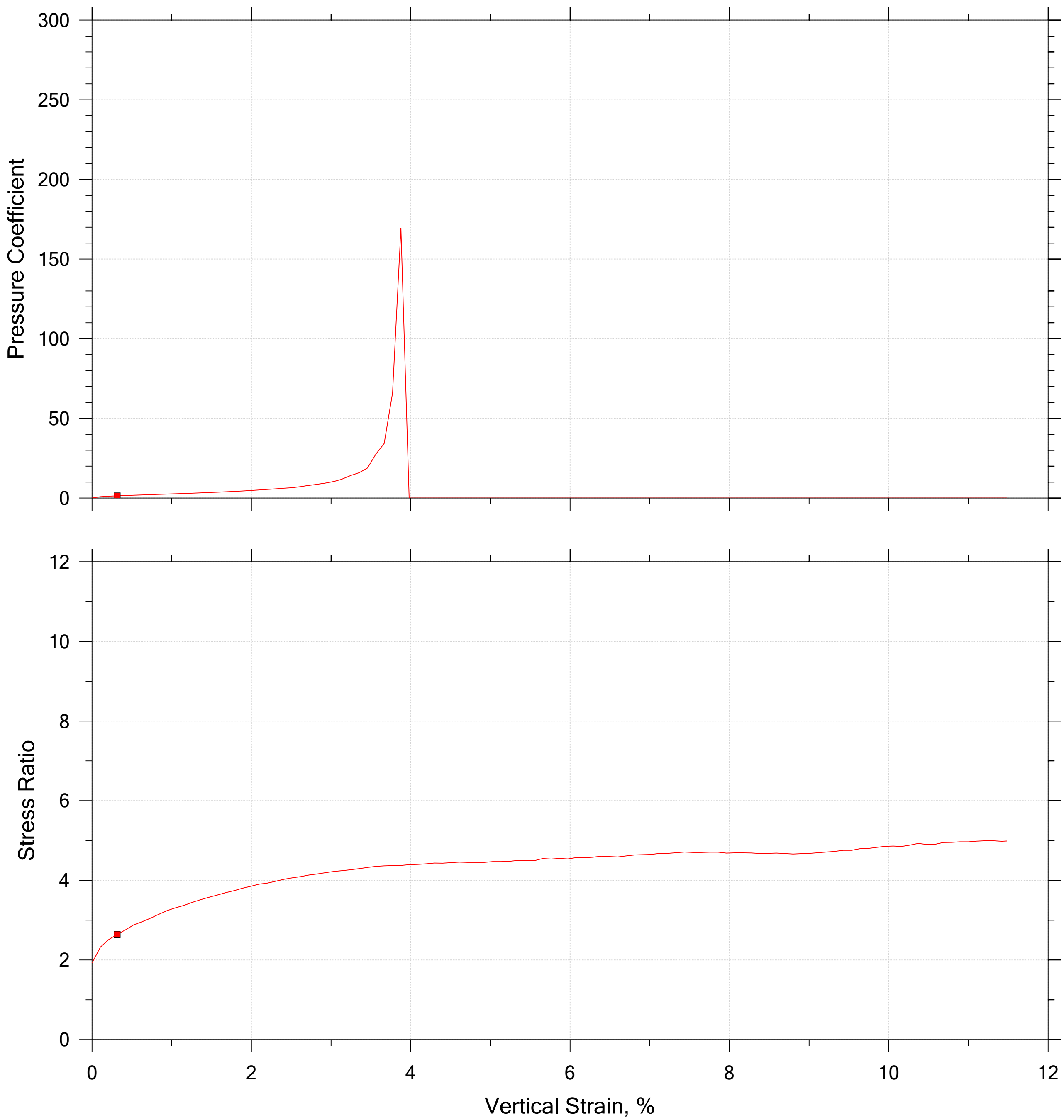




	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	2U	CkoU 305	41.75	SJR	10/5/19	sjr		CKoU 305.dat

Soil Metrics LLC	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
	Sample No.: 2U	Test Date: 10/5/19	Depth: 41.75
	Test No.: CkoU 305	Sample Type:	Elevation:
	Description:		
	Remarks:		

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	2U	CkoU 305	41.75	SJR	10/5/19	sjr		CKoU 305.dat

Soil Metrics LLC			
	Project: Johnson Road Bridge	Location: Falmouth, Maine	Project No.: 166-11
	Boring No.: BB FJR 104	Tested By: SJR	Checked By: sjr
	Sample No.: 2U	Test Date: 10/5/19	Depth: 41.75
	Test No.: CkoU 305	Sample Type:	Elevation:
	Description:		
	Remarks:		

APPENDIX E – ENGINEERING CALCULATIONS

Settlement

GZA GeoEnvironmental, Inc
707 Sable Oaks Drive, Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099
http://www.gza.com

Engineers and Scientists

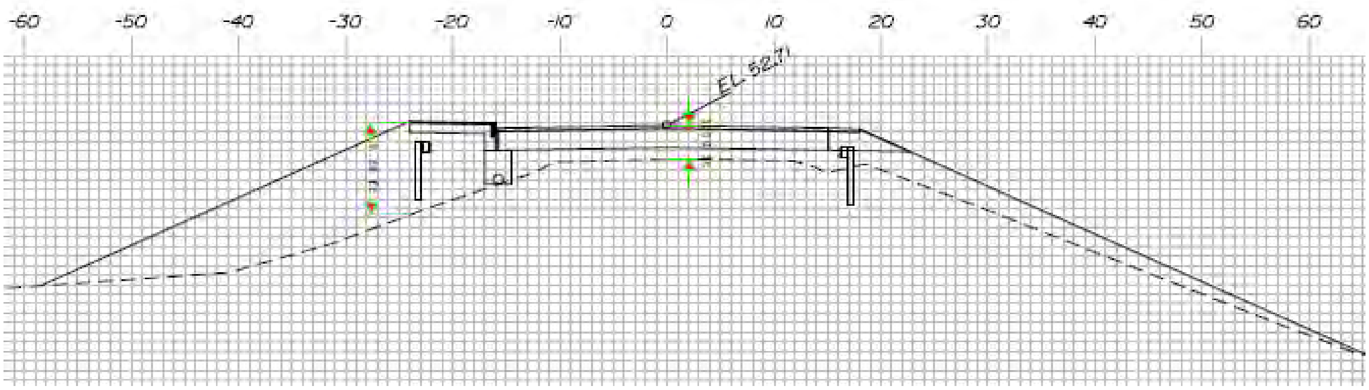
JOB: 09.0026024.00 Johnson Road Bridge
SUBJECT: Settlement
SHEET: 1 OF 18
CALCULATED BY B. Cardali, 11/11/21
REVIEWED BY A. Blaisdell, 2/22/22

Objective

Evaluate the Se: lement under the proposed grade raises and alignment configura< on. GZA will evaluate se: lement at 5- and 10 years post construc<on based on the se: lement criteria that we understand has been adopted by MaineDOT, which is a maximum of 2 inches of pavement se: lement within 100 feet of the abutment in the first 5 years, and an addi<onal 2 inches in the next 5 years (total 4 inches in the first 10 years). GZA will also evaluate Total se: lement for a total design life of 75 years understanding that shimming will be conducted typically every 15 years.

Abutment 1: Proposed fill heights in the vicinity of abutment 1 (Sta<on 4+32.6) include approximately 3.5 feet at the centerline and up to 9.5 feet in the new widened shoulder to the north, as shown in the figure below.

Abutment 2: Proposed fill heights in the vicinity of abutment 2 (Sta<on 6+73.6) include approximately 1.5 feet at the centerline and up to 6.5 feet in the new widened shoulder to the north (Not shown).



Methodology

1. Develop Se: lement Proper<es and Se: le3 Inputs.
2. Complete Se: lement analysis using Se: le3 for both no mi<ga<on at 5,10, and 75 years Post Construc<on and Lightweight fill alterna<ve. (Primary Consolida<on Se: lement Only)
3. Calculate an<cipated Secondary Se: lement
4. Evaluate Total se: lement at 5, 10, and 75 years.

1. Soil Properties

GZA developed consolida< on proper<es for the Marine Clay deposit based on laboratory and measured field data which is presented on Page 6 and summarized below. The proper<es were developed to be consistent with condi<ons outside of the exis<ng embankment footprint since our model began prior to the exis<ng embankment construc<on. Se: le3 inputs are provided on pages 15 through 62. . The 5-year and 10-year se: lement es<mates are heavily influenced by the assumed rate of ver<cal consolida<on (Cv). We evaluated the likely range of Cv values based on the consolida<on data. The data indicated a range of Cv of about 0.01-0.15 G2/day. During preliminary evalua<ons the faster consolida<on rate of 0.15 did not meet the design requirements, therefore GZA u<lized the 0.15 for our evalua<ons.

Layers For Settlement Model	Soil Properties					
	CR	RR	Over Consolidated Margin (ksf)	Cv (ft^2/day)	Unit Weight (pcf)	Cα
Upper Marine Sand	--	--	--	--	125	--
Marine Clay Crust	0.24	0.02	1.8	0.15	116	0.002
Upper Marine Clay	0.24	0.02	1.0	0.15	114	0.007
Lower Marine Clay	0.24	0.02	0.5	0.15	114	0.007
Lower Marine Sand	--	--	--	--	125	--

2. Settle3 Settlement Analysis (Primary Consolidation)

The Se: le3 model Staging for the project is:

Year 0: Original Embankment Construc<on mid to late 1950s
Year 63 (757 months): New Fills in place (2022),
Year 63.5 (758 months): final pavement (6 months aGer fill placement)
Year 68.5 (824 months): 5 years Post Construc<on
Year 73.5 (884 Months) 10 years Post Construc<on
Year 138.5 (1664 Months): 75 years Post Construc<on

GZA has iden<fied inconsistencies with Se: le3's calcula<on of secondary compression with a staged filling scenario. We have therefore u<lized a separate calcula<on to es<mate secondary compression.

Existing Embankment:

Considering GZA's model begins prior to the exis<ng embankment, GZA es<mated se: lement from the 1950s to now and compared to boring data to check model. The model es<mates that approximately 16 to 23 inches of consolida<on se: lement would have occurred at the Abutment 1 approach. GZA assumes regular pavement shimming between 1950s and 2021 would have occurred and the pavement thickness at the <me of the borings was 18 to 24 inches. Historic plans indicate 3-inches of pavement was used, resul<ng in approximately 15 to 21 inches of se: lement under the exis<ng embankment. Therefore, GZA's model is assumed to reasonably es<mate future se: lements.

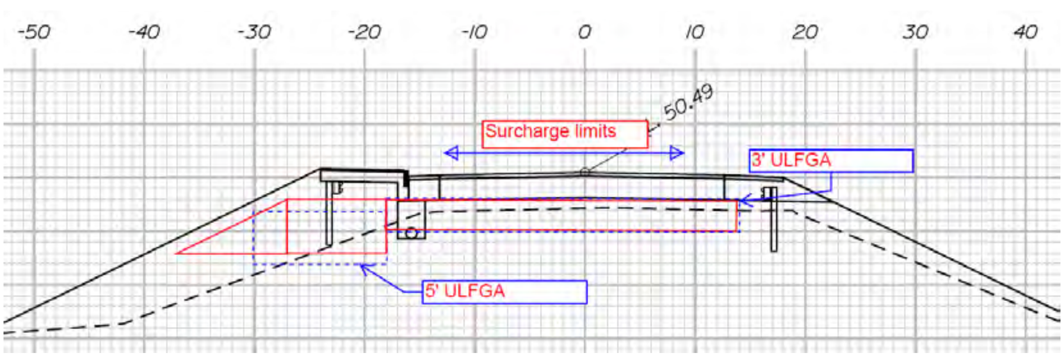
Abutment 1 (New Fill):

No Mitigation

Se: lement evalua<ons were conducted on the proposed embankment without se: lement mi<ga<on alterna<ves to es<mate the primary consolida<on se: lement under the proposed fills at the periods indicated in the se: lement criteria. The es<mated maximum primary consolida<on se: lement of approximately 3.0 inches in 5 years was calculated at Sta<on 4+16. Therefore, se: lement mi<ga<on is required to meet the se: lement criteria. The results are tabulated in the table on page 5 of this design package and on the Se: le3 plot on page 7.

Ultra-Lightweight Foamed Glass Aggregate

Se: lement evalua<ons were conducted for the preferred mi<ga<on alterna<ve of Ultra Lightweight Foamed Glass Aggregate (ULFGA). The ULFGA has an approximate in-place unit weight of 20 pcf. The limits extend from the back of the abutment to Sta<on 3+32 to reduce se: lement within 100 feet of the bridge. The sec<on shown below presents the proposed typical configura<on of the ULFGA including 3 feet within the roadway limits and a 5 feet in the shoulder, the blue line indicates the input into the model and the red lines present an equal volume of ULFGA for the proposed configura<on. Maximum primary consolida<on se: lement under the proposed sec<on was es<mated to be 1 inch in 5 years with an addi<onal 0.5 inches in the following 5 years at sta<on 4+16.



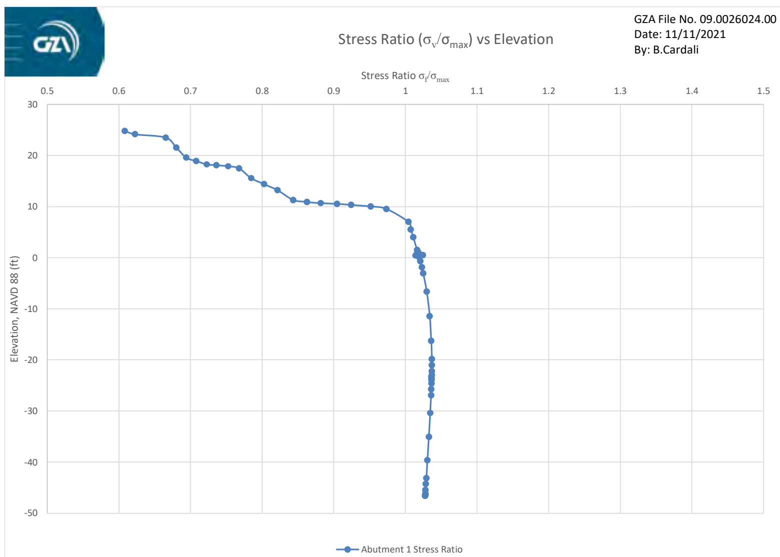
Abutment 2:

No Mitigation

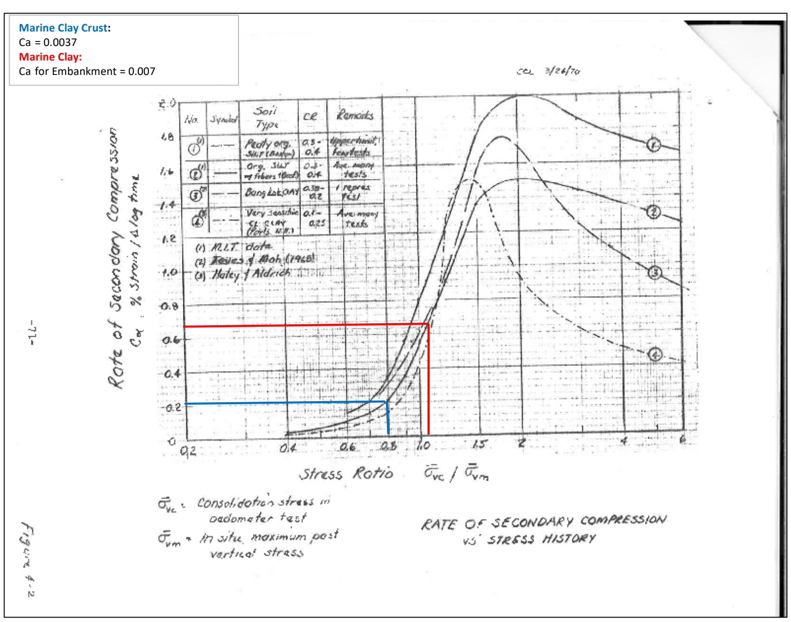
Considering the high shear strength and thinner clay profile encountered near Abutment 2. GZA es<mated that minimal se: lement will occur within 100 feet of the bridge. Therefore, se: lement mi<ga<on evalua<ons were not considered at the Abutment 2 approach.

3. Secondary Compression

The Stress ra<o is calculated by dividing the final effec<ve stress by the maximum previous effec<ve stress. The profile at Sta<on 4+00 was considered and plo: ed in the following graph.



From the plot the average stress ra<os were determined for the clay crust to be approximately 0.8 between Eleva<on 24 and 7. The underlying marine clay resulted in an average stress ra<o of approximately 1.05. U<lizing the stress ra<o vs Cα from the plot below, Cα was es<mated to be 0.002 for the Clay Crust and 0.007 for the Marine Clay.



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Engineers and Scientists

JOB: 09.0026024.00 Johnson Road Bridge
SUBJECT: Settlement
SHEET: 4 OF 18
CALCULATED BY B. Cardali, 11/11/21
REVIEWED BY A. Blaisdell, 2/22/22

Time to end of primary consolida<on was calculated for the deepest soil profile near Abutment 1. Under the proposed load, the <me to achieve an average degree of consolida<on of 90% is approximately 1 year for the Marine Clay Crust and 4.5 years for the Marine Clay. Secondary compression was then evaluated for the 5, 10, and the 75 year design life. The es<mated secondary compression in the vicinity of Abutment 1 is presented below:

Secondary Settlement									
Layer	Layer Thickness (H_o) (ft)	H_{dr} (ft)	t_{90} (days)	Number of Log cycles at 5 yr	Number of Log cycles at 10 yr	Number of Log cycles at 75 yr	S_s per layer 5 yr	S_s per layer 10 yr	S_s per layer 75 yr
			$t_{90} = \frac{T_{90} * Hdr^2}{c_v}$	$\# \text{Cycles} = \log \frac{t}{t_{90}}$			$S_s = H_o * c_\alpha * \log \frac{t}{t_{90}}$		
Marine Clay Crust	8	8	362	0.7	1.0	1.9	0.2	0.2	0.4
Marine Clay*	34	17	1634	0.0	0.3	1.2	0.1	1.0	3.5
* indicates Layer is double drained						Total	0.3	1.2	3.9

4. Results

The Settlement with ULFGA vs. No Mitigation Summary Table below shows the es<mated primary consolida<on and secondary compression in 5, 10, and 75 years with no mi<ga<on and using the ULFGA configura<on shown in sec<on 2 at various sta<ons along the alignment.

Settlement Results						
Settlement Mitigation Alternative			Maximum Settlement at Station within Roadway Limits (inches)			
			3+00	3+50	4+00	4+16 (Approach Slab)
No Mitigation	5 Year Post Construction	Primary	1	2	2.5	3
		Secondary	0.3	0.3	0.3	0.3
		Total	1.3	2.3	2.8	3.3
No Mitigation	10 Year Post Construction	Primary	1	2.5	3	3.5
		Secondary	1.2	1.2	1.2	1.2
		Total	2.2	3.7	4.2	4.7
		Total (5 - 10 yr)*	0.9	1.4	1.4	1.4
Ultra Lightweight Foamed Glass Aggregate	5 Year Post Construction	Primary	0.5	0.5	1	1
		Secondary	0.3	0.3	0.3	0.3
		Total	0.8	0.8	1.3	1.3
Ultra Lightweight Foamed Glass Aggregate	10 Year Post Construction	Primary	1	1	1	1.5
		Secondary	1.2	1.2	1.2	1.2
		Total	2.2	2.2	2.2	2.7
		Total (5 - 10 yr)*	1.4	1.4	0.9	1.4
Ultra Lightweight Foamed Glass Aggregate	75 Year Post Construction	Primary	1	1.5	1.5	1.5
		Secondary	3.9	3.9	3.9	3.9
		Total	4.9	5.4	5.4	5.4
		Total (10 - 75 yr)*	3.5	4.0	4.5	4.0

Notes:
1. Primary Consolidation Settlement from Settle3 rounded to the nearest 0.5 inches.
2. Settlement criteria is less than 2 inches in the first five years, then another 2-inches in the following 5 years
3. "*" indicates the total settlement to date less the previous period total settlement. Use this total to compare to criteria identified in note 2.
4. 75 Year settlement results are included to show additional anticipated settlement within the remainder of the design life, It is anticpated that pavement shimming will be conducted in 15 year intervals to maintain long term settlements
5. Highlighted cell indicates settlment values exceed criteria within period.

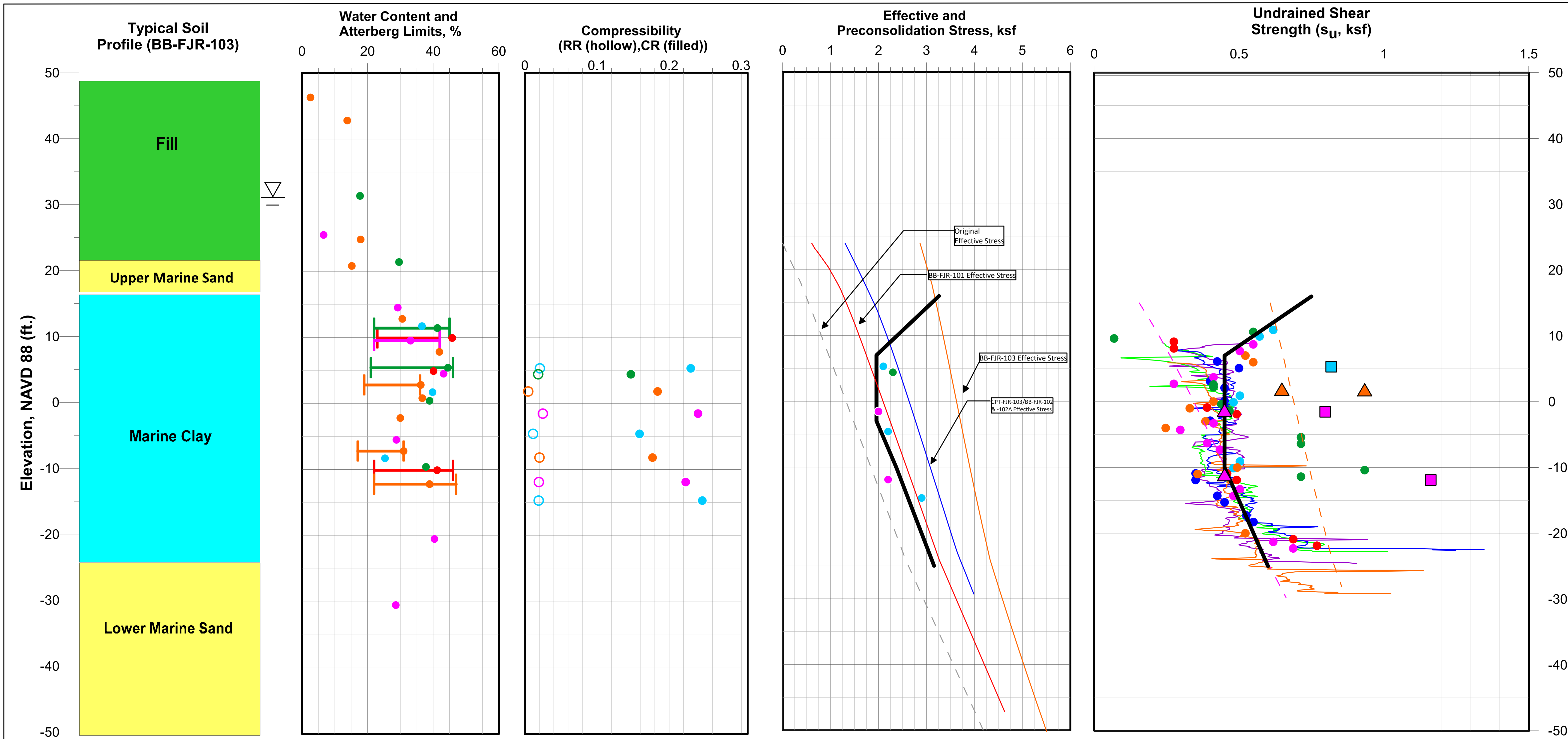
5. Conclusions

The No Mitigation results indicate that predicted settlements exceed the project criteria within the first five year period, therefore, settlement mitigation is required. The lightweight fill scheme was considered to be appropriate. The results show that the configuration of Ultra light weight foamed glass aggregate shown on typical sections herein provide settlement mitigation to achieve the project settlement criteria of less than 2 inches in the first 5 years and another 2 inches between 5 and 10 years within 100 feet of the abutment, with predictions of approximately 0.5- to 1-inch in each of the criteria time periods. Additionally, we predict greater than 0.5 inches of consolidation at the bridge abutments during the design life of the bridge, therefore downdrag needs to be considered in pile design.

6. Attachments

The a: achments are described as follows:

- **GZA PLOT Water Content, Compressibility, Stress History and Strength of Cohesive Soils (page 6)**
- **Settle3 Plots (page 7-13)**
  - **Consolidation Settlement Existing Embankment (1957 to present)**
  - **Consolidation Settlement No Mitigation at 5 Yr. Post-Construction**
  - **Consolidation Settlement No Mitigation at 10 Yr. Post-Construction**
  - **Consolidation Settlement No Mitigation at 75 Yr. Post-Construction**
  - **Consolidation Settlement with ULFGA at 5 Yr. Post-Construction**
  - **Consolidation Settlement with ULFGA at 10 Yr. Post-Construction**
  - **Consolidation Settlement with ULFGA at 75 Yr. Post-Construction**
- **Settle3 Input Report (page 14 -18)**



LEGEND

Plastic Limit, PL — Water Content, Wn — Liquid Limit, LL

BB-FJR-101 (FV)	BB-FJR-104 (Su/P=.21)
BB-FJR-102 (FV,Consol)	BB-FJR-103 (Su/P=.21)
BB-FJR-102A (FV,Consol)	DESIGN Su PROFILE
BB-FJR-103 (FV, Consol)	CPT-FJR-103 (Nkt 22 NDu 15)
BB-FJR-104 (FV, Consol)	CPT-FJR-104 (Nkt 18 NDu 13)
BB-FJR-105 (FV, Consol)	CKoUC - BB-FJR-102A
CPT-FJR-101 (Nkt 22 NDu 15)	CKoUC - BB-FJR-104
CPT-FJR-102 (Nkt 22 NDu 15)	DSS (BB-FJR-104)
DSS (BB-FJR-103)	

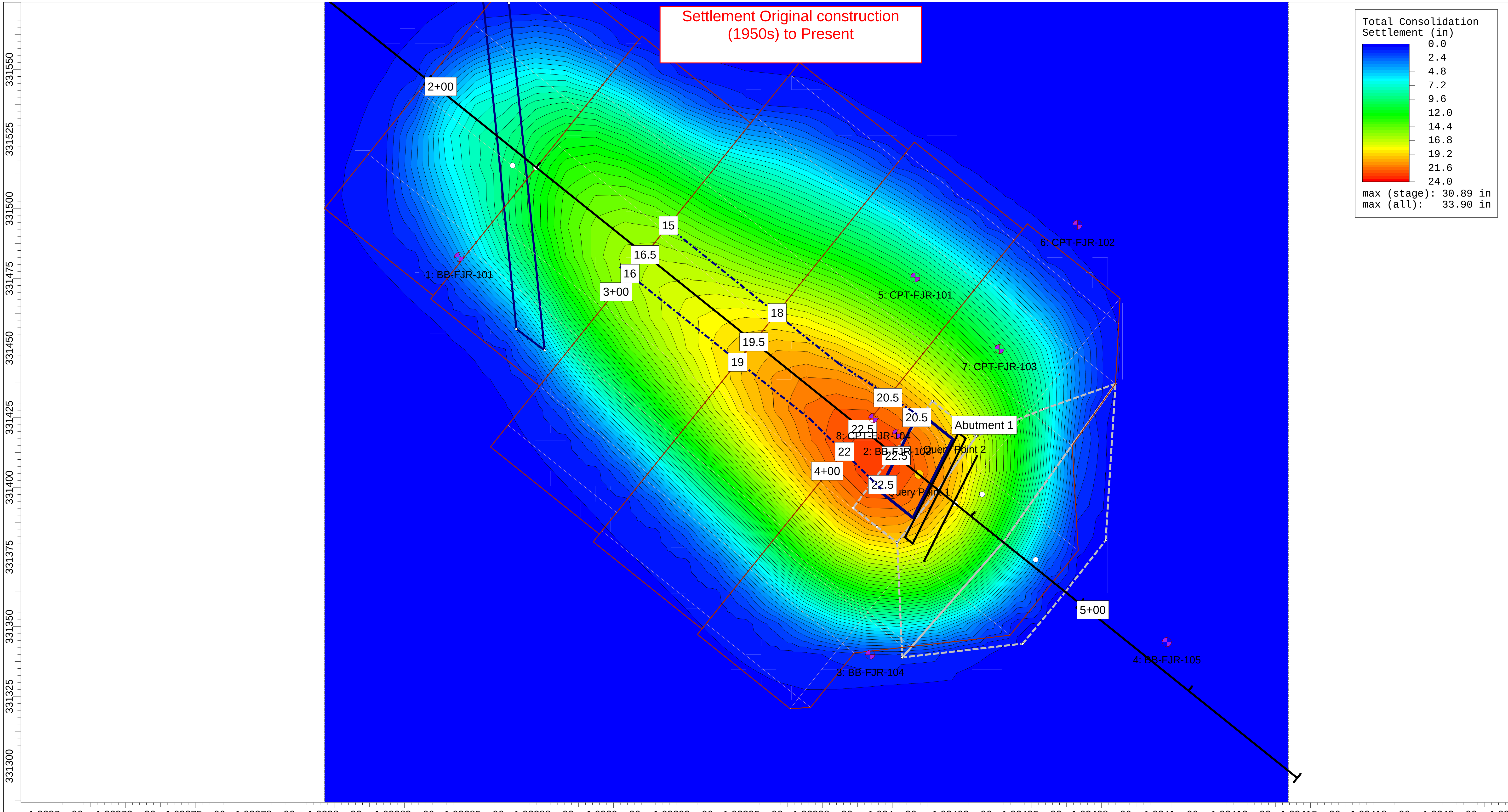
NOTES:

1. DATA BASED ON TEST BORINGS (BB-FJR-101, -102A -103, -104, -105 AND -106) PERFORMED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE BETWEEN JUNE 3 AND AUGUST 21, 2019 AND TEST BORING BB-FJR-101 AND -107 PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE BETWEEN MAY 28 AND MAY 30, 2019. BORINGS PERFORMED BY NEW ENGLAND BORING CONTRACOTRS WERE OBSERVED AND LOGGED BY GZA PERSONNEL.
2. CPT EXPLORATIONS (CPT-FJR-101, -102, -103, -104 AND -105) PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE BETWEEN MAY 28 AND 30, 2019.
3. TYPICAL SOIL PROFILE BASED ON BORING BB-FJR-103.
4. WATER CONTENTS BASED ON LABORATORY TESTS PERFORMED ON SAMPLES TAKEN FROM RECENT BORINGS.
5. EFFECTIVE STRESS BASED ON INITIAL EFFECTIVE STRESS CALCULATED BY SETTLE3D BY ROCSCIENCE. EXISTING EMBANKMENT (FILL) MODELED AS AN EMBANKMENT LOAD OVER ORIGINAL GRADE. THEREFORE, EFFECTIVE STRESS ONLY CALCULATED BELOW EL. 18'.
6. PRECONSOLIDATION PRESSURE CALCULATED FROM CONSOLIDATION TESTS USING THE WORK METHOD.
7. CORRELATED UNDRAINED SHEAR STRENGTH FROM CPT DATA IS BASED ON NDu=15 FROM EL. 15 TO -5 AND N_{kt}=22 BELOW (CPT-FJR-101, -102, AND -103) and NDu=13 FROM EL. 15 TO -5 AND N_{kt}=18 BELOW (CPT-FJR-104).
8. IN LEGEND, FV=UNDRAINED SHEAR STRENGTH FROM IN-SITU FIELD VANE, CONSOL=LAB DATA FROM CONSOLIDATION TEST.

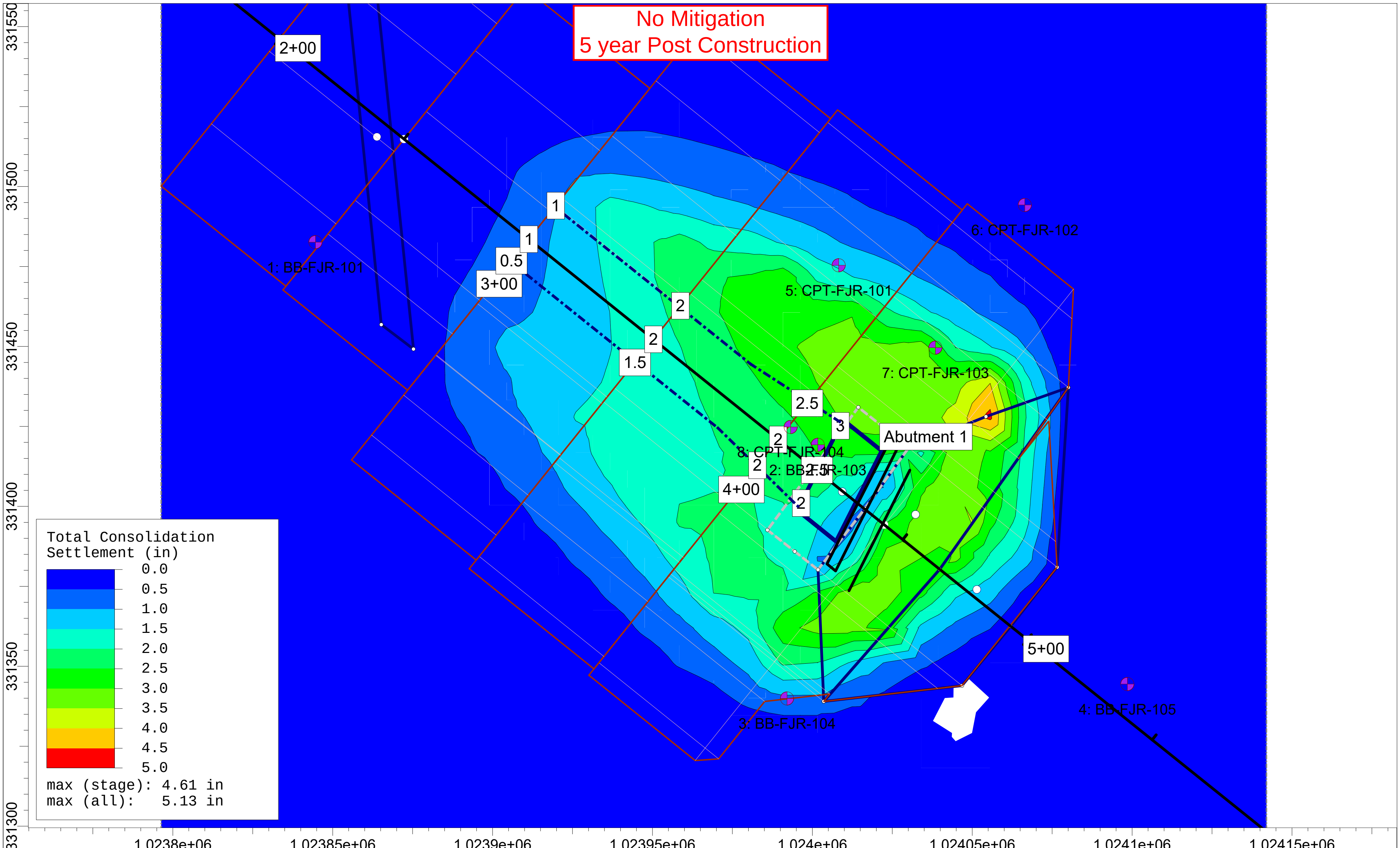
JOHNSON ROAD BRIDGE REPLACEMENT
FALMOUTH, ME

IN-SITU SOIL CONDITIONS VS. ELEVATION
Abutment 1 Approach

PREPARED BY:	PREPARED FOR:		
GZA GeoEnvironmental, Inc. Engineers & Scientists www.gza.com	MAINE DEPARTMENT OF TRANSPORTATION		
PROJ MGR: BMC	REVIEWED BY: ARB	CHECKED BY: --	FIGURE 3
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: N/A	
DATE: 11/15/2021	PROJECT NUMBER: 09.0026024.00	REVISION NUMBER: 0	

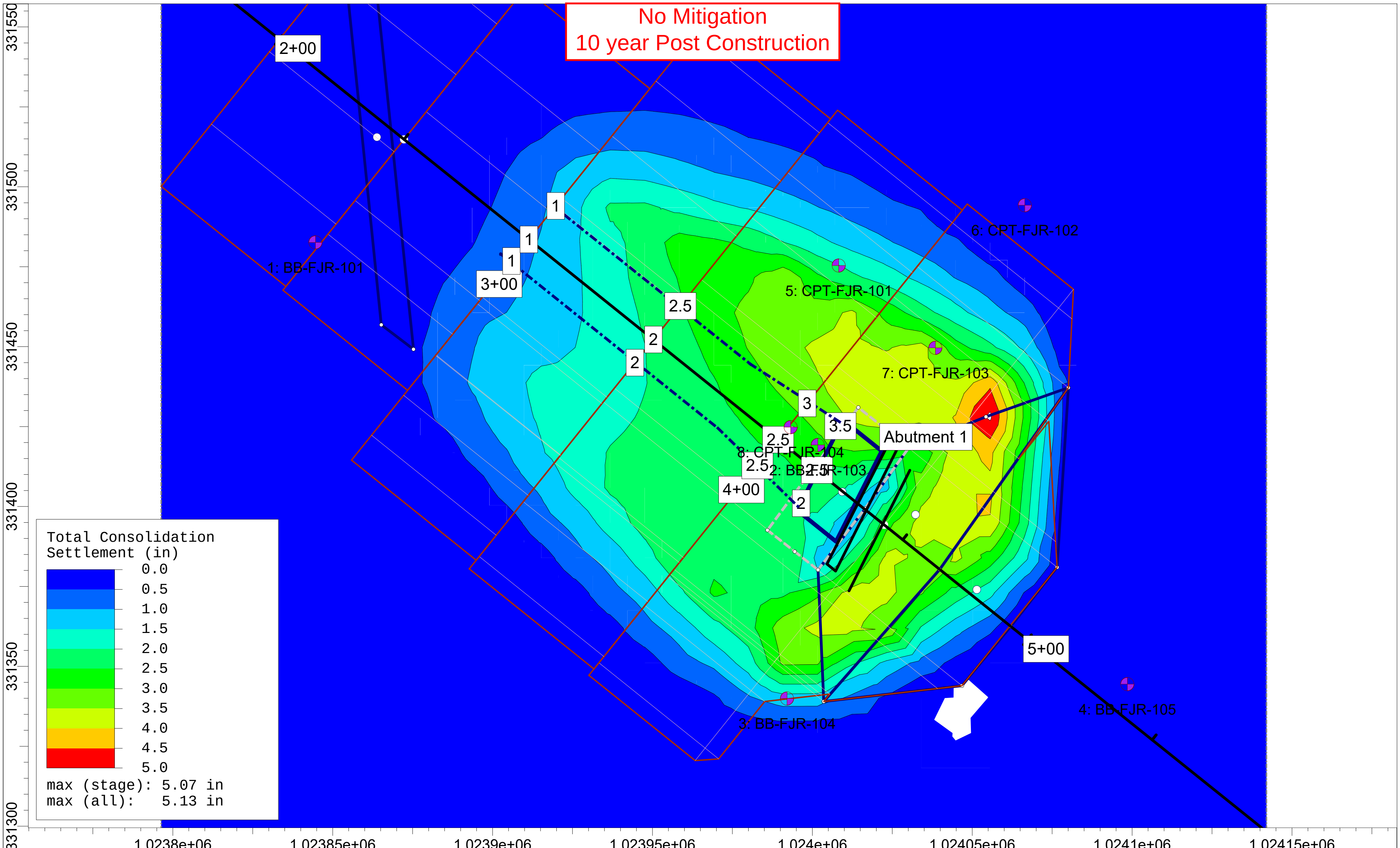


SETTLE3 5.013	*Project* Johnson Road		
	Analysis Description		
	Drawn By B. Cardali	*Company* GZA GeoEnvironmental, Inc.	
	Date 10/23/2019, 8:14:06 AM	*File Name* Johnson Road 11 11 21 Cv .15 Double Drained No mitigation.s3z	



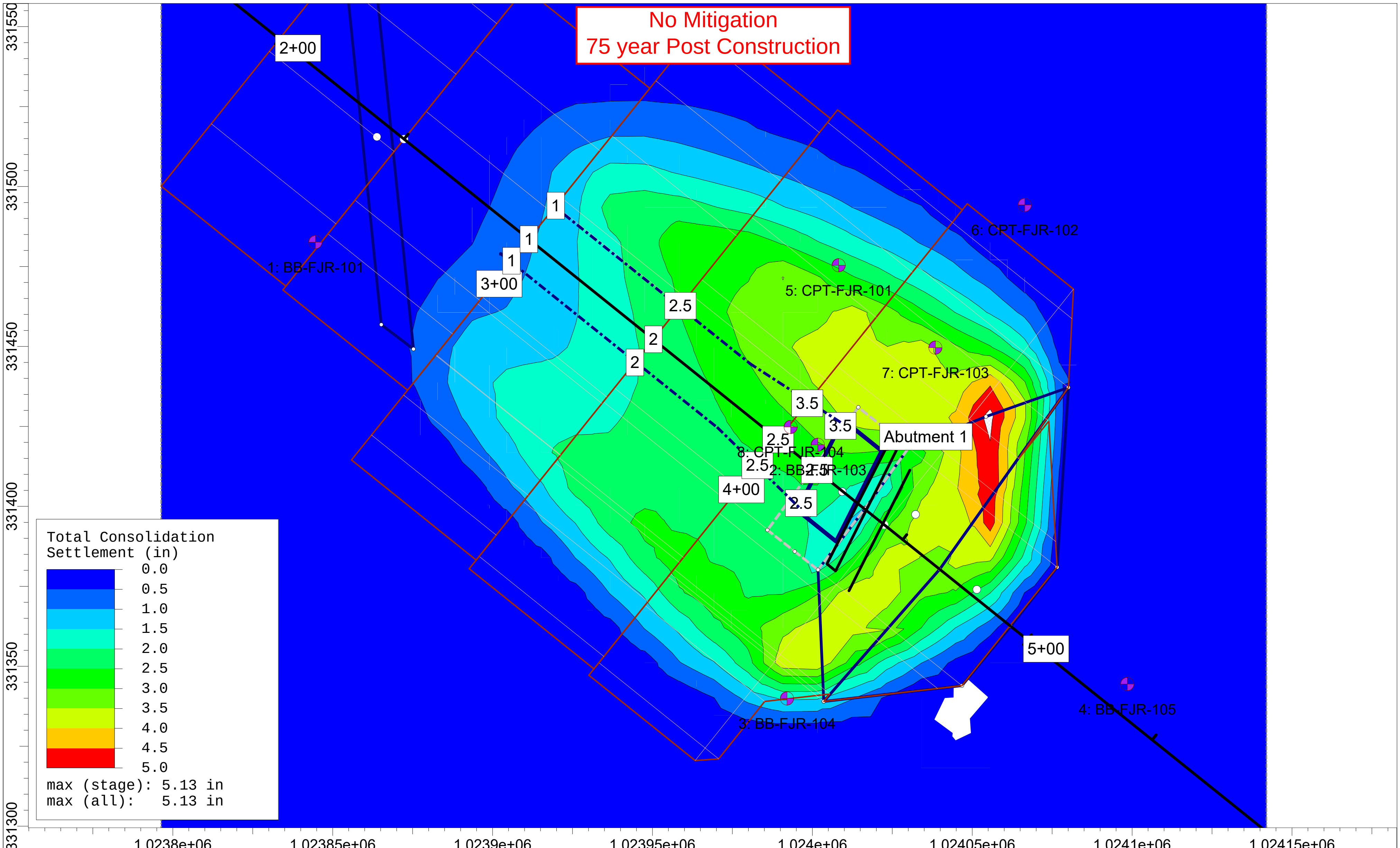
Project	Johnson Road Bridge
Analysis Description	No mitigation
Drawn By B. Cardali	*Company* GZA GeoEnvironmental, Inc.
Date 10/23/2019, 8:14:06 AM	*File Name* Johnson Road 11 11 21 Cv .15 Double Drained No mitigation.s3z

SETTLE3 5.012



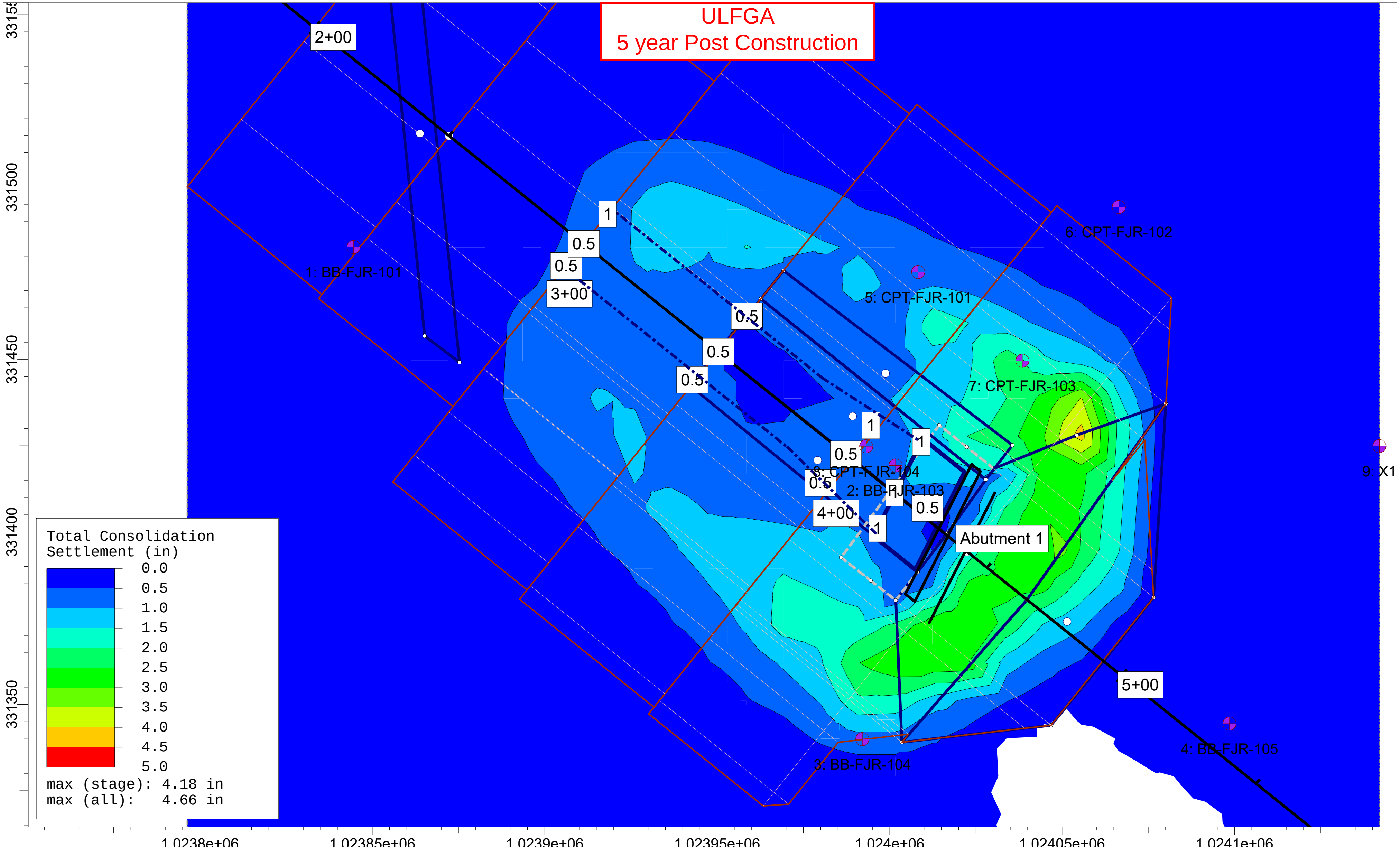
Project	Johnson Road Bridge		
Analysis Description	No mitigation		
Drawn By	B. Cardali	*Company*	GZA GeoEnvironmental, Inc.
Date	10/23/2019, 8:14:06 AM	*File Name*	Johnson Road 11 11 21 Cv .15 Double Drained No mitigation.s3z

SETTLE3 5.012



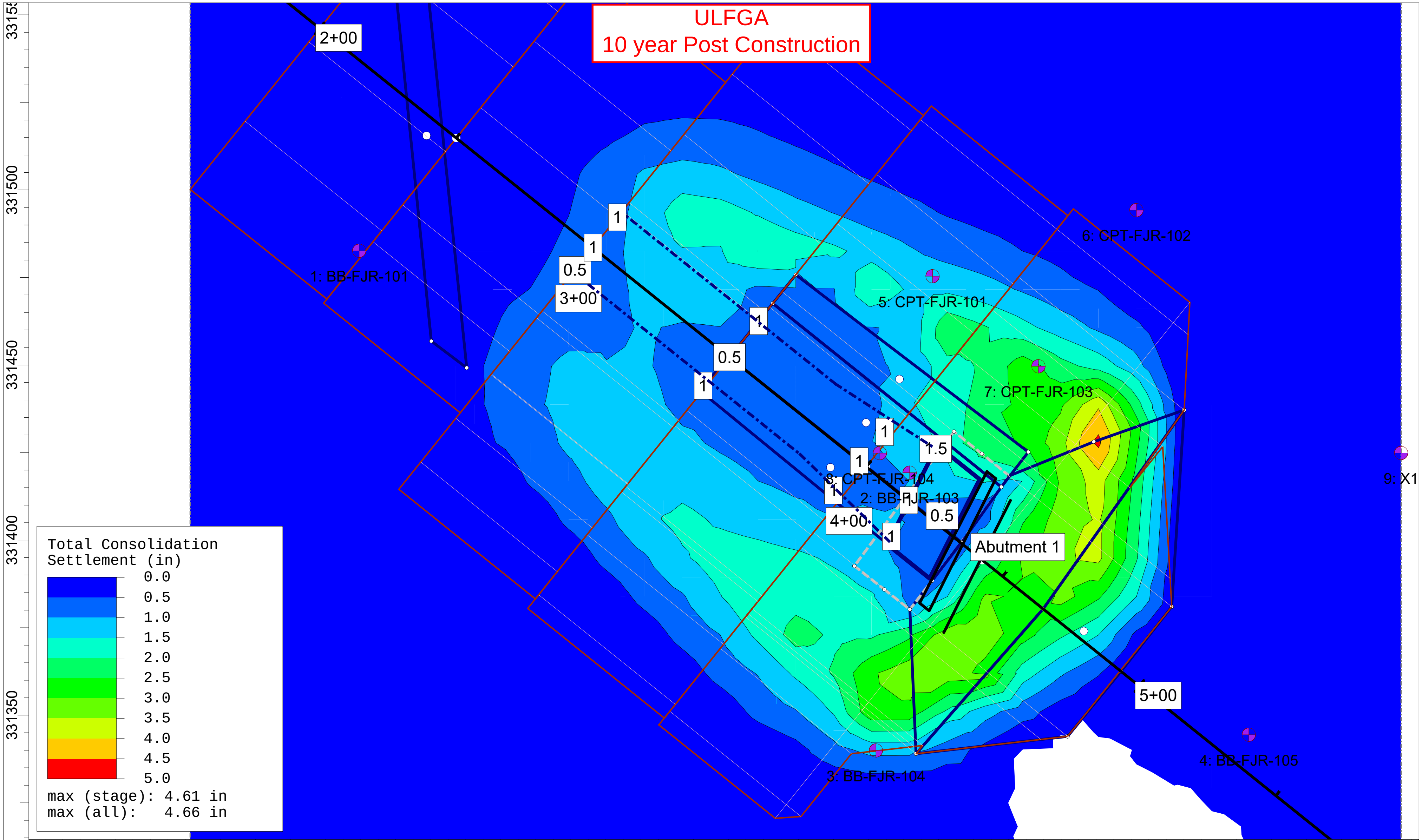
Project	Johnson Road Bridge
Analysis Description	No mitigation
Drawn By B. Cardali	*Company* GZA GeoEnvironmental, Inc.
Date 10/23/2019, 8:14:06 AM	*File Name* Johnson Road 11 11 21 Cv .15 Double Drained No mitigation.s3z

SETTLE3 5.012



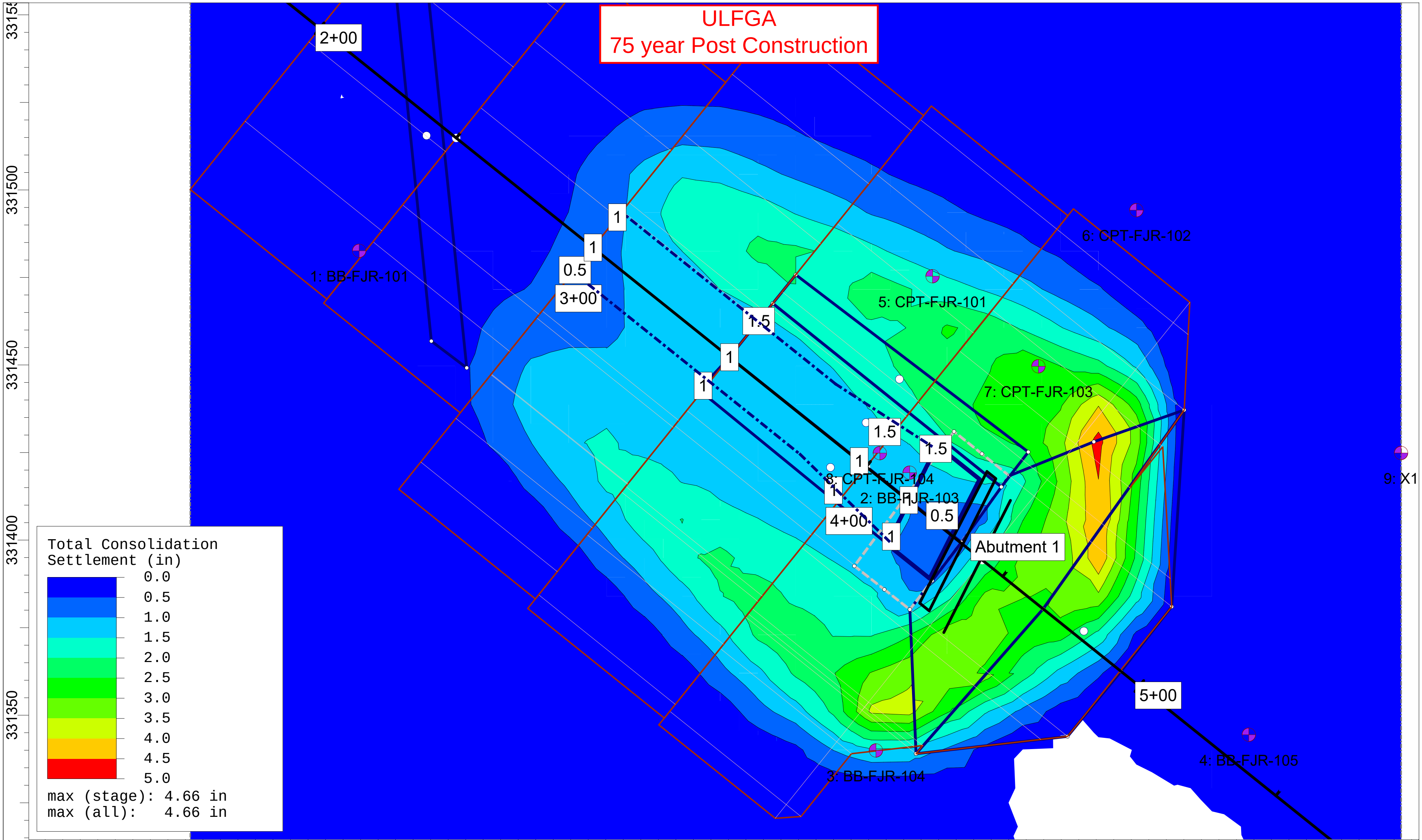
Project	Johnson Road Bridge		
Analysis Description	ULFGA ALTERNATIVE		
Drawn By	B. Cardali	*Company*	GZA GeoEnvironmental, Inc.
Date	10/23/2019, 8:14:06 AM	*File Name*	Johnson Road 10 4 21 Cv .15 Double Drained LWF.s3z

SETTLE3 5.012



Project	Johnson Road Bridge
Analysis Description	ULFGA ALTERNATIVE
Drawn By	B. Cardali
Company	GZA GeoEnvironmental, Inc.
Date	10/23/2019, 8:14:06 AM
File Name	Johnson Road 10 4 21 Cv .15 Double Drained LWF.s3z

SETTLE3 5.012



Project	Johnson Road Bridge		
Analysis Description	ULFGA ALTERNATIVE		
Drawn By	B. Cardali	*Company*	GZA GeoEnvironmental, Inc.
Date	10/23/2019, 8:14:06 AM	*File Name*	Johnson Road 10 4 21 Cv .15 Double Drained LWF.s3z

SETTLE3 5.012

Johnson Road
GZA GeoEnvironmental, Inc.
Report Creation Date: 2021/11/15, 12:25:34

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Soil Properties 5

Settle3 Analysis Information

Johnson Road

Project Settings

Document Name	Johnson Road 10 4 21 Cv .15 Double Drained LWF.s3z
Project Title	Johnson Road
Author	B. Cardali
Company	GZA GeoEnvironmental, Inc.
Date Created	10/23/2019, 8:14:06 AM
Stress Computation Method	Boussinesq
Time-dependent Consolidation Analysis	
Time Units	months
Permeability Units	feet/day
Minimum settlement ratio for subgrade modulus	0.9
Poisson ratio for Boussinesq stress computation	0.3
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name	Time [months]
1	Stage 1	0
2	Stage 2	1
3	Stage 3	757
4	Stage 4	758
5	Stage 5	764
6	Stage 6 - 2 yr	788
7	Stage 7 - 5 yr	824
8	Stage 8 - 10 yr	884
9	Stage 9 - 75 yr	1664
10	Stage 9 - 150 yr	2564

Soil Properties

Property		Upper Marine Sand	Marine Clay Crust	Upper Marine Clay	Lower Marine Sand
Color					
Unit Weight [kips/ft3]		0.125	0.116	0.114	0.125
Saturated Unit Weight [kips/ft3]		0.125	0.116	0.114	0.125
K0		1	1	1	1
Primary Consolidation		Disabled	Enabled	Enabled	Disabled
Material Type			Non-Linear	Non-Linear	
Cce		-	0.24	0.24	-
Cre		-	0.02	0.02	-
e0		-	1.1	1.1	-
OCM [ksf]	top	-	1.8	1	-
	bottom	-	1	0.5	-
Cv [ft2/d]		-	0.15	0.15	-
Cvr [ft2/d]		-	0.15	0.15	-
B-bar		-	1	1	-
Secondary Consolidation		Disabled	Standard	Standard	Disabled
Cae		-	0.008	0.008	-
Care		-	0.004	0.004	-
Undrained Su A [kips/ft2]		0	0	0	0
Undrained Su S		0.2	0.2	0.2	0.2
Undrained Su m		0.8	0.8	0.8	0.8
Piezo Line ID		1	1	1	1

Property	Lower Marine Clay
Color	
Unit Weight [kips/ft3]	0.114
Saturated Unit Weight [kips/ft3]	0.114
K0	1
Primary Consolidation	Enabled
Material Type	Non-Linear
Cce	0.24
Cre	0.02
e0	1.1
OCM [ksf]	0.5
Cv [ft2/d]	0.15
Cvr [ft2/d]	0.15
B-bar	1
Secondary Consolidation	Standard
Cae	0.008
Care	0.004
Undrained Su A [kips/ft2]	0
Undrained Su S	0.2
Undrained Su m	0.8
Piezo Line ID	1

Global Stability

Objective

Evaluate global stability of the proposed approach embankments at cri<cal cross sec<ons near the proposed abutments and in the longitudinal direc<on beneath each abutment using the profile and proper<es shown in the Interpre<ve Subsurface Profile and Design Profile and Cross Sec<ons for the Johnson Road Bridge replacement.

References

1. American Associa<on of State Highway and Transporta<on Officials, AASHTO LRFD Bridge Design Specifica<ons: Customary U.S. Units, 9th edi<on, 2020. (AASHTO LRFD)
2. Hynes-Griffin and Franklin (1984), "Ra<onalizing the seismic coefficient method," Miscellaneous Paper GL-84-13, U.S. Army Corps of Engineers Waterway Experiment Sta<on, Vicksburg, Mississippi.
3. Kramer (1996), "Geotechnical Earthquake Engineering," Pren<ce-Hall, Inc., Upper Saddle River, NY.

Analysis

Soil Proper<es

Fric<on angles were developed for granular materials based on corrected N60 blow counts of exis<ng materials or an<cipated/MaineDOT BDG design proper<es of new fills. Shear strengths and unit weights of marine clay were developed based on laboratory tes<ng, field vane tes<ng, and CPT correla<ons, and are shown on Figure 3 of the Geotechnical Report. Design soil proper<es are shown on the Slope/W output.

Stability Calcula<on and Performance Criteria

To consider the longitudinal and cross-sec<on analyses, the minimum factor of safety for global stability is selected as 1.5 and 1.3, respec<vely. These correspond to approximate resistance factors of 0.65 and 0.75, for a slope that supports structures and for a slope that does not contain or support a structural element, as specified in AASHTO LRFD Ar<cle 11.6.2.3.

Pseudosta<c Analysis

The minimum factor of safety for global stability is selected as 1.0, which is recommended by Hynes-Griffin and Franklin (1984), referenced in Geotechnical Earthquake Engineering (1996). The pseudosta<c analysis includes an addi<onal earthquake load by incorpora<ng the horizontal seismic coefficient, kh, as defined in AASHTO 11.6.5.2.2 and recommended by Hynes-Griffin and Franklin using the Newmark method as half of the maximum peak ground accelera<on at the ground surface (see seismic analysis). This criterion is considered suitable to limit poten<al for large deforma<on of the embankments, which is considered suitable for this Extreme Event scenario.

GZA evaluated the stability of the proposed approach embankments, using the computer analy<cal sol ware *Slope/W 2018*, developed by Geo-Slope Interna<onal, based on the Modified Bishop method.

Longitudinal Analysis

The analyzed profiles considered the interpreted typical subsurface condi<ons along the project baseline without the beneficial reinforcing effect of the proposed new HP14x89 piles. The proposed embankment at the abutments consist of the travelway with a traffic surcharge placed over the exis<ng fill with a proposed slope of 2H:1V.

The aL ached Slope-W outputs show that the proposed embankment provides the following minimum global factors of safety in the longitudinal direc<on:

Abutment	Analysis	Factor of Safety Against Rota<on	Required Factor of safety
1	Sta<c	1.8	1.5
2	Sta<c	1.6	1.5
2	Pseudosta<c	1.5	1.0

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Engineers and Scientists

Cross Sec<on Analysis

The highest embankment and/or steepest side slopes are iden<fied at the Abutment 1 approach. The two analyzed sec<ons are at sta<on 3+50 and 4+00. Considering the thinner soil profiles and higher strength soils at Abutment 2, the Abutment 1 approach global stability controls, therefore cross sec<ons at the Abutment 2 approach were not evaluated. The analyzed profiles considered the typical subsurface condi<ons. Exis<ng condi<on models were evaluated at both sec<ons to consider current embankment factors of safety in the Lel to Right direc<on, primarily for comparison to finite element results. A Slope-W model was developed for the proposed 22 foot wide travelway with a traffic surcharge load placed over the fill and side slopes of 2H:1V. Type D subbase gravel was modeled beneather the TRM with a fric<on angle of 36 degrees as provided for Gravel Borrow in the BGD.

The aL ached Slope-W outputs show that the proposed embankment provides the following minimum global factors of safety:

Sta<on	Analysis	Factor of Safety Against Rota<on	Required Factor of safety
3+50 Lel to Right	Sta<c (exis<ng)	1.1	1.3
3+50 Lel to Right	Sta<c	1.0	1.3
3+50 Right to Lel	Sta<c	1.5	1.3
4+00 Lel to Right	Sta<c (exis<ng)	1.2	1.3
4+00 Lel to Right	Sta<c	1.1	1.3
4+00 Right to Lel	Sta<c	1.2	1.3

Sheet 10 shows the poten<al for safety factors around 1.2 for near-surface circles in the TRM area with a 1.5H:1V inclina<on, but the TRM and dense gravel fill are planned to mi<gate poten<al for surficial instability. Therefore, this is acceptable. Based on the results, global stability is not suitable at Sta<on 3+50 in the lel to right direc<on (south), but is suitable for Right to lel (north). Addi<onally, global stability is not suitable at Sta<on 4+00 in either direc<ons. Therefore the three analyses were further evaluated for remedial alterna<ves to improve global stability.

Station 4+00 (Right to Left) was evaluated by adding a toe berm to the north of the slope. The toe berm that meets stability criteria can be constructed outside of the wetland area to the north and matches the exis<ng ground surface contour at approximately 130 feet lel of the centerline, then rises at a 2H:1V slope un<l reaching eleva<on 35, then remains flat un<l reaching the base of the highway embankment slope (approx. 50 feet lel). This sec<on was also analyzed for pseudosta<c analysis. The aL ached Slope-W outputs show that the proposed embankment with a Toe Berm provides the following minimum global factors of safety:

Sta<on	Analysis	Factor of Safety Against Rota<on	Required Factor of safety
4+00 Right to Lel	Sta<c	1.3	1.3
4+00 Right to Lel	Pseudosta<c	1.0	1.0

Station 3+50 and 4+00 (Left to Right) also were ini<ally considered for toe berm solu<ons, but a combina<on of an exis<ng water line loca<on and an exis<ng brook made that solu<on not feasible. Therefore, pile reinforcement alterna<ves were considered. This approach was conducted using Plaxis2D sol ware to evaluate the soil-pile interac<on. A c-phi reduc<on analysis was completed in Plaxis, in which the soil strengths are reduced at a uniform ra<o un<l failure, which is considered when large-scale deforma<on occurs in the model at constant strengths. The inverse of the strength reduc<on ra<o is considered to be the factor of safety. GZA evaluated con<nuous embedded sheet pile wall at both sta<ons. Discrete H-Piles driven at various spacing were also evaluated at Sta<on 4+00. H-Pile are not appropriate from sta<on 3+00 to 3+75 due to the poten<al for scour adjacent to the brook. The results are aL ached to this package and summarized in the table below.

Station	Structure	Embedment in sand (ft)	surcharge (psf)	spacing (ft)	FS
3+50	Existing/no structure	-	250	-	1.07
	Sheetpile SKZ-38	10	250	-	1.27
		15	250	-	1.33
4+00	Existing/no structure	-	250	-	1.18
	Sheetpile SKZ-38	10	250	-	1.26
		15	250	-	1.29
	HP14x89	10	250	5	1.21
		10	250	2.5	1.26
		10	250	2	1.26

The Plaxis results indicate that and SKZ-38 sheet pile can achieve the required factor of safety of 1.3 with 15 feet of embedment into the underlying sand. In order to achieve the required factor of safety with the discrete HP14x89 piles, the spacing would have to be reduced to 2 feet on-center which is nearly con<nuous. GZA compared the average sec<on area per foot of slope which is tabulated below and found that the sheet pile walls u<lize approximately 4.7 in^2/foot compared to 13.1 in^2/l of the 2.0 l spaced H piles.

Pile Type		Section Area (in^2)	Section Width (in)	Spacing (ft)	Average Section Area per Foot Slope Length (in^2/ft)
HP	14x89	26.1	13.8	2	13.1
SKZ	38	11.07	28.5	NA	4.7

An alterna<ve pile-support analysis was also conducted using Slope/W and the Brom's method. The results indicated that a pile shear resistance corresponding to sand embedment between the "long pile" and "short pile" case would provide the same factor of safety as Plaxis2D c-phi reduc<on. Therefore, Plaxis was considered to be validated. The Slope/W check is not shown in the aL achments.

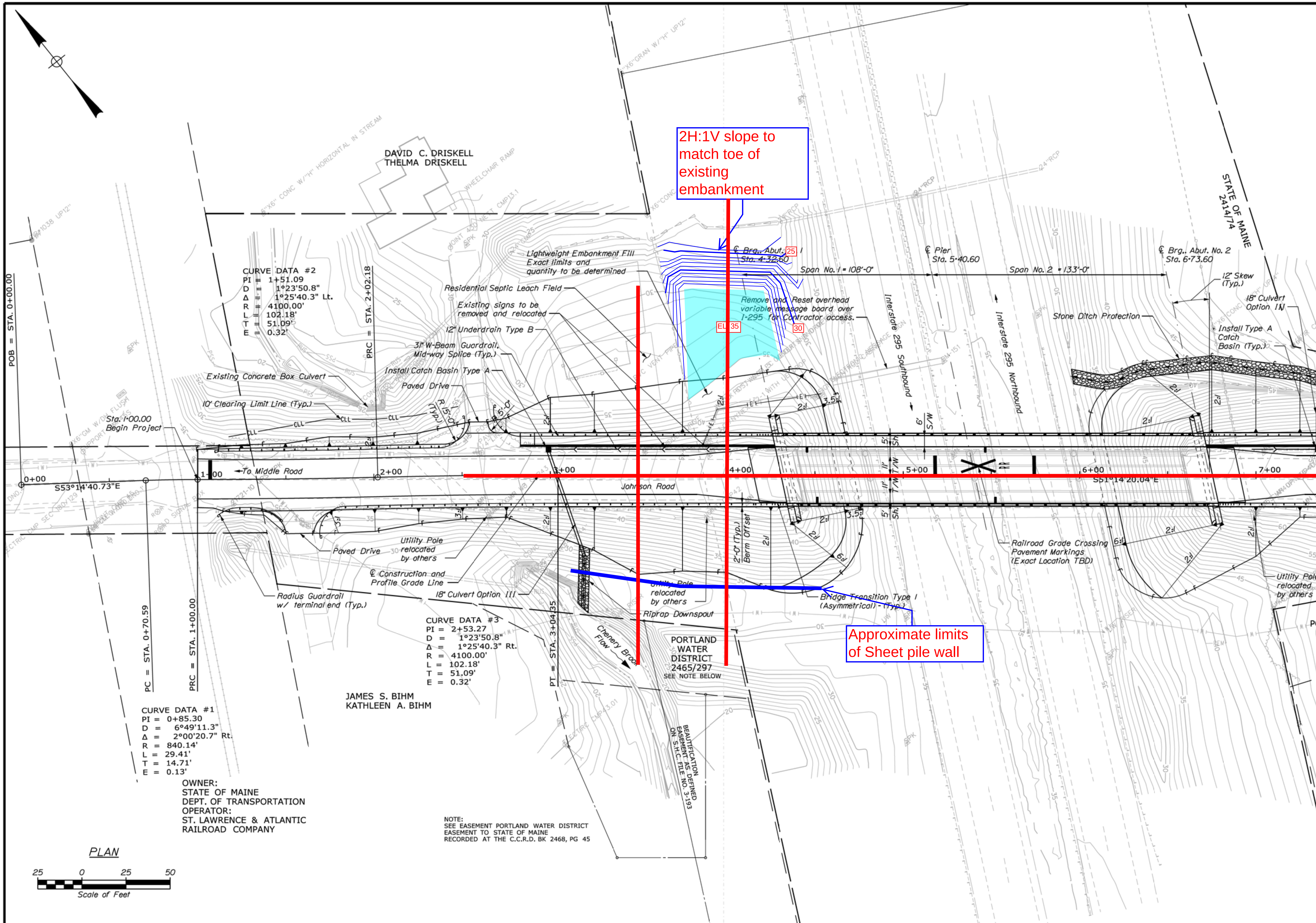
Conclusion

1. Longitudinal: The global stability meets the minimum required criteria for sta<c condi<ons in both direc<ons and pseudosta<c condi<on at Abutment 2.

2. Cross Section (Station 3+50, Right to Left): The global stability meets the minimum required criteria for sta<c condi<on. Since this sec<on includes weaker underlying clay soils than the Abutment 2 sec<on at approximately 7+00, the Abutment 2 Approach sec<ons are considered to meet the minimum factor of safety by observa<on. Although the factor of safety for deeper seated instability is greater than 1.3, the model predicts a factor of safety as low as 1.2 infinite slope-type surfaces through the Type D gravel, where surfaces with FS< 1.3 extend approximately 1.7 feet beyond the slope face. In our opinion, the TRM will mi<gate the poten<al for these shallow slip surfaces. It is also noted that the actual effec<ve fric<on angle for compacted Type D gravel is likely 40 degrees or more, which would also mi<gate instability of the 1.5:1 slope.

3. Cross Section (Station 4+00, Right to Left): A toe berm describe above and shown on the sketch on the following page is required to meet the minimum required criteria for sta<c condi<on for global stability.

4. Cross Section (Station 3+50 & 4+00, Left to Right): To meet the minimum required factor of safety for global stability, GZA recommends sheet piles with a minimum sec<on modulus of 62.32 in^3/l that are a minimum 70 feet long at the approximate loca<on shown on the plan on the following page.



Date:10/11/2021

Username:

Division: BRIDGE

Filename: ...\00\BRIDGE\MSTA\004_BDPlan1.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

021721.00

WIN 021721.00

BRIDGE PLANS

BRIDGE NO. 5792

PROJ. MANAGER	BY	DATE
DESIGN-DETAILED	A. Lachance	OCT 2021
CHECKED-REVIEWED	B. Nichols	
DESIGN2-REVIEWED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

JOHNSON ROAD BRIDGE
INTERSTATE 295
FALMOUTH CUMBERLAND COUNTY

GENERAL PLAN

SHEET NUMBER

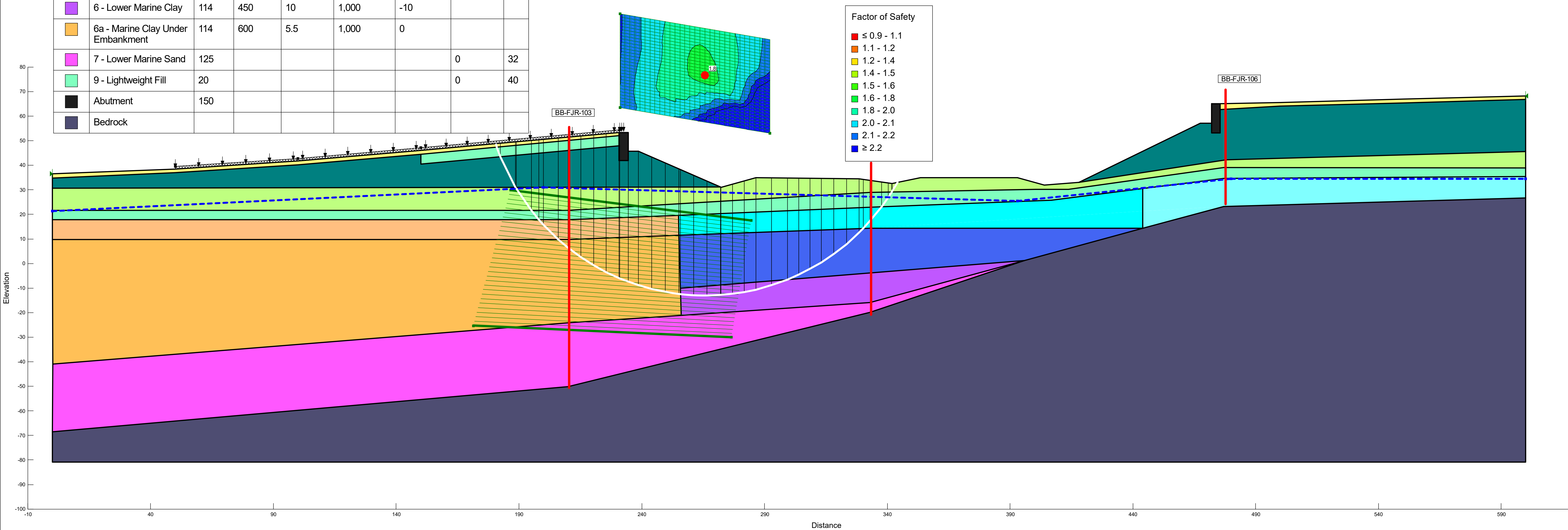
4

OF 25

Abutment 1

File Name: Abutments 10.15.21.gsz
Date: 11/17/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\
Method: Bishop
Surcharge (Unit Weight): 250 pcf

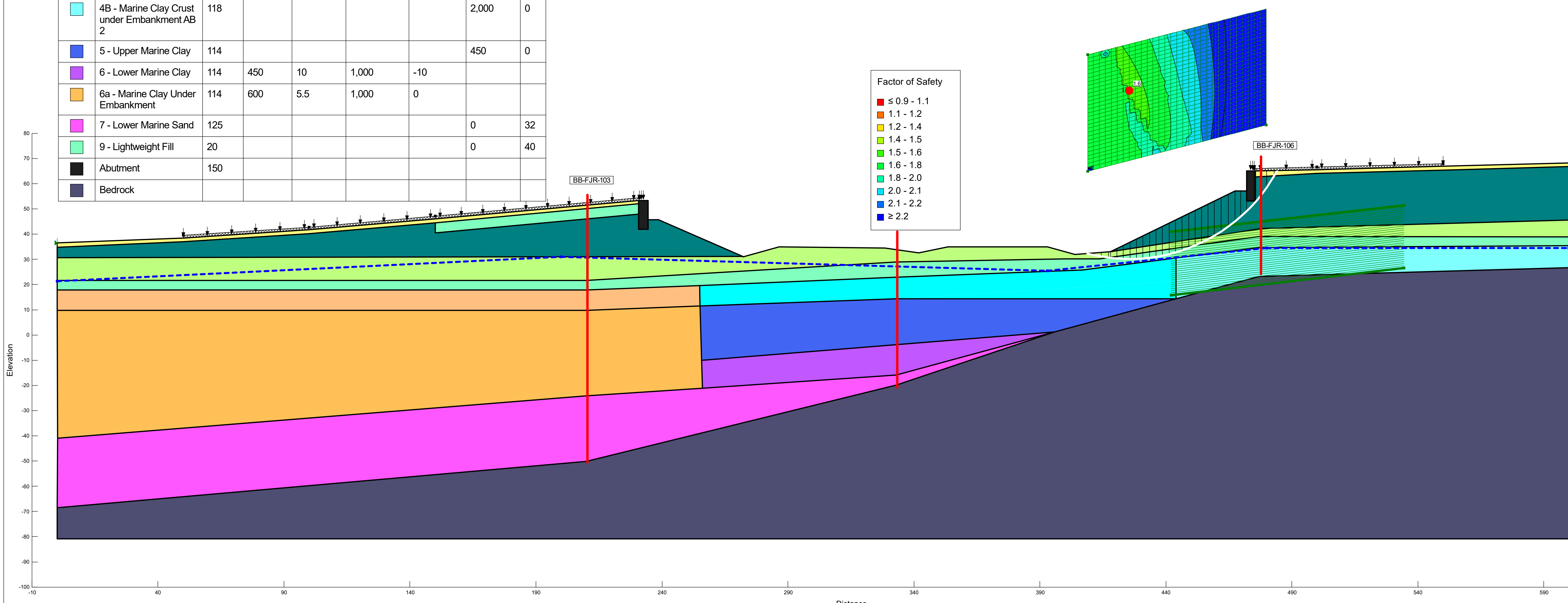
Color	Name	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Cohesion' (psf)	Phi' (°)
	1 - New Fill	125					0	32
	2 - Upper Existing Fill	130					0	40
	2A - Lower Existing Fill	125					0	36
	3 - Upper Marine Sand	125					0	30
	4- Marine Clay Crust	116	1,250	-50	450	23		
	4A - Marine Clay Crust under Embankment	116	1,250	-50	630	23		
	4B - Marine Clay Crust under Embankment AB 2	118					2,000	0
	5 - Upper Marine Clay	114					450	0
	6 - Lower Marine Clay	114	450	10	1,000	-10		
	6a - Marine Clay Under Embankment	114	600	5.5	1,000	0		
	7 - Lower Marine Sand	125					0	32
	9 - Lightweight Fill	20					0	40
	Abutment	150						
	Bedrock							



Abutment 2

File Name: Abutments 10.15.21.gsz
Date: 11/17/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\
Method: Bishop
Surcharge (Unit Weight): 250 pcf

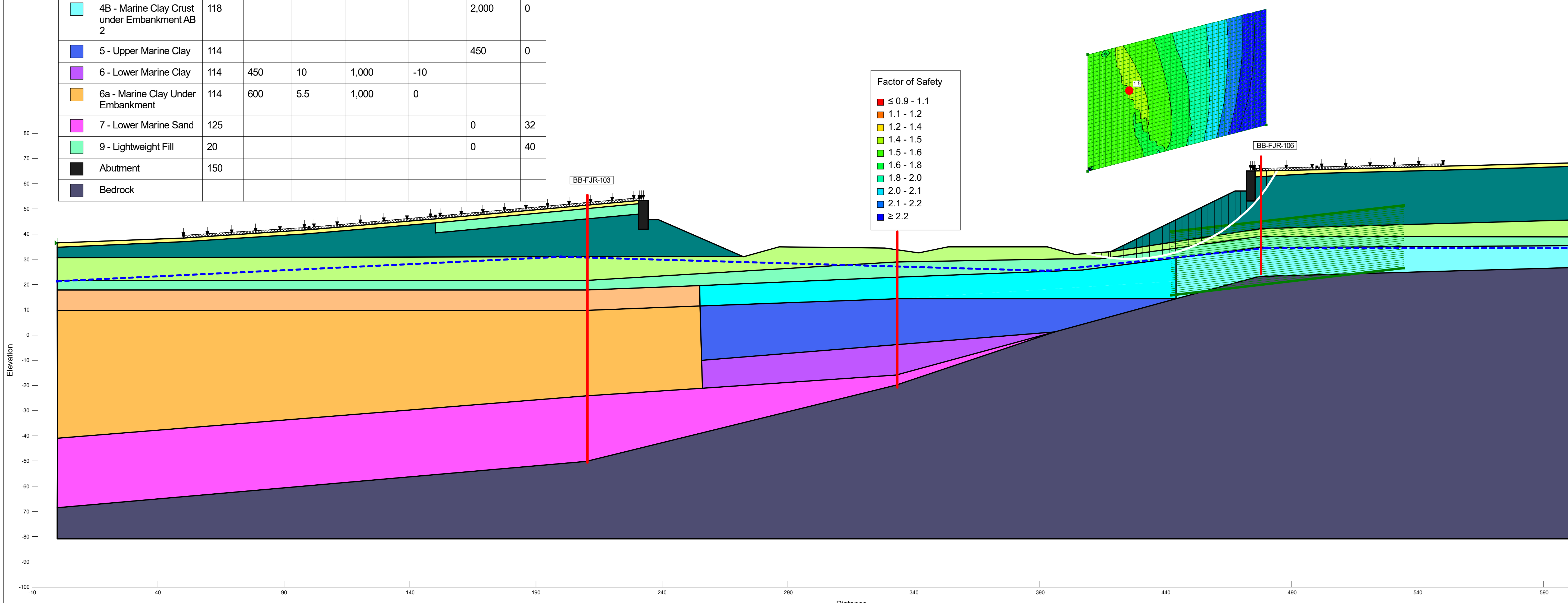
Color	Name	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Cohesion' (psf)	Phi' (°)
	1 - New Fill	125					0	32
	2 - Upper Existing Fill	130					0	40
	2A - Lower Existing Fill	125					0	36
	3 - Upper Marine Sand	125					0	30
	4- Marine Clay Crust	116	1,250	-50	450	23		
	4A - Marine Clay Crust under Embankment	116	1,250	-50	630	23		
	4B - Marine Clay Crust under Embankment AB 2	118					2,000	0
	5 - Upper Marine Clay	114					450	0
	6 - Lower Marine Clay	114	450	10	1,000	-10		
	6a - Marine Clay Under Embankment	114	600	5.5	1,000	0		
	7 - Lower Marine Sand	125					0	32
	9 - Lightweight Fill	20					0	40
	Abutment	150						
	Bedrock							



Abutment 2 - Pseudostatic

File Name: Abutments 10.15.21.gsz
Date: 11/17/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\
Method: Bishop
Surcharge (Unit Weight): 250 pcf

Color	Name	Unit Weight (pcf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Cohesion' (psf)	Phi' (°)
	1 - New Fill	125					0	32
	2 - Upper Existing Fill	130					0	40
	2A - Lower Existing Fill	125					0	36
	3 - Upper Marine Sand	125					0	30
	4- Marine Clay Crust	116	1,250	-50	450	23		
	4A - Marine Clay Crust under Embankment	116	1,250	-50	630	23		
	4B - Marine Clay Crust under Embankment AB 2	118					2,000	0
	5 - Upper Marine Clay	114					450	0
	6 - Lower Marine Clay	114	450	10	1,000	-10		
	6a - Marine Clay Under Embankment	114	600	5.5	1,000	0		
	7 - Lower Marine Sand	125					0	32
	9 - Lightweight Fill	20					0	40
	Abutment	150						
	Bedrock							



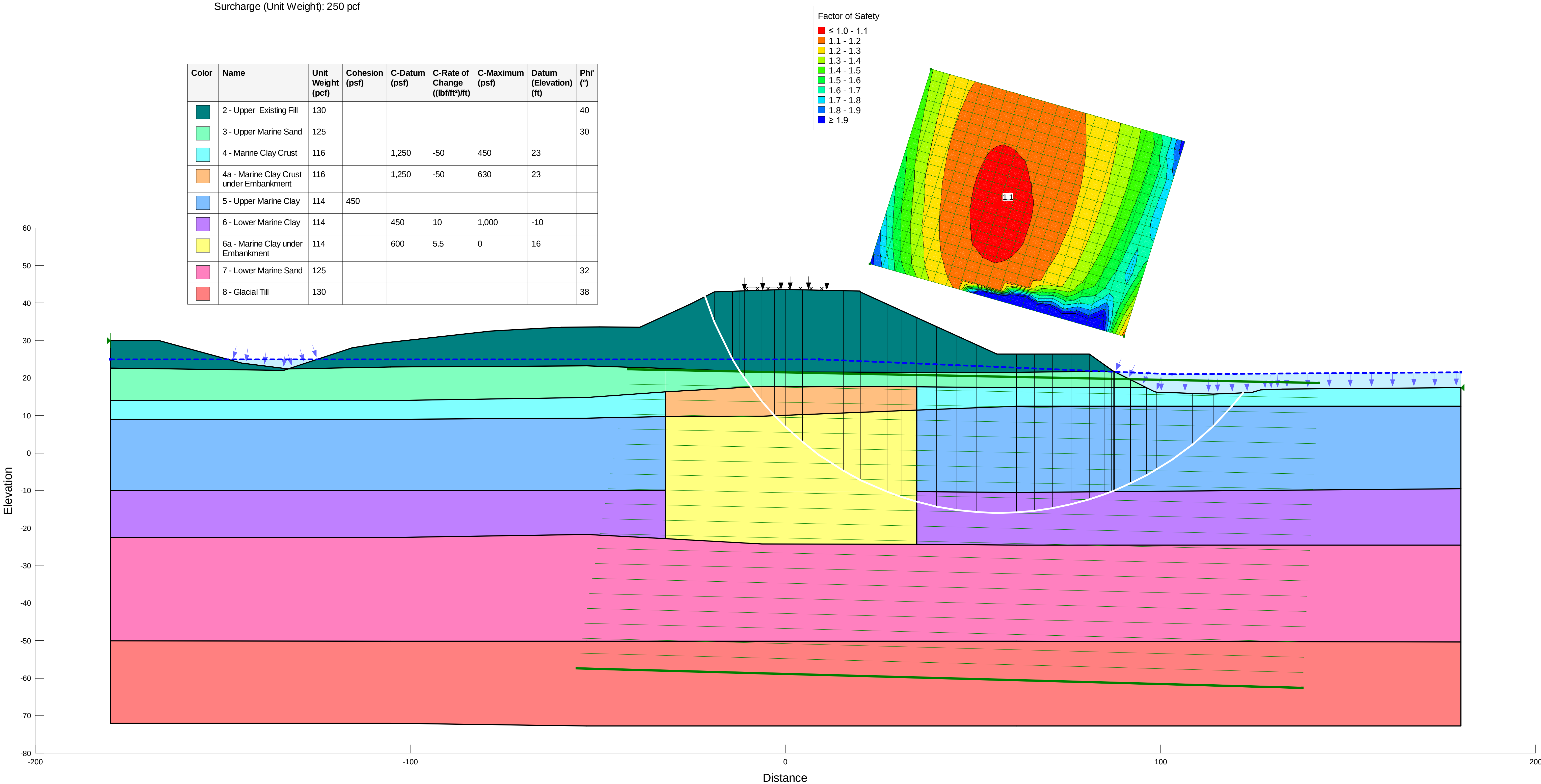
Station 3+50 Existing Conditions

File Name: Johnson Road_AB 1_STA 3+50_10.18.2021.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	2 - Upper Existing Fill	130						40
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	0	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38



Station 3+50 Left to Right

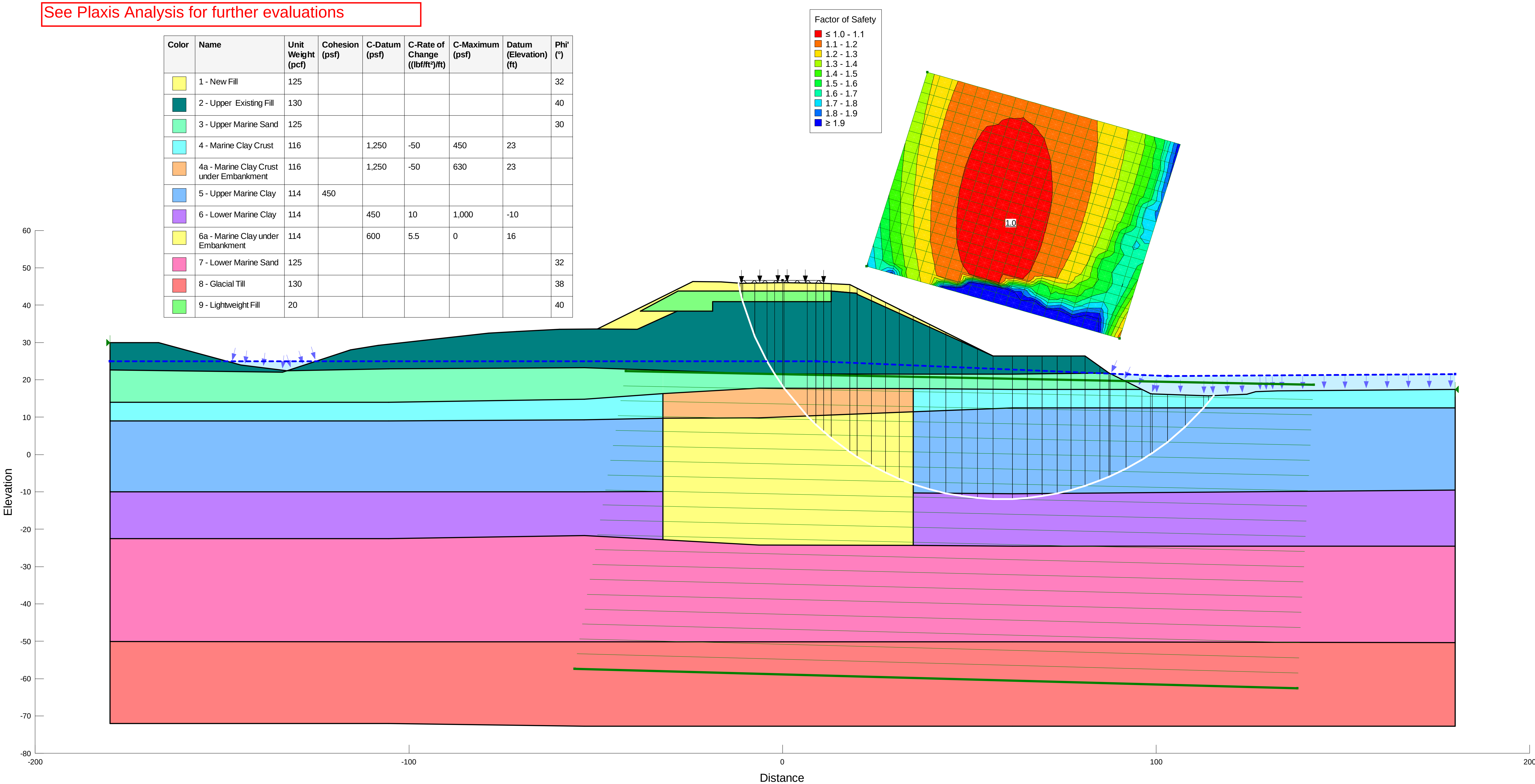
File Name: Johnson Road_AB 1_STA 3+50_10.18.2021.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

See Plaxis Analysis for further evaluations

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
	2 - Upper Existing Fill	130						40
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	0	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40

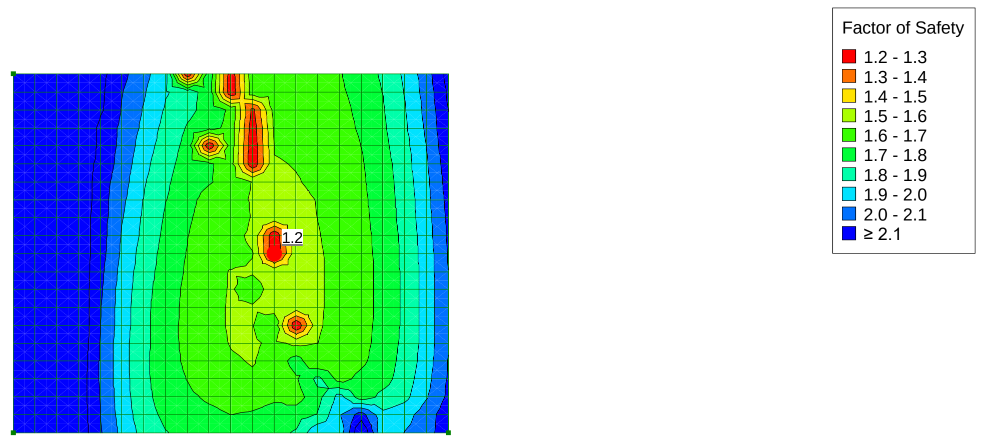


Station 3+50 Right to Left
Surficial Failures

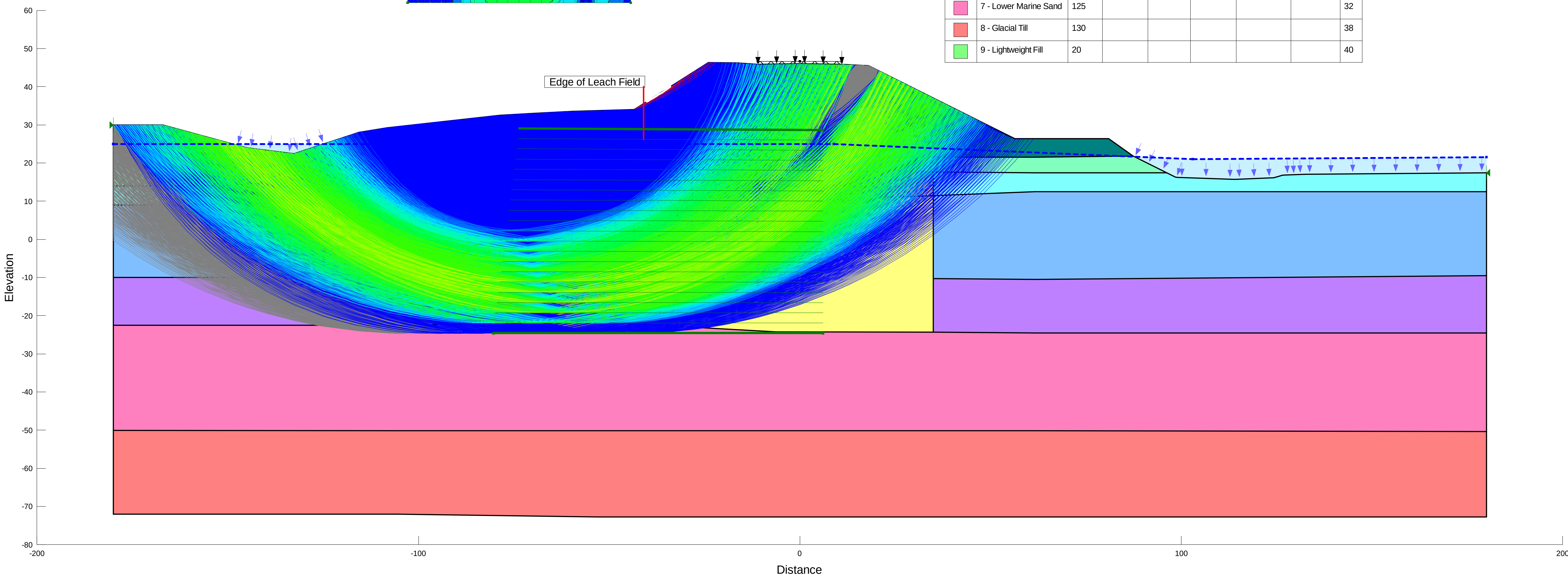
File Name: Johnson Road_AB 1_STA 3+50_12.02.2021.gsz
Date: 02/18/2022
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf



Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft^2)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
	10 - Type D Gravel	135						36
	2 - Upper Existing Fill	130						40
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	0	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40



Station 3+50 Right to Left

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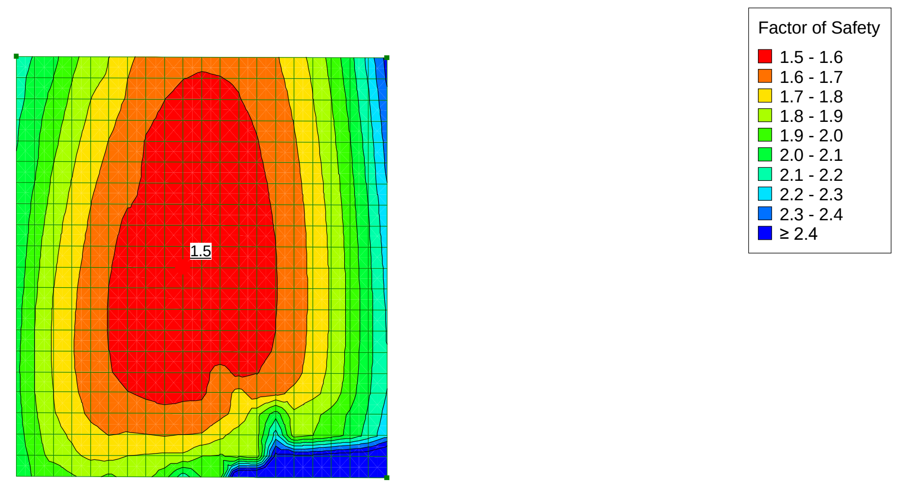
Method: Bishop

Surcharge (Unit Weight): 250 pcf



Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft^2)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
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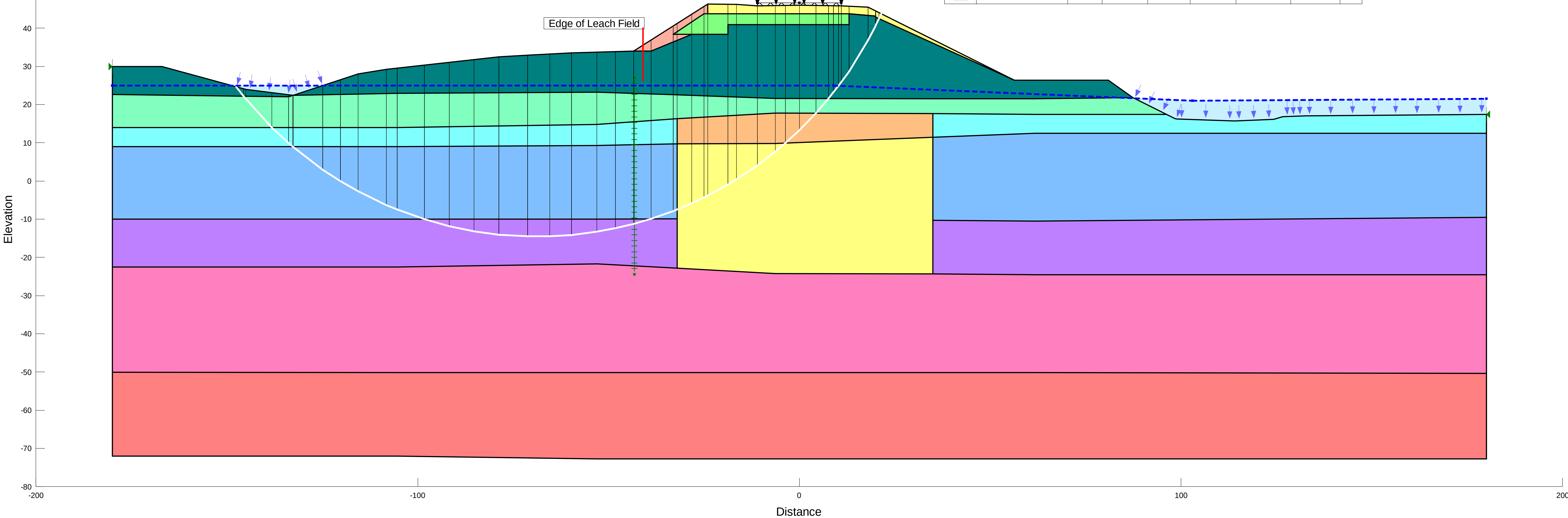
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	0	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40



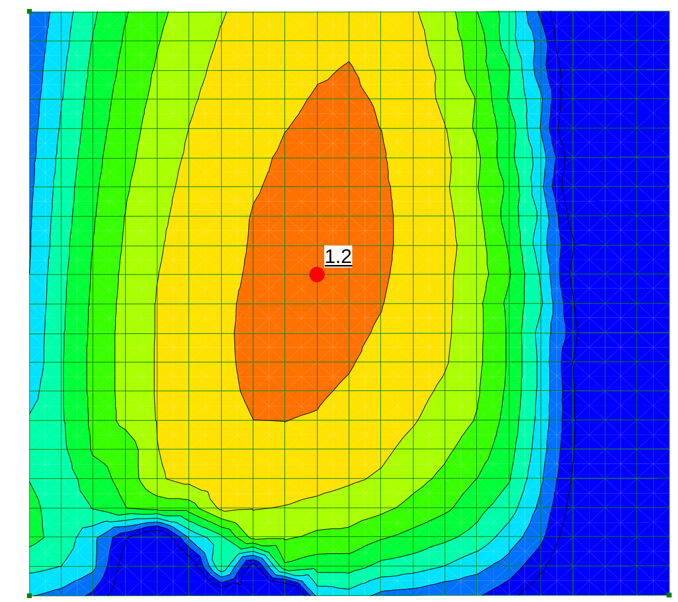
Station 4+00 Existing Conditions

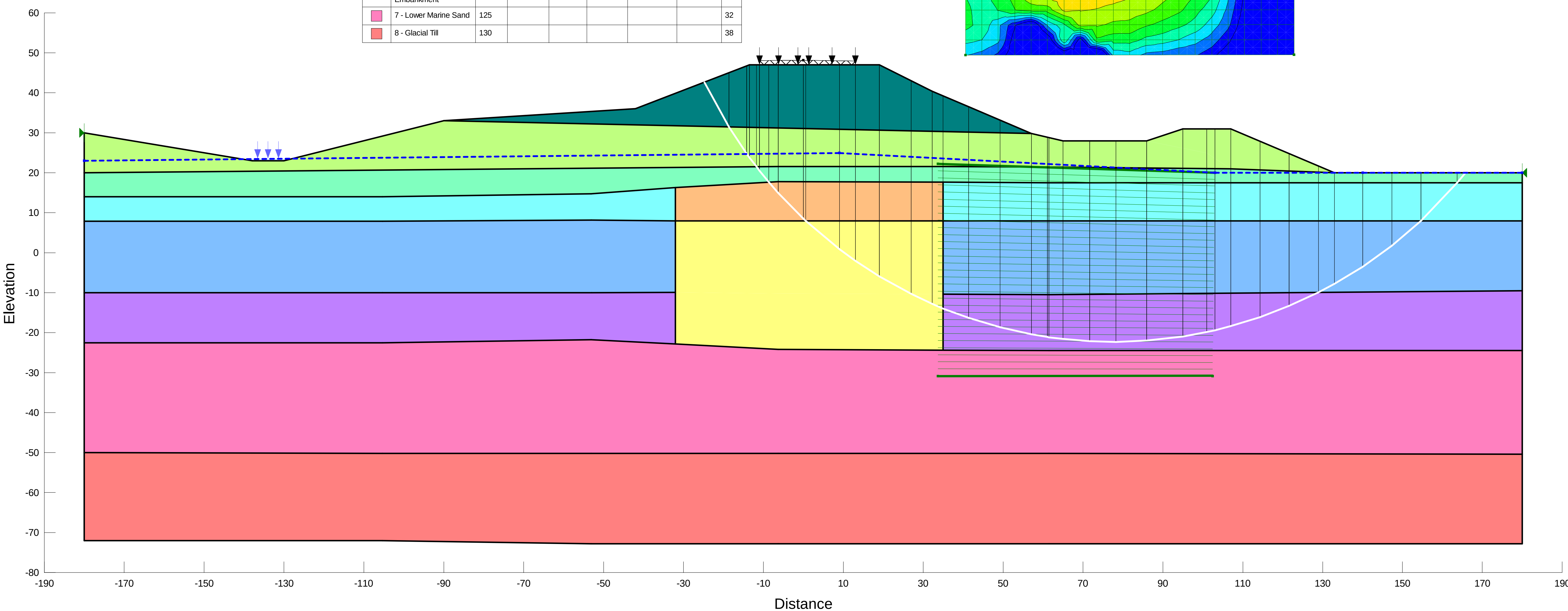
File Name: Johnson Road_AB 1_STA 4+00.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	2 - Upper Existing Fill	130						40
	2A - Lower Existing Fill	125						36
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	1,000	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38





Station 4+00 Left to Right

File Name: Johnson Road_AB 1_STA 4+00.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

See Plaxis Analysis for further evaluations

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
	2 - Upper Existing Fill	130						40
	2A - Lower Existing Fill	125						36
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
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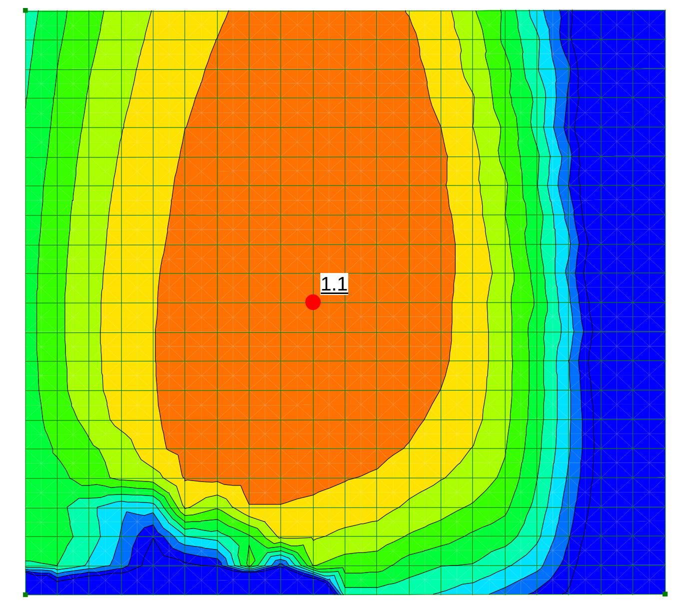
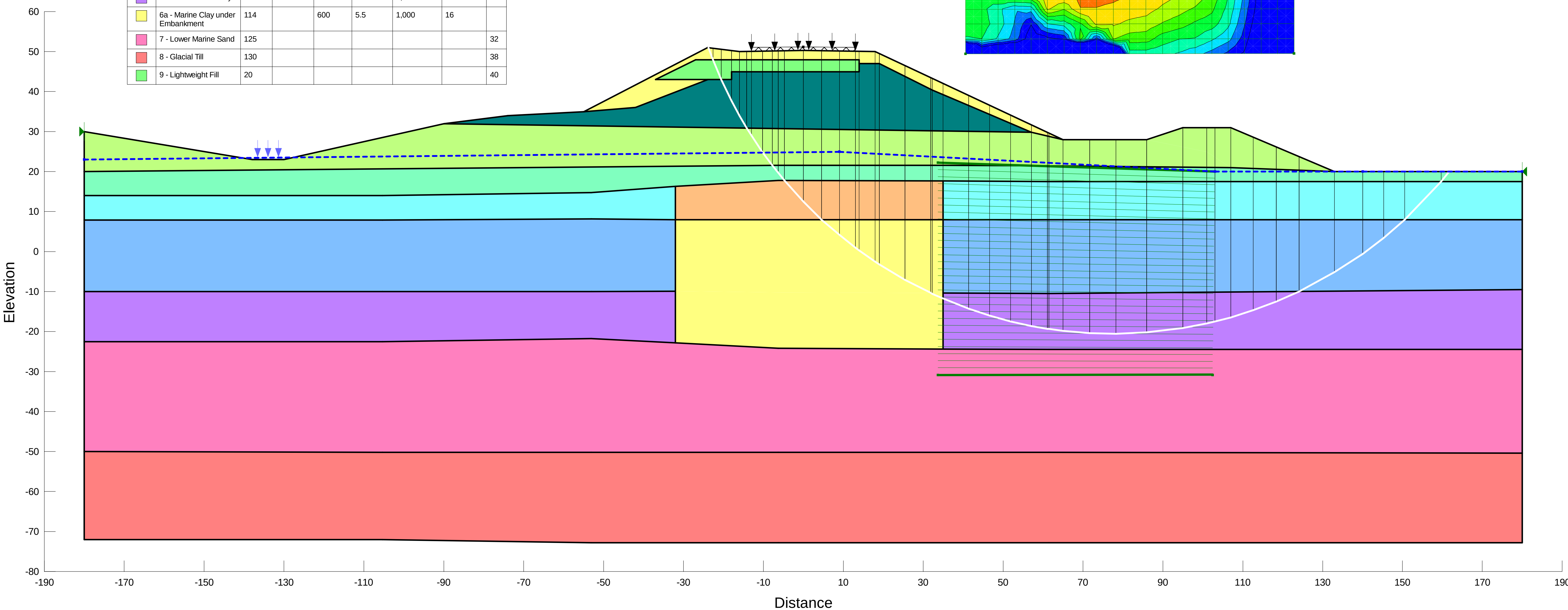
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	1,000	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40



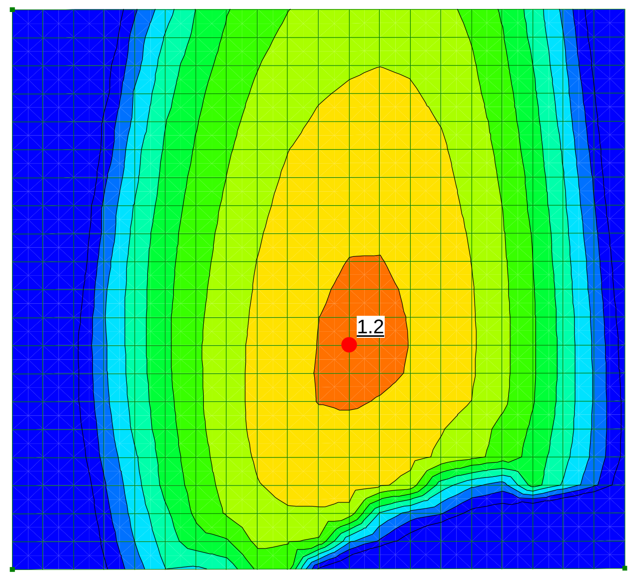


Station 4+00 Right to Left

File Name: Johnson Road_AB 1_STA 4+00.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf



Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
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	6 - Lower Marine Clay	114		450	10	1,000	-10	
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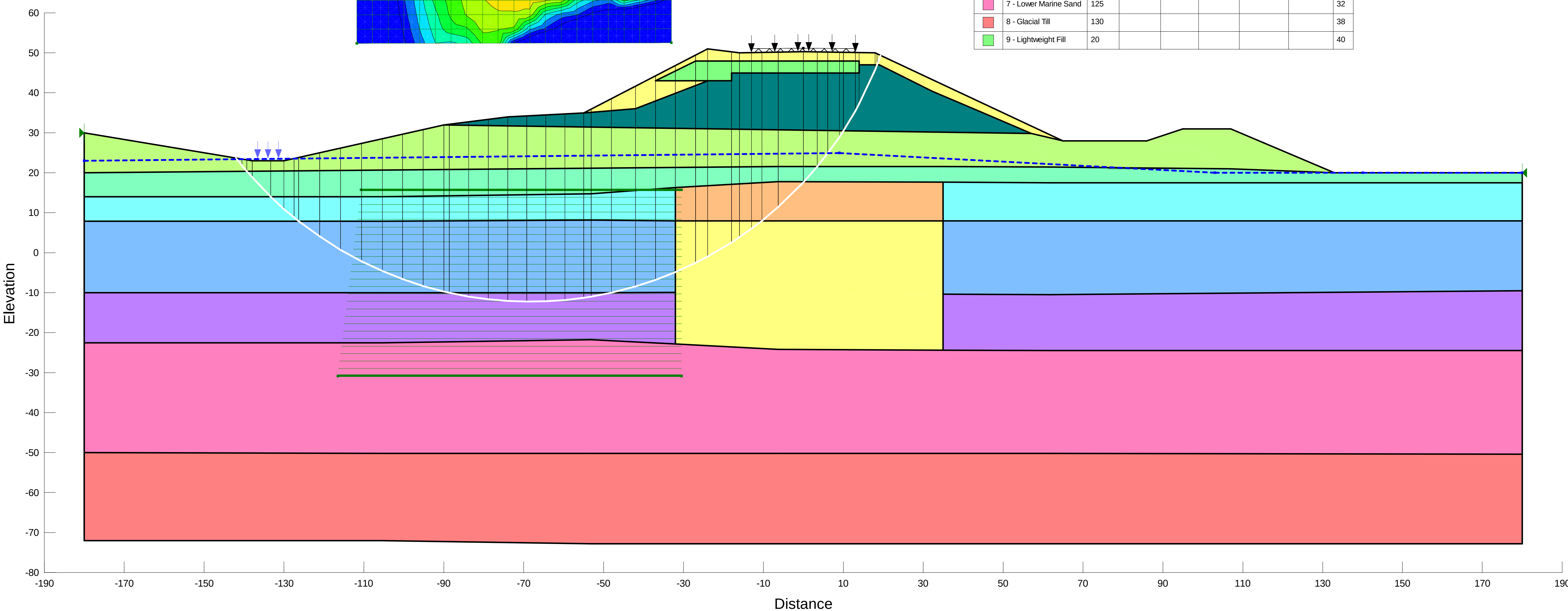
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40



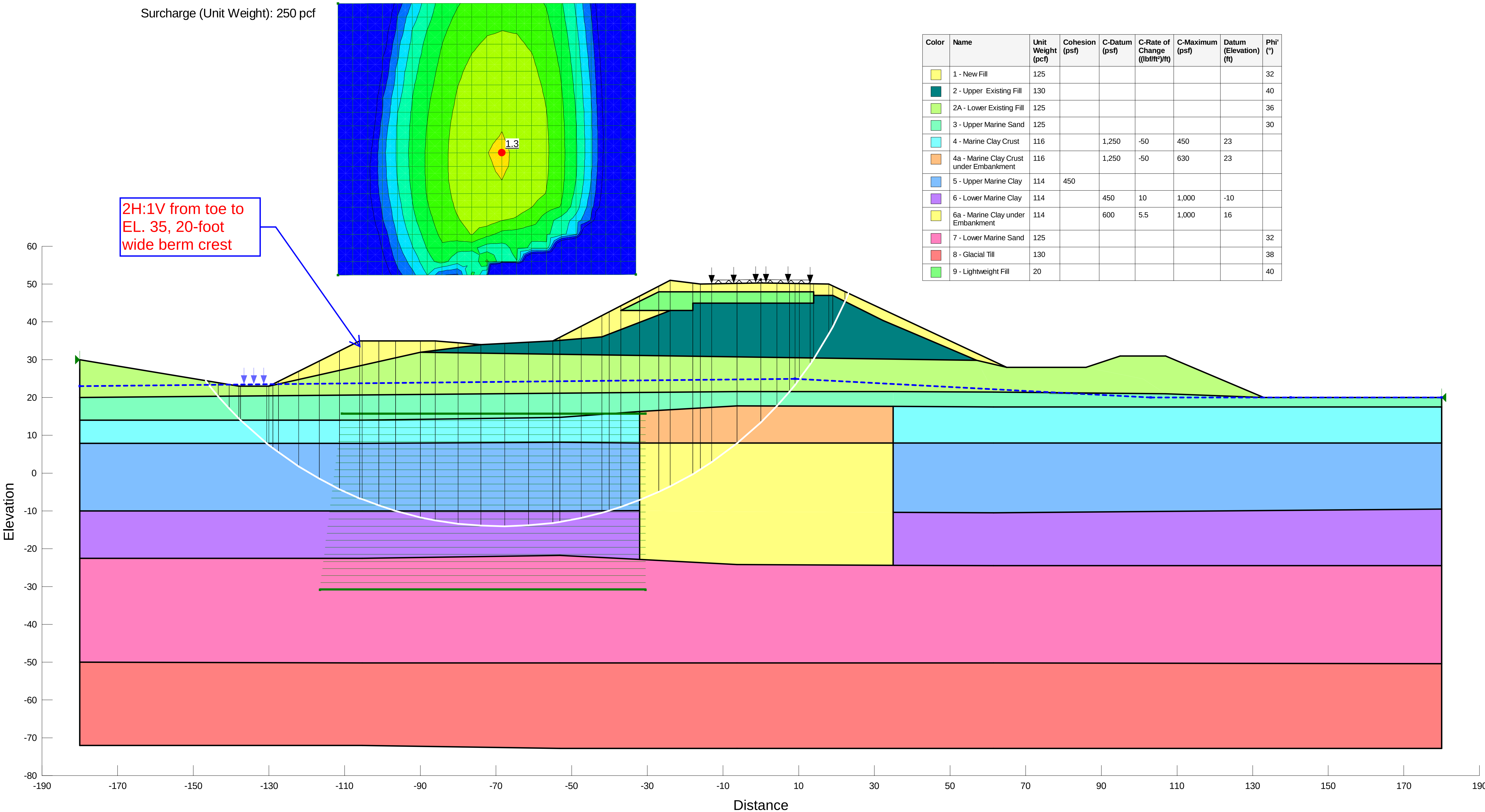
Station 4+00 Right to Left
with Toe Berm

File Name: Johnson Road_AB 1_STA 4+00.gsz
Date: 11/11/2021
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
	2 - Upper Existing Fill	130						40
	2A - Lower Existing Fill	125						36
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	1,000	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
	9 - Lightweight Fill	20						40



Station 4+00 with Berm
Pseudostatic

File Name: Johnson Road_AB 1_STA 4+00.gsz
Date: 03/04/2022
Directory: P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Slope Stability\

Method: Bishop

Surcharge (Unit Weight): 250 pcf

Kh=0.105

Color	Name	Unit Weight (pcf)	Cohesion (psf)	C-Datum (psf)	C-Rate of Change ((lbf/ft²)/ft)	C-Maximum (psf)	Datum (Elevation) (ft)	Phi' (°)
	1 - New Fill	125						32
	2 - Upper Existing Fill	130						40
	2A - Lower Existing Fill	125						36
	3 - Upper Marine Sand	125						30
	4 - Marine Clay Crust	116		1,250	-50	450	23	
	4a - Marine Clay Crust under Embankment	116		1,250	-50	630	23	
	5 - Upper Marine Clay	114	450					
	6 - Lower Marine Clay	114		450	10	1,000	-10	
	6a - Marine Clay under Embankment	114		600	5.5	1,000	16	
	7 - Lower Marine Sand	125						32
	8 - Glacial Till	130						38
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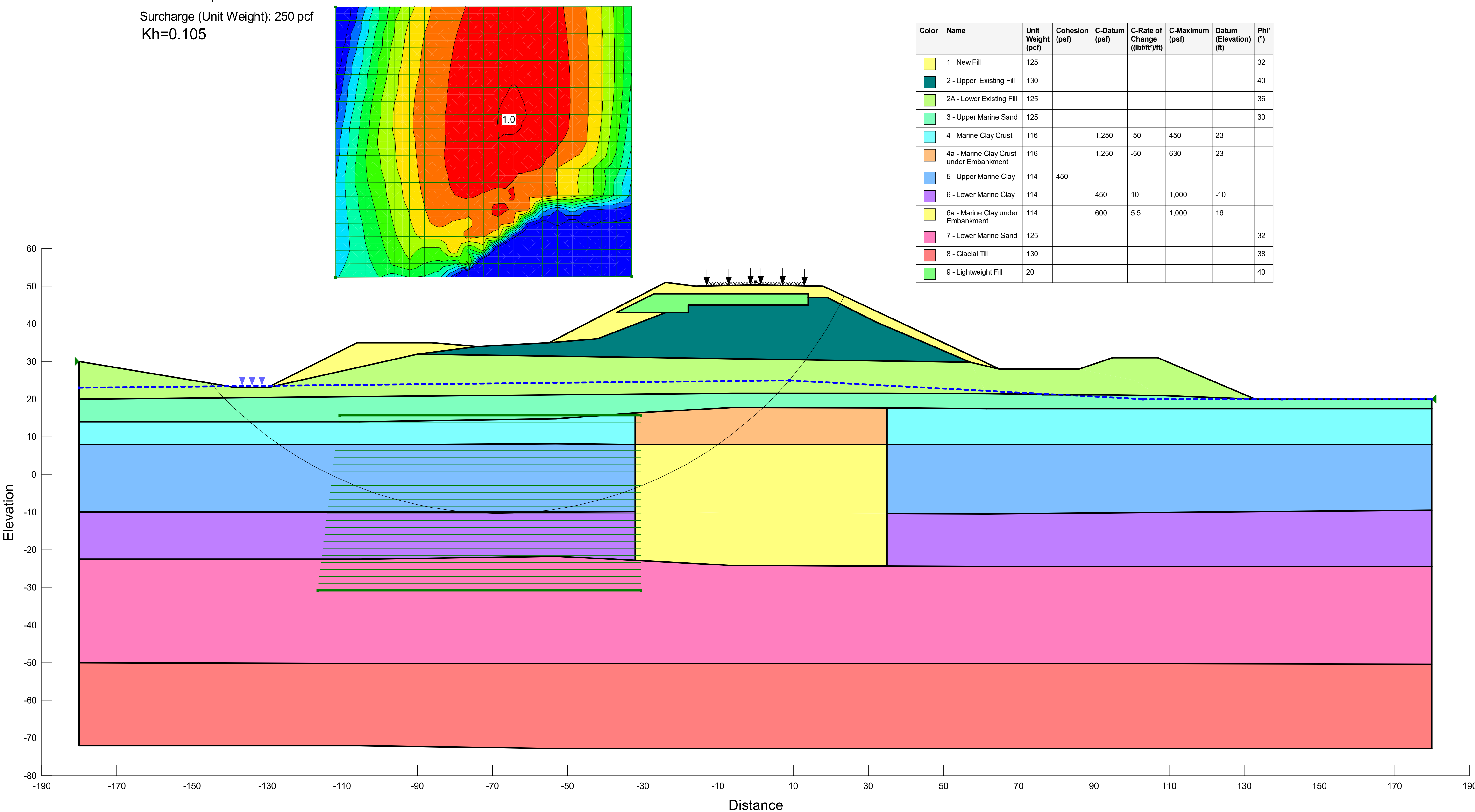


Sheet Pile Wall PLAXIS Analysis
(Station 3+50 and 4+00, Left to Right)

Design Properties_Sheetpile and H-pile

Select SKZ38 for sheetpile and HP14x89 for H-piles

Design Properties - Steel H Piling

Section Number	Weight Per Foot lb	Area of Selection A in.^{2}	Depth of Section d in.	Flange Width b_f in.	Flange Thickness t_f in.	Web Thickness t_w in.	Axis X-X I_x in.^{4}	Axis X-X S_x in.^{3}	Axis X-X r_x in.	Axis Y-Y I_y in.^{4}	Axis Y-Y S_y in.^{3}	Axis Y-Y r_y in.	Surface Area ft^2/ft
HP14X	117	34.4	14.21	14.885	0.805	0.805	1220	172	5.96	443	59.5	3.59	7.11
	102	30	14.01	14.735	0.705	0.705	1050	150	5.92	380	51.4	3.56	7.06
	89	26.1	13.83	14.695	0.615	0.615	904	131	5.88	326	44.3	3.53	7.02

SECTION	Width (w) in (mm)	Height (h) in (mm)	Thickness (t) in (mm)	Cross Sectional Area in^2/ft (cm^2/m)	WEIGHT Pile lb/ft (kg/m)	WEIGHT Wall lb/ft^2 (kg/m^2)	SECTION MODULUS Elastic in^3/ft (cm^3/m)	SECTION MODULUS Plastic in^3/ft (cm^3/m)	Moment of Inertia in^4/ft (cm^4/m)	COATING AREA Both Sides ft^2/ft (m^2/m)	COATING AREA Coating Area ft^2/ft^2 (m^2/m^2)
SCZ 12	28.50 723.9	10.00 254.0	0.200 5.1	3.32 70.3	27.01 40.20	11.37 55.51	11.52 619	13.10 704	57.6 7866	6.10 1.86	1.28 1.28
SCZ 14	28.50 723.9	10.00 254.0	0.250 6.4	4.18 88.48	33.81 50.31	14.23 69.50	14.36 772	16.32 877.4	71.82 9808	6.10 1.86	1.28 1.28
SCZ 16	28.50 723.9	10.00 254.0	0.276 7.0	4.62 97.79	37.37 55.61	15.73 76.82	15.75 847	17.97 965.9	78.73 10751	6.10 1.86	1.28 1.28
SCZ 17N	28.50 723.9	10.00 254.0	0.295 7.5	4.95 104.78	40.03 59.58	16.86 82.32	16.87 907	19.21 1033	84.35 11519	6.10 1.86	1.28 1.28
SCZ 18N	28.50 723.9	10.00 254.0	0.317 8.1	5.31 112.39	42.94 63.91	18.08 88.28	18.10 973	20.61 1108	90.48 12356	6.10 1.86	1.28 1.28
SCZ 21N	28.50 723.9	10.00 254.0	0.375 9.5	6.29 133.06	50.84 75.66	21.41 104.54	21.43 1152	24.40 1312	107.13 14629	6.10 1.86	1.28 1.28
SKZ 20	28.50 723.9	16.00 406.4	0.315 8.0	6.00 127.00	48.24 71.79	20.31 99.17	31.69 1704	36.66 1970.97	253.51 34618	7.60 2.32	1.60 1.60
SKZ 22	28.50 723.9	16.00 406.4	0.335 8.5	6.30 133.35	51.30 76.34	21.60 105.46	33.43 1797	38.94 2093.55	267.40 36515	7.60 2.32	1.60 1.60
SKZ 23	28.50 723.9	16.00 406.4	0.354 9.0	6.70 141.82	54.20 80.66	22.82 111.42	35.61 1915	41.12 2210.75	284.90 38905	7.60 2.32	1.60 1.60
SKZ 24	28.50 723.9	16.00 406.4	0.375 9.5	7.10 150.28	57.43 85.47	24.18 118.06	37.73 2028	43.52 2339.78	301.80 41213	7.60 2.32	1.60 1.60
SKZ 25	28.50 723.9	16.00 406.4	0.399 10.1	7.60 160.87	61.10 90.93	25.73 125.61	40.14 2158	46.24 2486.02	321.12 43851	7.60 2.32	1.60 1.60
SKZ 31	28.50 723.9	18.00 457.2	0.450 11.4	9.07 192.04	73.82 109.85	31.08 151.75	51.56 2772	60.51 3253.29	464.05 63369	8.06 2.46	1.70 1.70
SKZ 33	28.50 723.9	18.00 457.2	0.475 12.1	9.40 198.97	77.64 115.54	32.69 159.61	54.89 2951	63.57 3417.68	494.03 67462	8.06 2.46	1.70 1.70
SKZ 34	28.50 723.9	18.00 457.2	0.500 12.7	9.89 209.25	81.42 121.17	34.28 167.38	57.62 3098	66.86 3594.60	518.62 70821	8.06 2.46	1.70 1.70
SKZ 36	28.50 723.9	18.00 457.2	0.535 13.6	10.78 228.10	86.81 129.19	36.55 178.46	60.71 3264	71.58 3848.17	546.43 74619	8.06 2.46	1.70 1.70
SKZ 38	28.50 723.9	18.00 457.2	0.550 14.0	11.07 234.42	88.95 132.37	37.45 182.85	62.32 3350	73.52 3952.44	560.85 76588	8.06 2.46	1.70 1.70

Plaxis Material Parameters

Hardening Soil Model

Property	Unit	Value	Value	Value	Value	Value
Material set						
Identification number		3	5	6	7	10
Identification		Lower Marine Clay	Marine Clay Crust	Marine Clay Crust under Emba	Marine Clay under Embankmen	Upper Marine Clay
Material model		Hardening soil	Hardening soil	Hardening soil	Hardening soil	Hardening soil
Drainage type		Undrained (B)	Undrained (B)	Undrained (B)	Undrained (B)	Undrained (B)
Colour		RGB 174, 94, 217	RGB 166, 245, 252	RGB 238, 178, 119	RGB 250, 254, 22	RGB 161, 190, 232
Comments						
General properties						
γ_{unsat}	lbf/ft³	114.0	116.0	116.0	114.0	114.0
γ_{sat}	lbf/ft³	114.0	116.0	116.0	114.0	114.0
E_{50}^{ref}	lbf/ft²	50.31E3	50.31E3	50.31E3	50.31E3	50.31E3
E_{oed}^{ref}	lbf/ft²	40.25E3	40.25E3	40.25E3	40.25E3	40.25E3
E_{ur}^{ref}	lbf/ft²	434.7E3	434.7E3	434.7E3	434.7E3	434.7E3
power (m)		1.000	1.000	1.000	1.000	1.000
Alternatives						
Use alternatives		☐	☐	☐	☐	☐
C_c		0.2400	0.2400	0.2400	0.2400	0.2400
C_s		0.02000	0.02000	0.02000	0.02000	0.02000
e_{init}		1.000	1.000	1.000	1.000	1.000
Strength						
c_{ref}	lbf/ft²	450.0	1250	1250	600.0	450.0
φ (phi)	°	0.000	0.000	0.000	0.000	0.000
Strength						
c_{inc}	lbf/ft²/ft	10.00	-50.00	-50.00	5.500	0.000
y_{ref}	ft	-10.00	23.00	23.00	16.00	0.000

Lower Marine Clay | **Marine Clay Crust** | **Marine Clay Crust under Embankment** | **Marine Clay under Embankment** | **Upper Marine Clay**

Plaxis Material Parameters

Mohr Coulomb Model

Property	Unit	Value	Value	Value	Value	Value	Value	Value
Material set								
Identification number		1	2	4	8	9	11	14
Identification		Glacial Till	Light Weight Fill	Lower Marine Sand	New Fill	Upper Existing Fill	Upper Marine Sand	Lower Existing Fill
Material model		Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Drainage type		Drained	Drained	Drained	Drained	Drained	Drained	Drained
Colour		RGB 227, 149, 130	RGB 69, 251, 19	RGB 232, 135, 203	RGB 241, 246, 85	RGB 6, 101, 61	RGB 97, 255, 181	RGB 180, 253, 182
Comments								
General properties								
γ_{unsat}	lbf/ft³	130.0	20.00	125.0	125.0	130.0	125.0	125.0
γ_{sat}	lbf/ft³	130.0	20.00	125.0	125.0	130.0	125.0	125.0
Stiffness								
E	lbf/ft²	1.500E6	700.0E3	700.0E3	700.0E3	700.0E3	700.0E3	700.0E3
Alternatives								
G	lbf/ft²	576.9E3	269.2E3	269.2E3	269.2E3	269.2E3	269.2E3	269.2E3
E_{oed}	lbf/ft²	2.019E6	942.3E3	942.3E3	942.3E3	942.3E3	942.3E3	942.3E3
Strength								
c_{ref}	lbf/ft²	1.000	1.000	1.000	1.000	1.000	1.000	1.000
φ (phi)	°	38.00	40.00	32.00	32.00	40.00	30.00	36.00
		Glacial Till	**Light Weight Fill**	**Lower marine Sand**	**New Fill**	**Upper Existing Fill**	**Upper Marine Sand**	**Lower Existing Fill**

Plaxis Material Parameters

Piles

- Sheetpiles modeled as plates
- H-Piles modeled as embedded beam row elements

Property	Unit	Value
Material set		
Identification number		2
Identification		SKZ38
Comments		SKZ38
Colour		RGB 55, 121, 242
Material type		Elastoplastic
Properties		
Isotropic		☑
EA_1	lbf/ft	321.0E6
EA_2	lbf/ft	321.0E6
EI	lbf ft²/ft	112.9E6
d	ft	2.055
w	lbf/ft/ft	37.45
ν (nu)		0.2800
M_p	lbf ft/ft	259.7E3
$N_{p,1}$	lbf/ft	553.5E3
$N_{p,2}$	lbf/ft	553.5E3
Rayleigh α		0.000
Rayleigh β		0.000
Prevent punching		☐

SKZ38 Sheetpile

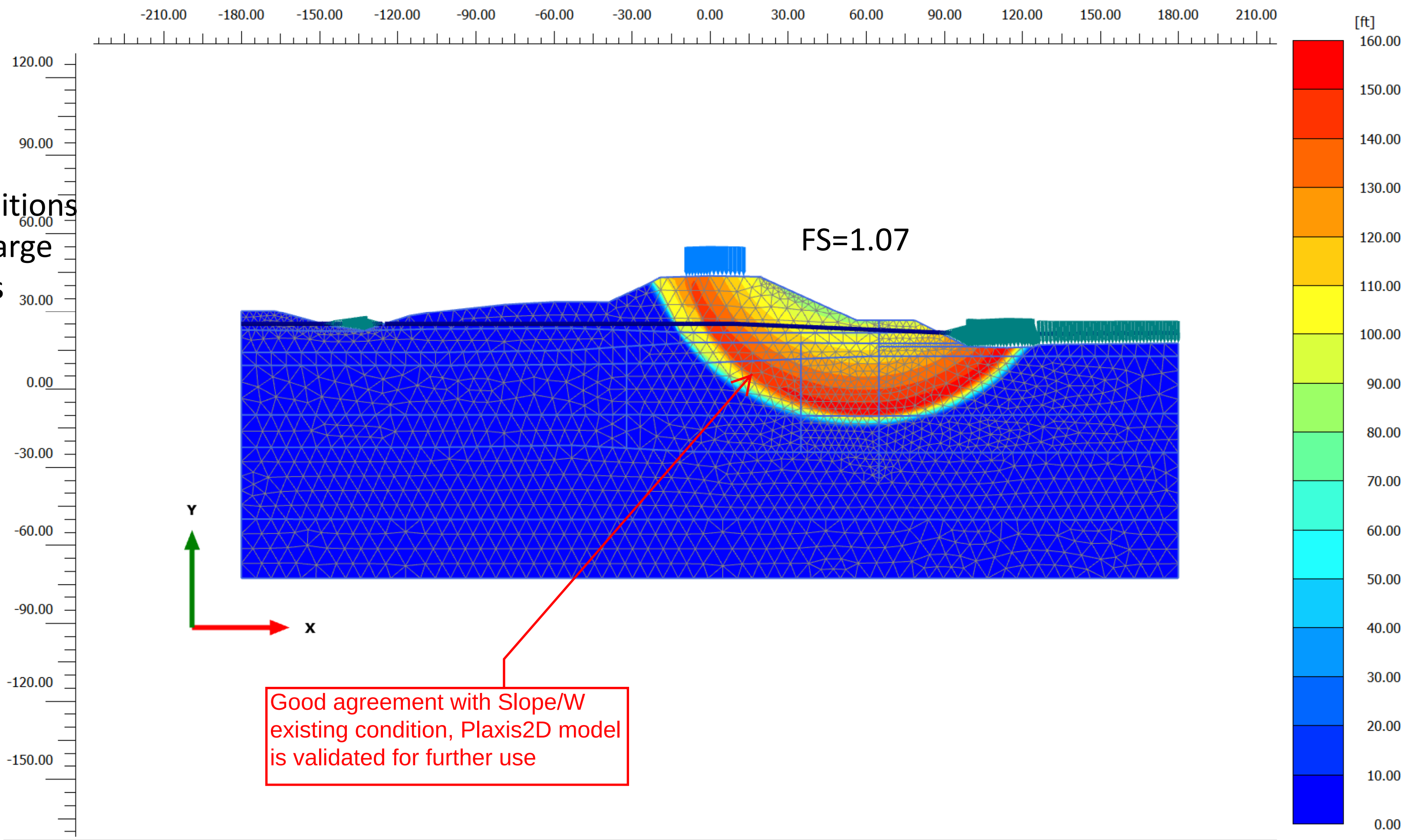
Property	Unit	Value
Material set		
Identification		HPile_5ftSpacing
Comments		
Colour		RGB 179, 72, 133
Material type		Elastoplastic
Properties		
E	lbf/ft²	4.176E9
γ	lbf/ft³	490.0
Beam type		User-defined
A	ft²	0.1826
I	ft⁴	0.04360
$L_{spacing}$	ft	5.000
M_p	lbf ft	545.8E3
N_p	lbf	1.315E6

Spacing varies

H- Pile (HP14x89)

St 3+50

Existing Conditions
250psf surcharge
No sheetpiles



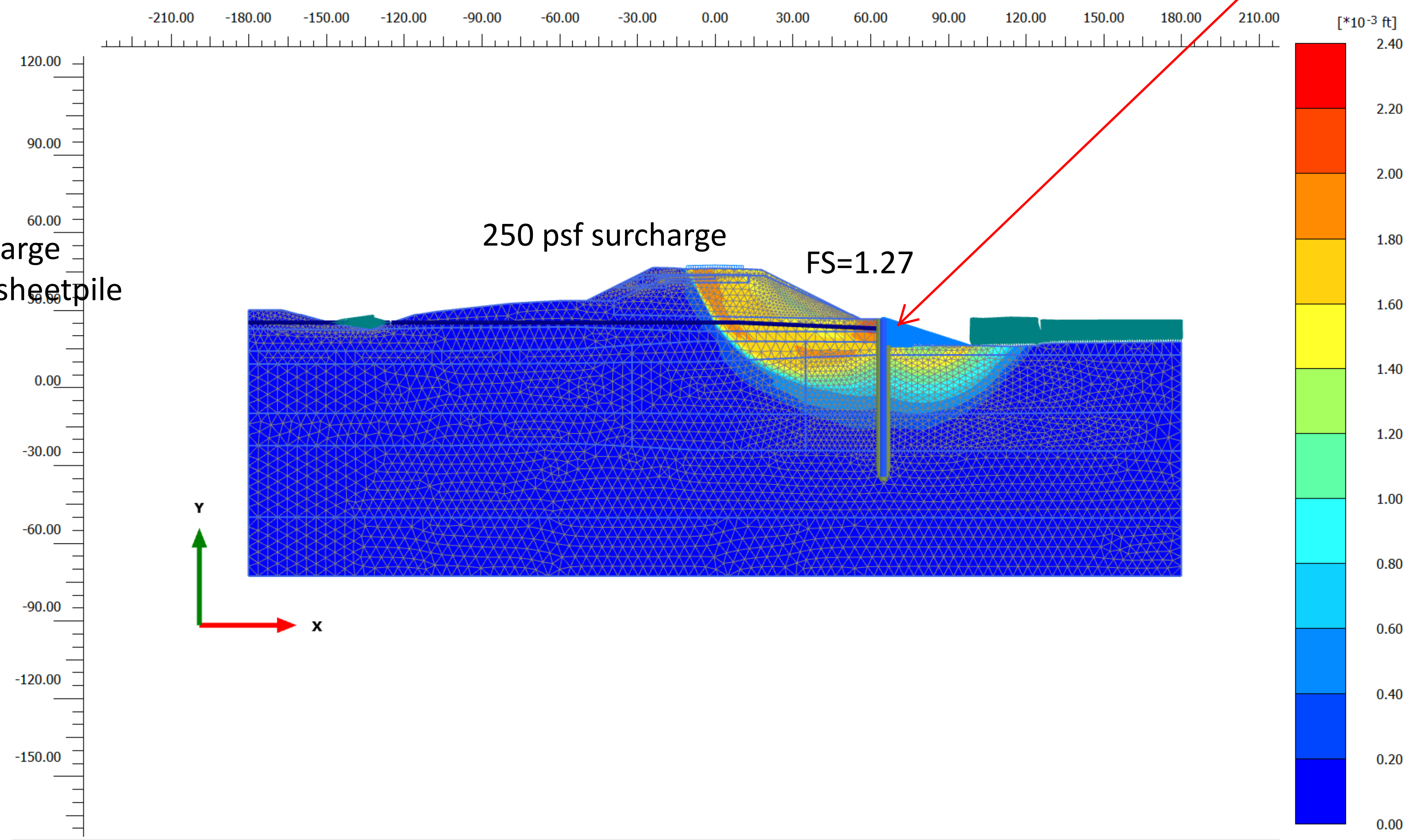
Incremental displacements |Δu| (scaled up 0.0500 times)

Maximum value = 155.2 ft (Element 1223 at Node 8535)

St 3+50

5ft ULFGA
250psf surcharge
10ft in sand sheetpile

Note: Vertical surcharge pressure used here in place of soil to prevent failures in this zone from controlling c-phi analysis.

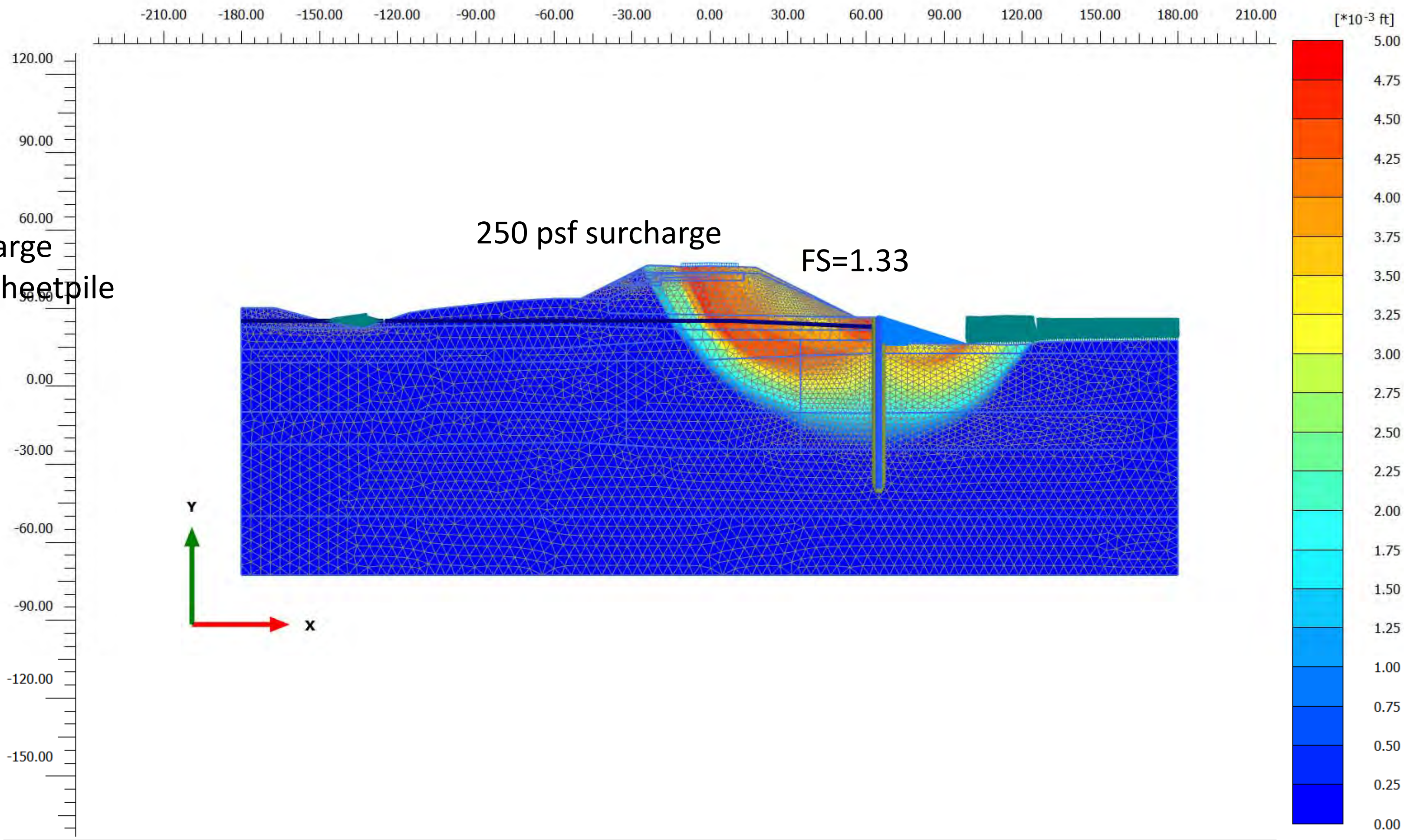


Incremental displacements |Δu| (scaled up 5.00*10³ times)

Maximum value = 2.356*10⁻³ ft (Element 2101 at Node 22552)

St 3+50

5ft ULFGA
250psf surcharge
15ft in sand sheetpile

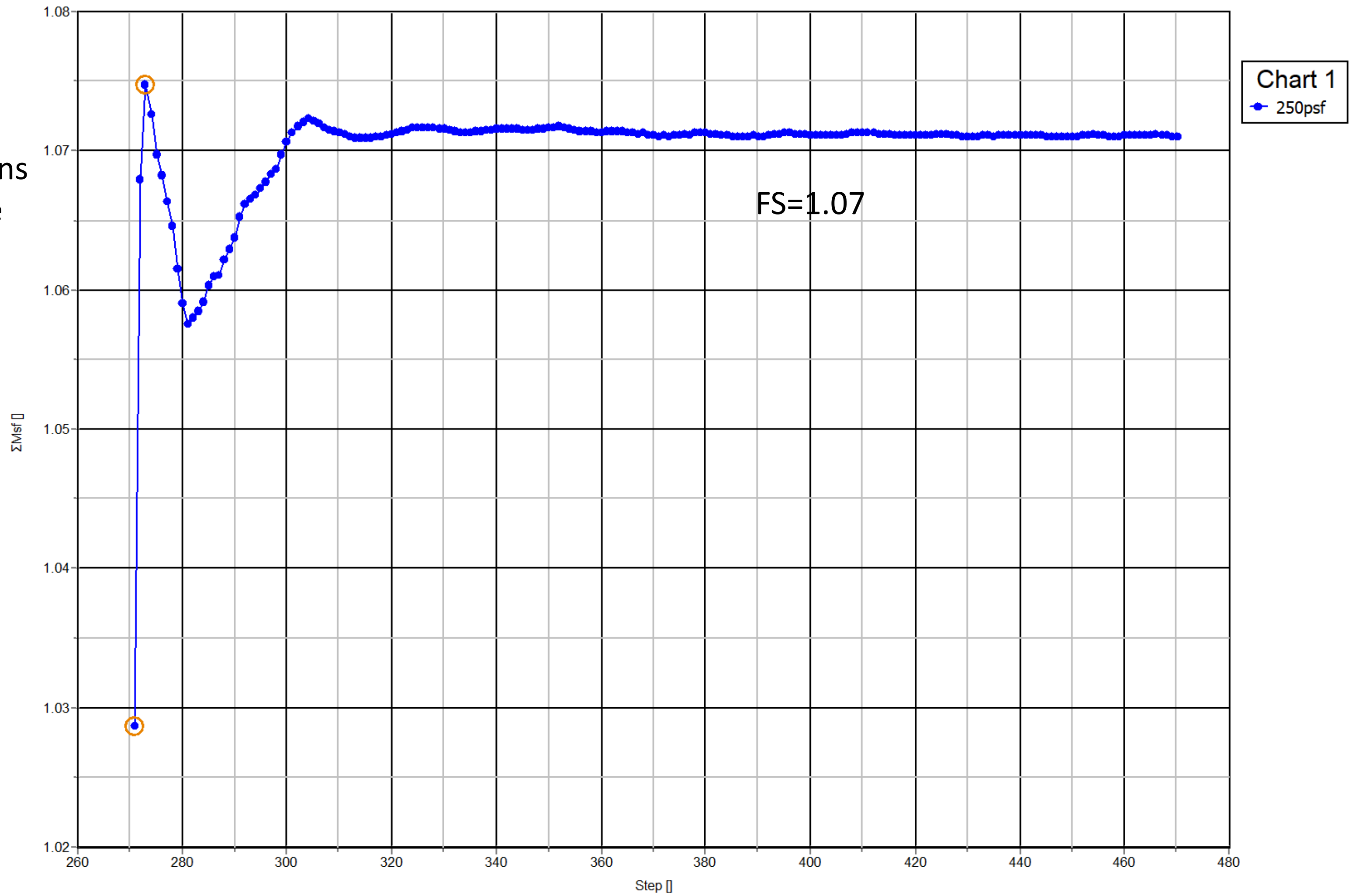


Incremental displacements |Δu| (scaled up 2.00*10³ times)

Maximum value = 4.992*10⁻³ ft (Element 2087 at Node 23535)

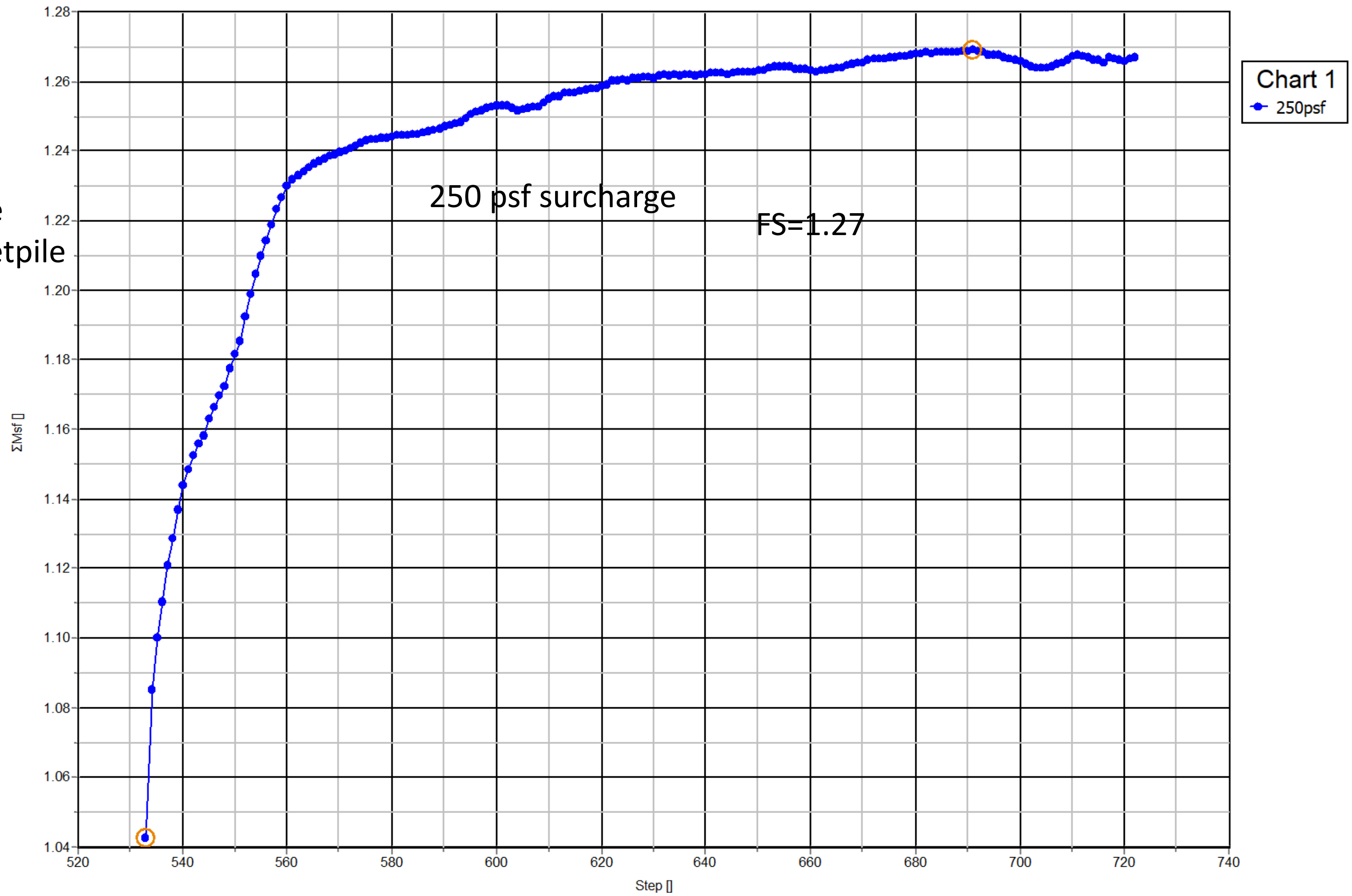
St 3+50

Existing Conditions
250psf surcharge
No sheetpiles



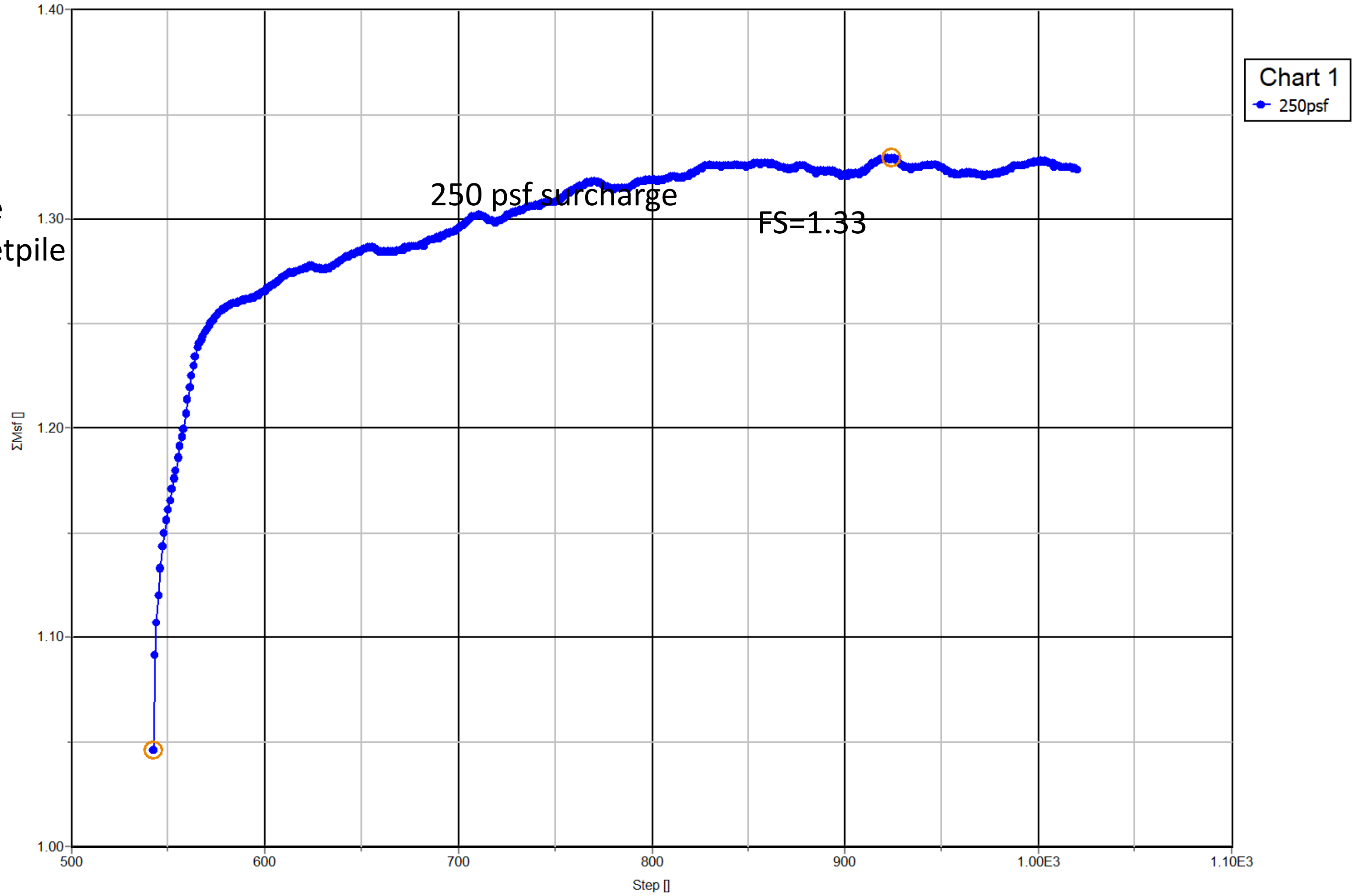
St 3+50

5ft ULFGA
250psf surcharge
10ft in sand sheetpile



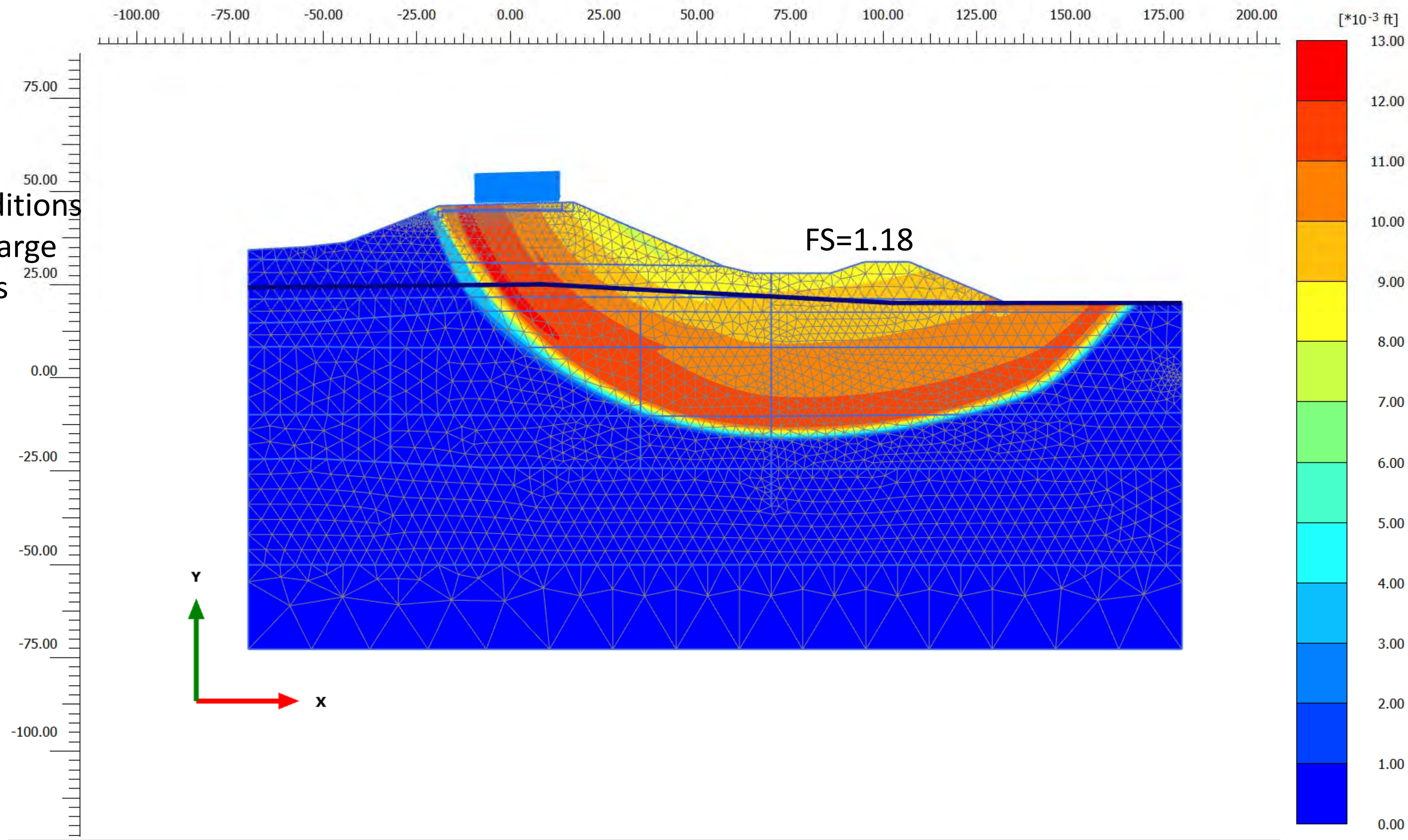
St 3+50

5ft ULFGA
250psf surcharge
15ft in sand sheetpile



St 4+00

Existing Conditions
250psf surcharge
No sheetpiles

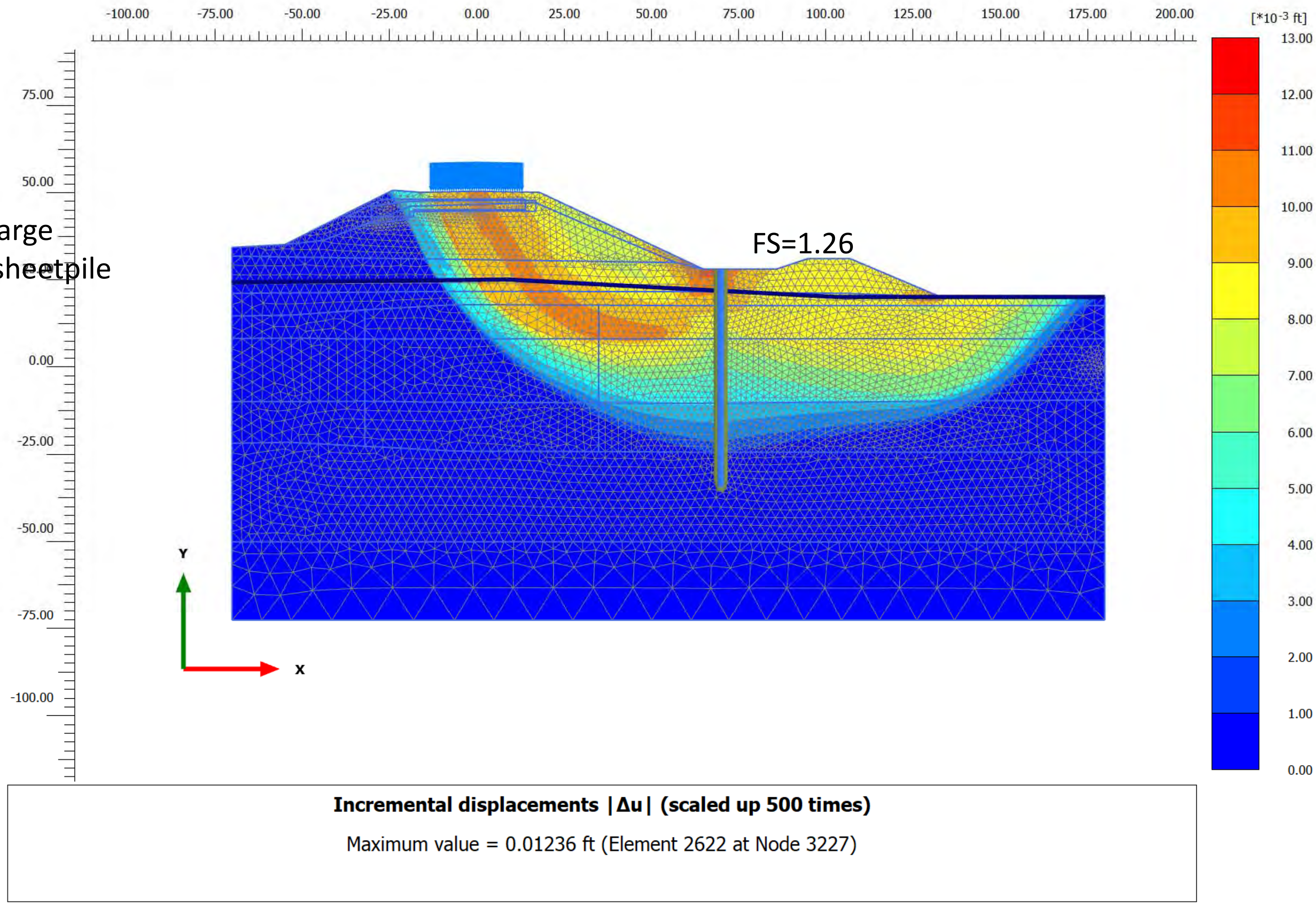


Incremental displacements |Δu| (scaled up 500 times)

Maximum value = 0.01241 ft (Element 868 at Node 34488)

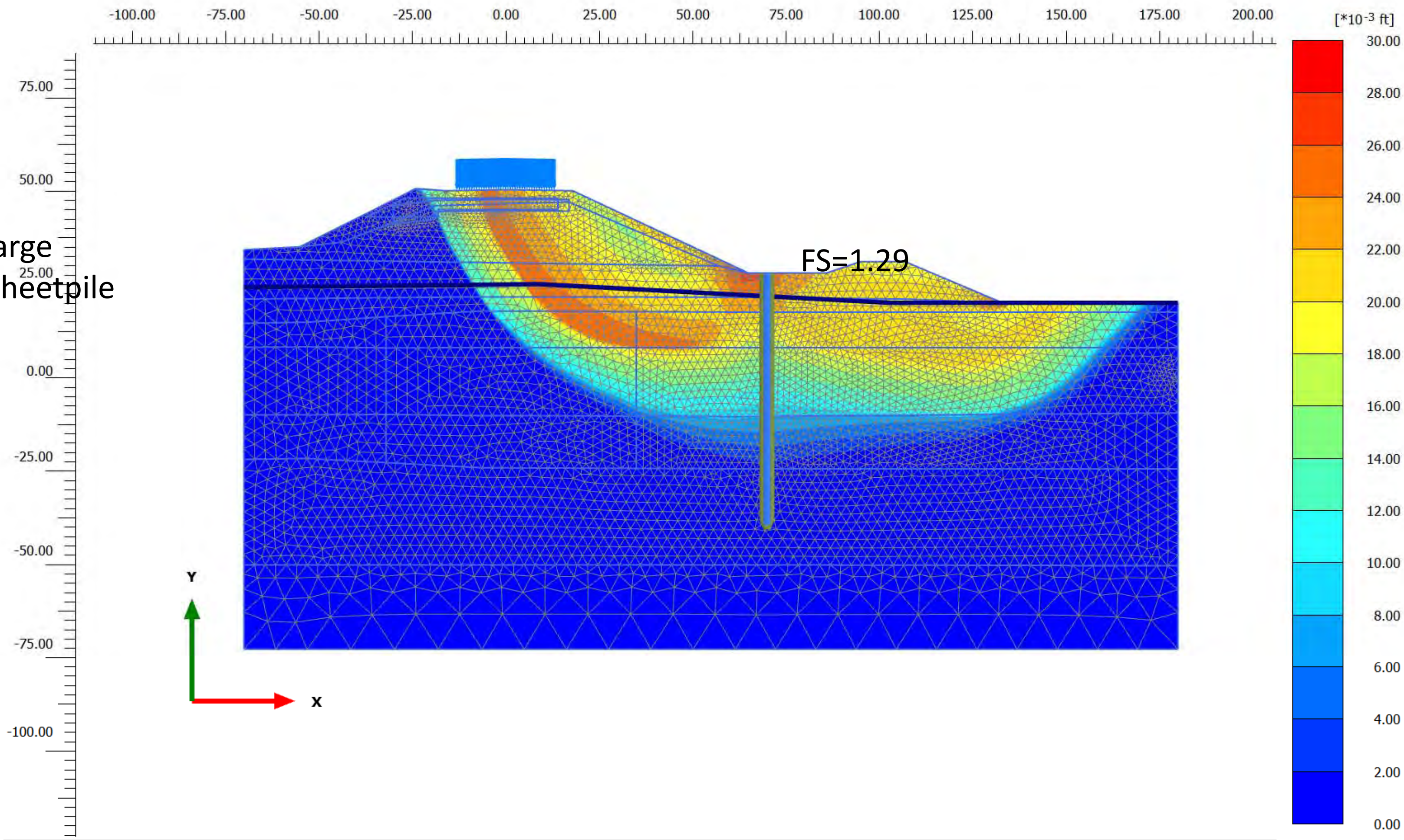
St 4+00

5ft ULFGA
250psf surcharge
10ft in sand sheetpile



St 4+00

5ft ULFGA
250psf surcharge
15ft in sand sheetpile

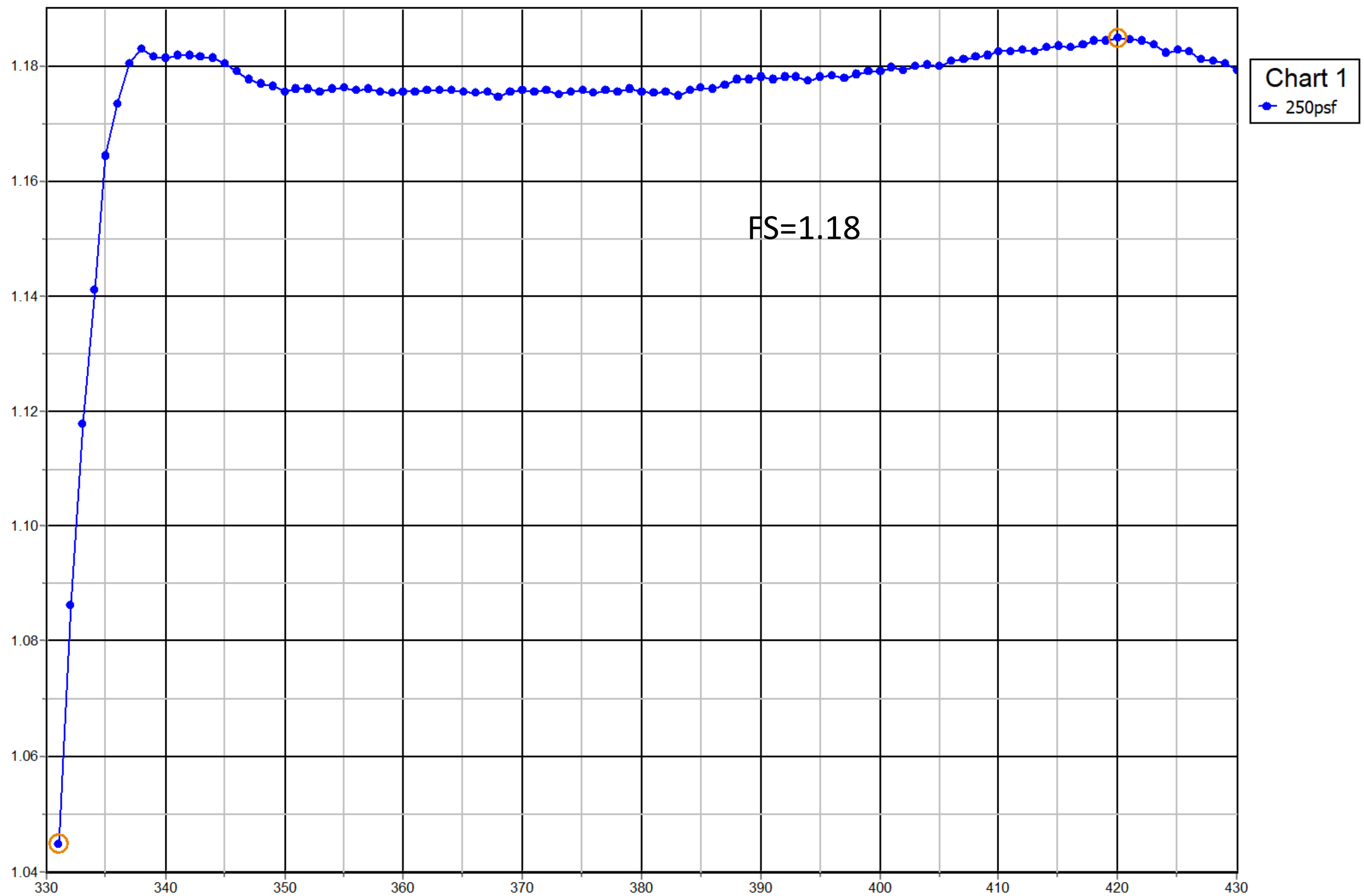


Incremental displacements |Δu| (scaled up 200 times)

Maximum value = 0.02867 ft (Element 2622 at Node 8833)

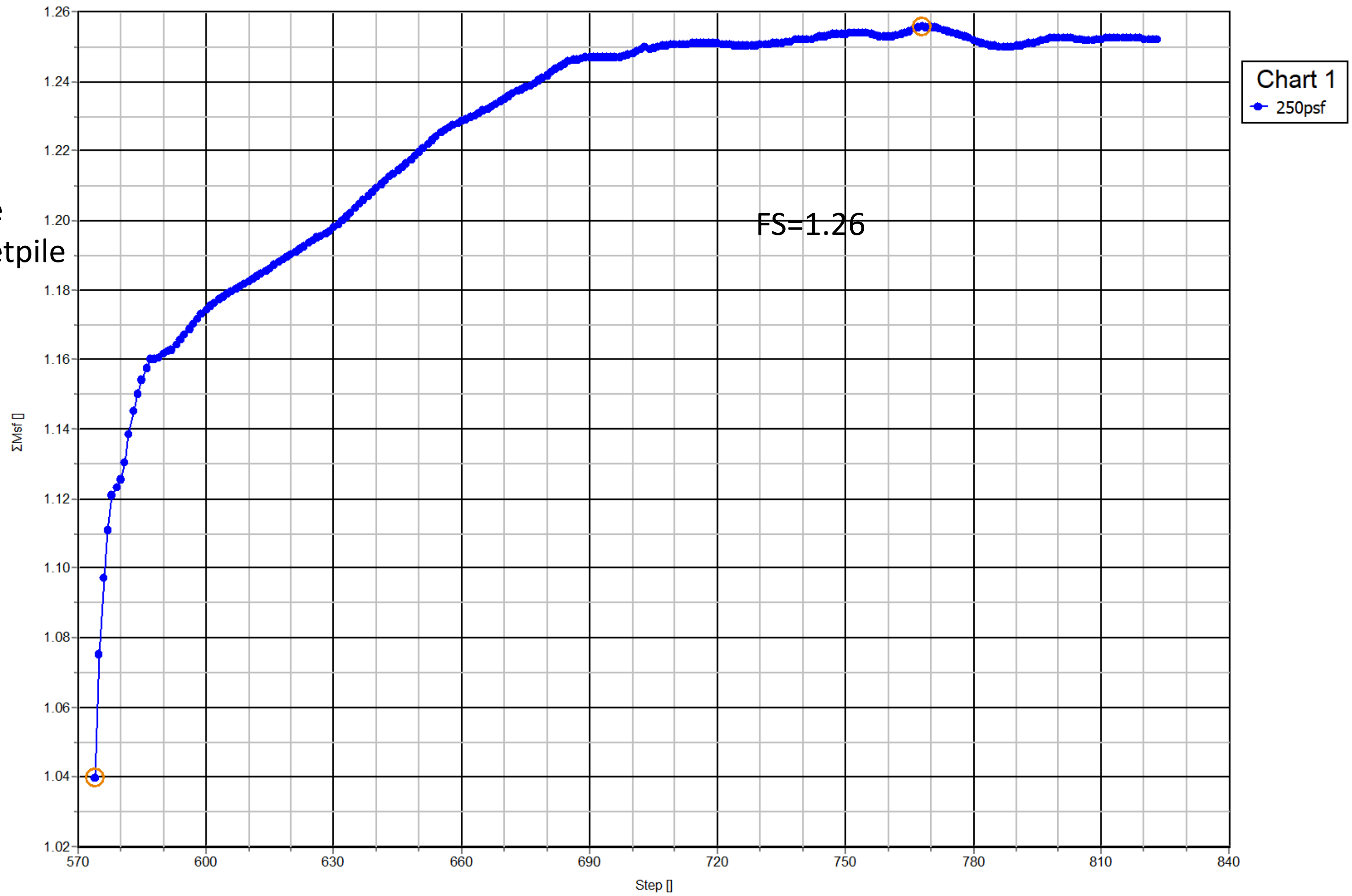
St 4+00

Existing Conditions
250psf surcharge
No sheetpiles



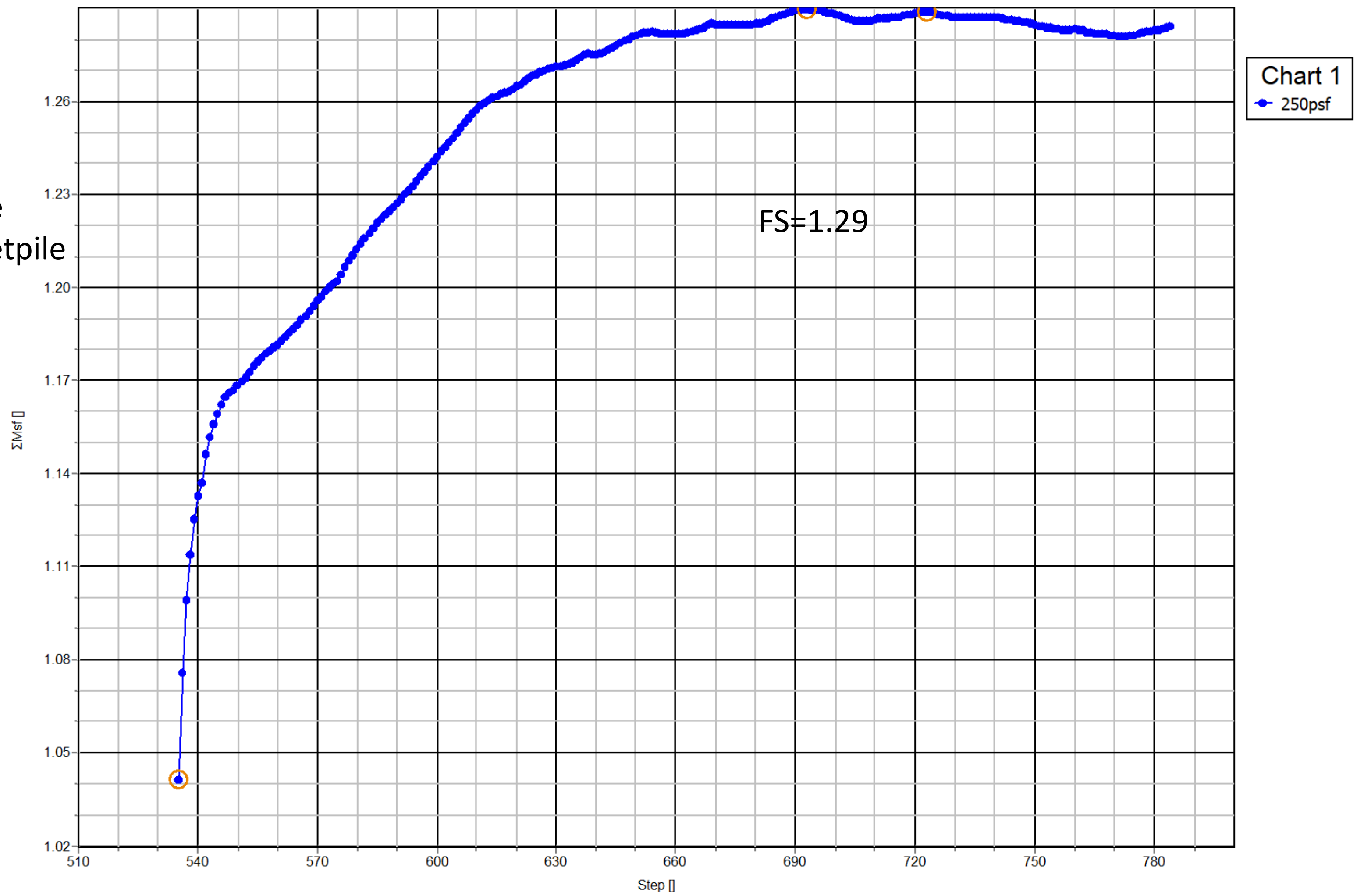
St 4+00

5ft ULFGA
250psf surcharge
10ft in sand sheetpile



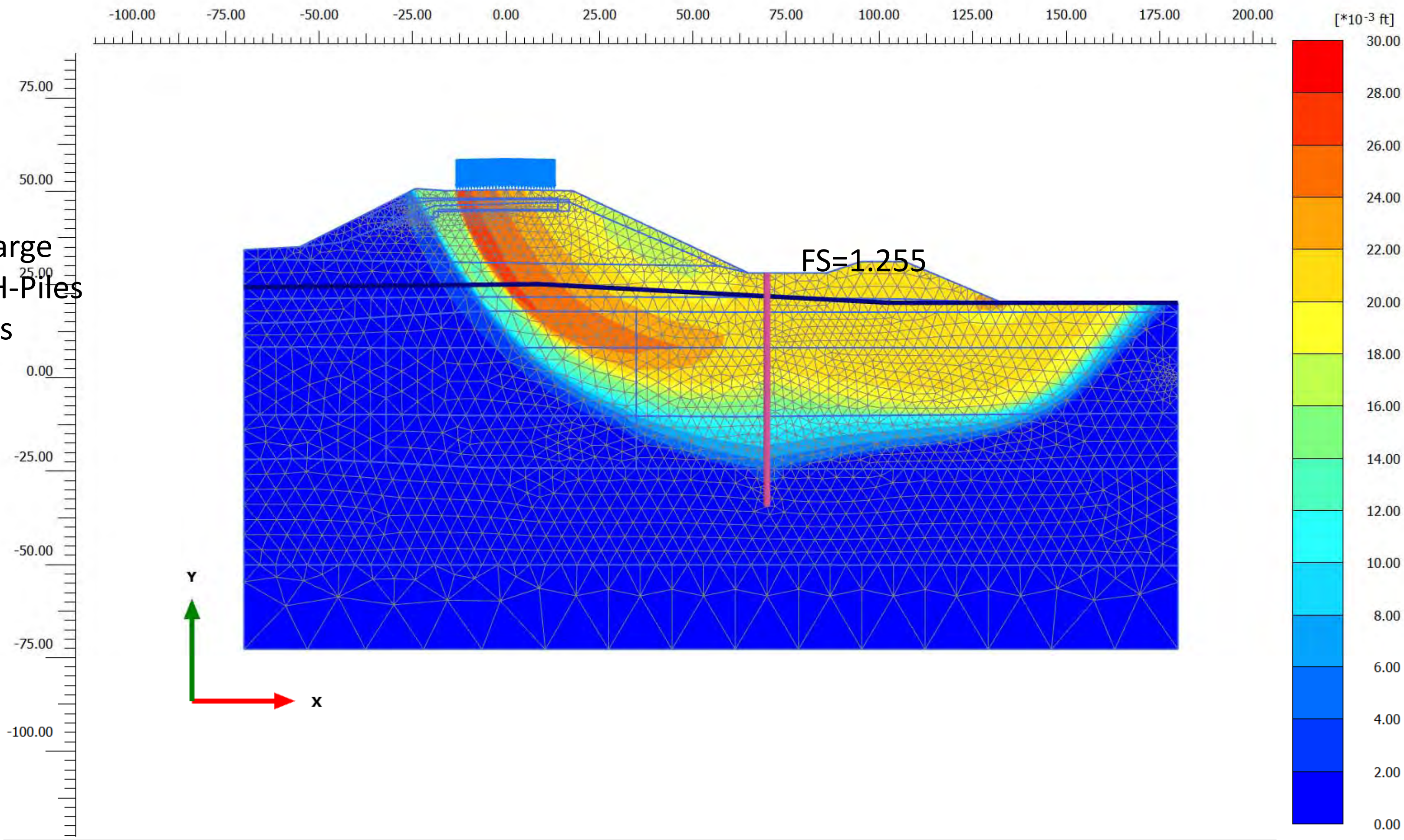
St 4+00

5ft ULFGA
250psf surcharge
15ft in sand sheetpile



St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
2.5 ft spacing

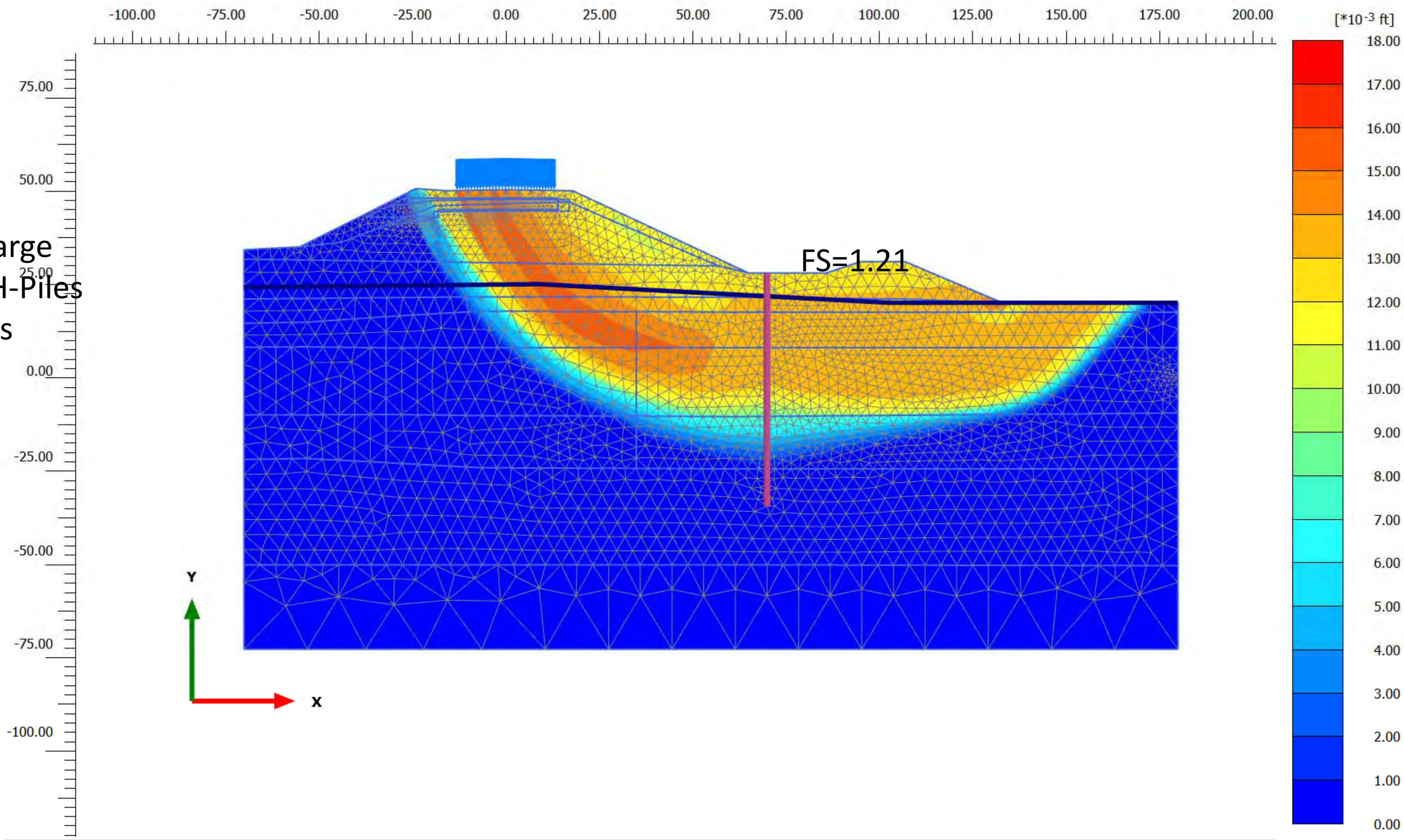


Incremental displacements |Δu| (scaled up 200 times)

Maximum value = 0.02864 ft (Element 1585 at Node 4073)

St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
5 ft spacing

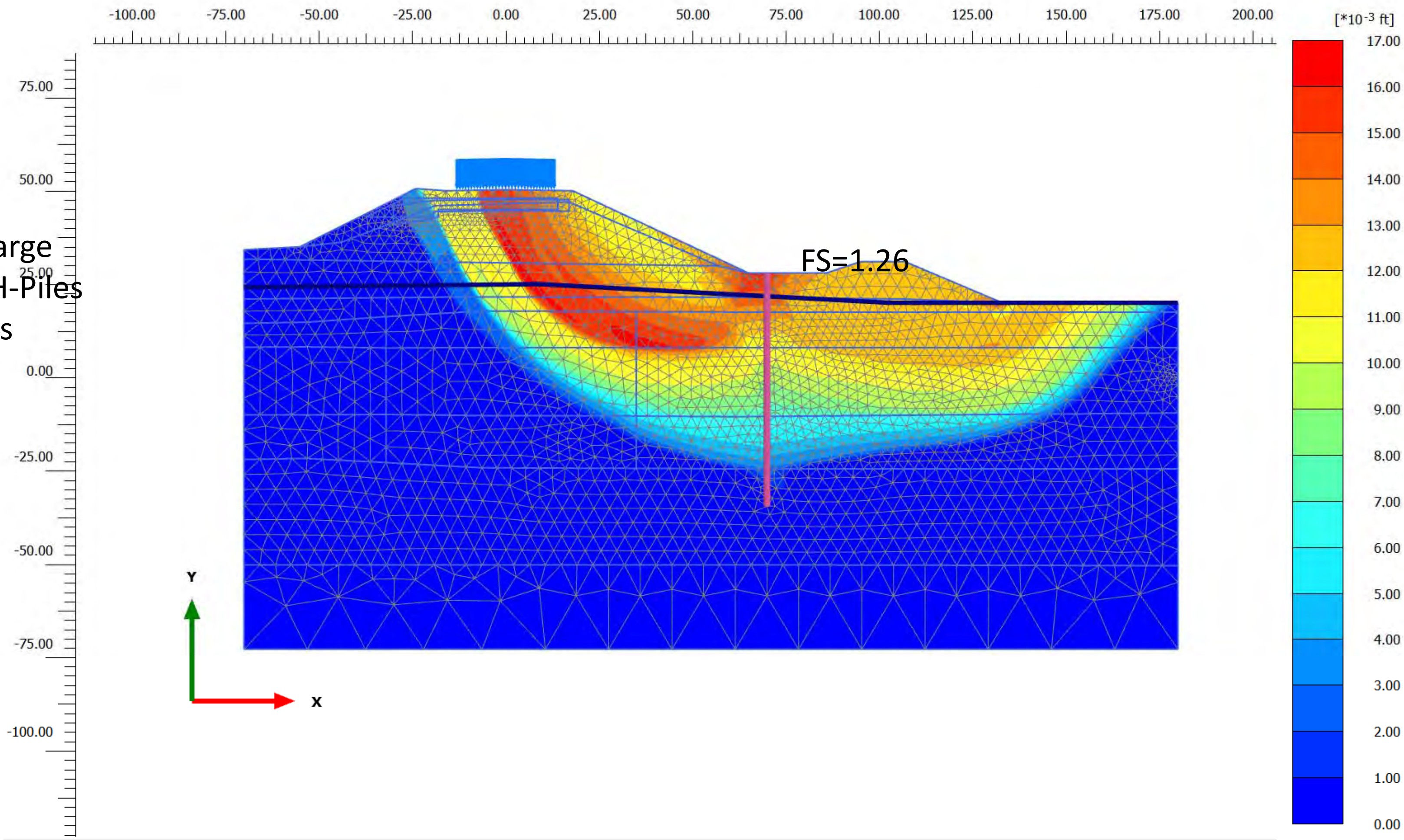


Incremental displacements |Δu| (scaled up 500 times)

Maximum value = 0.01723 ft (Element 1585 at Node 4073)

St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
2 ft spacing

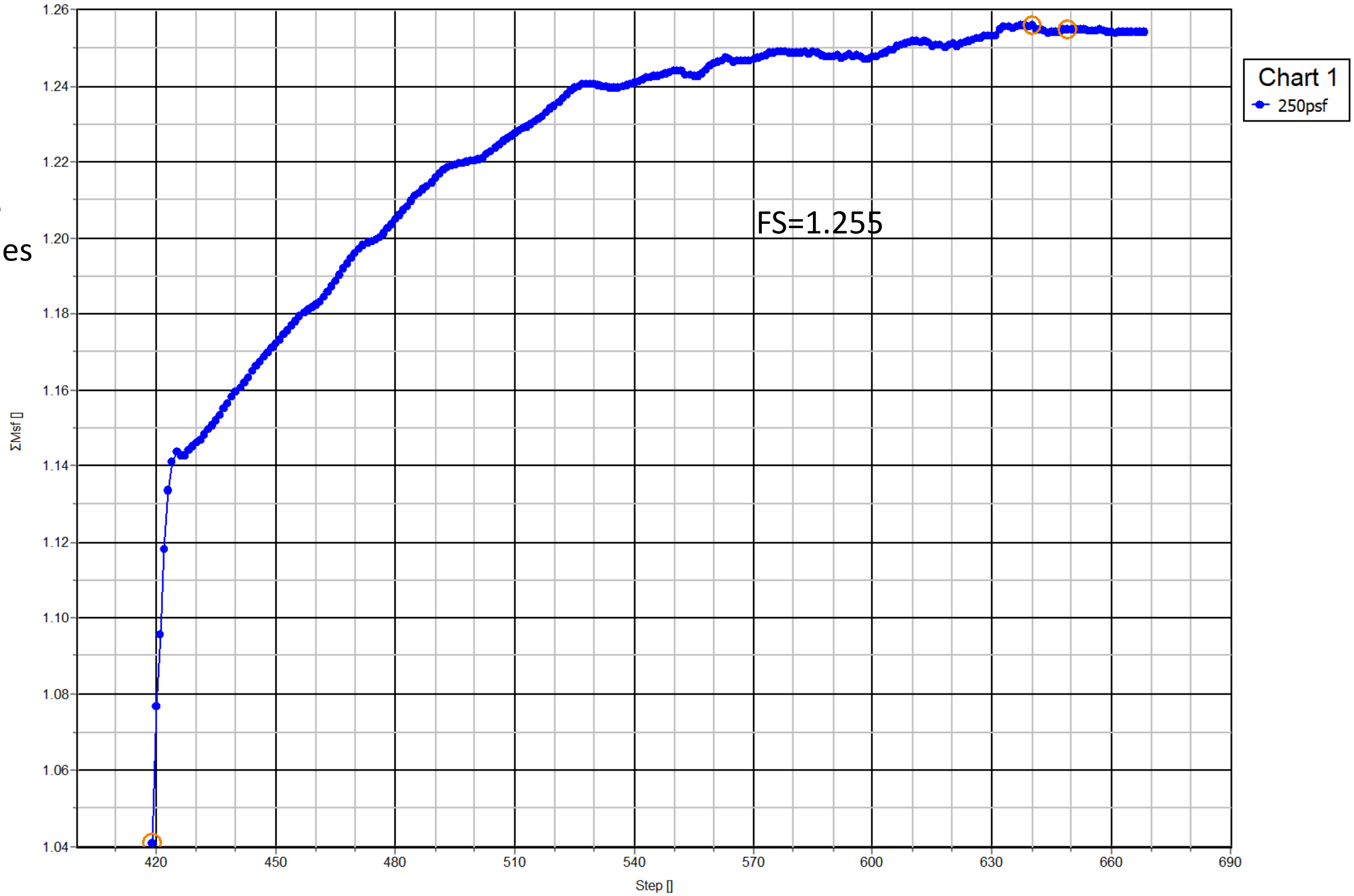


Incremental displacements |Δu| (scaled up 500 times)

Maximum value = 0.01654 ft (Element 761 at Node 32018)

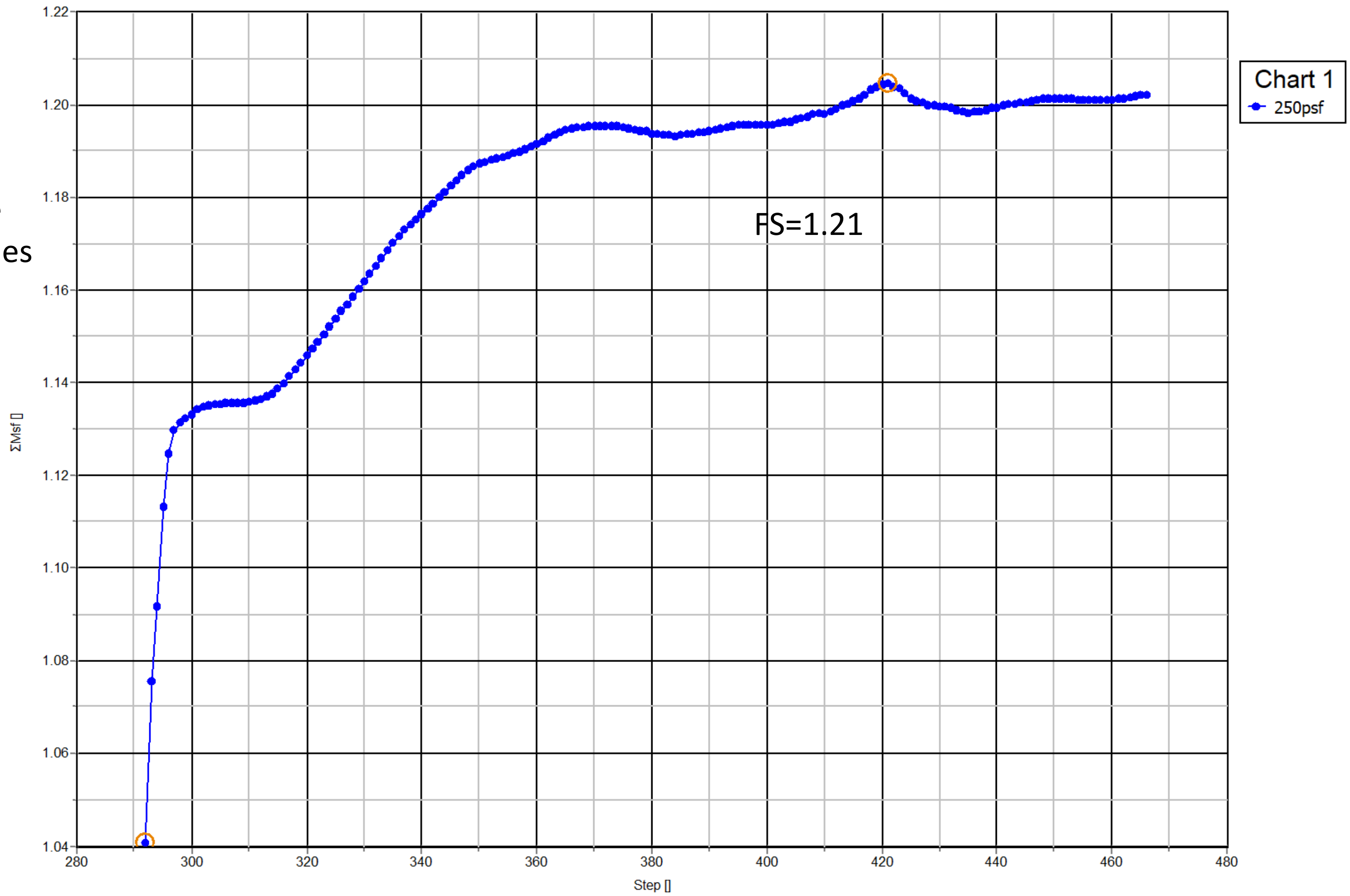
St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
2.5 ft spacing



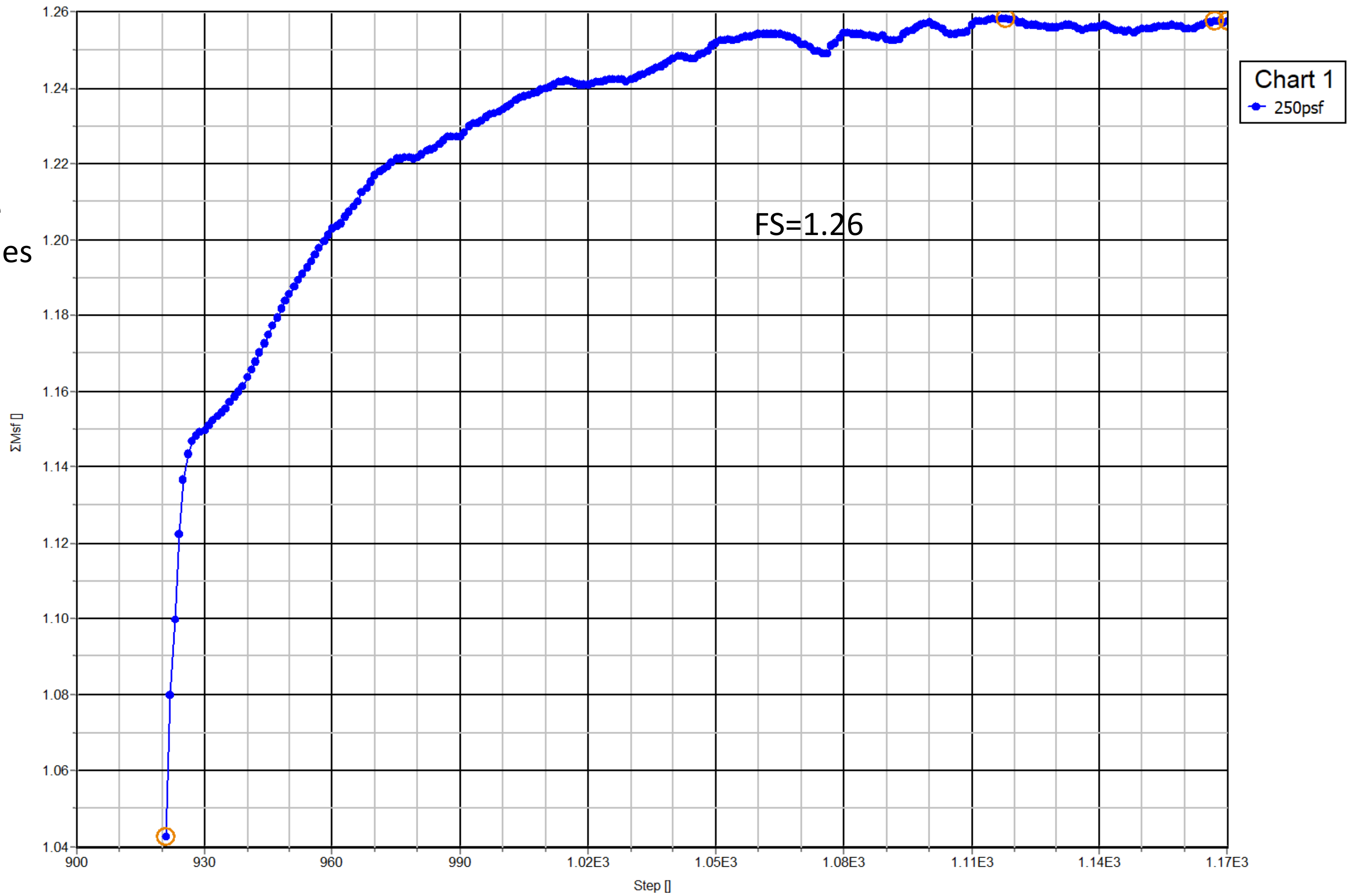
St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
5 ft spacing



St 4+00

5ft ULFGA
250psf surcharge
10ft in sand H-Piles
HP14*89 piles
2 ft spacing



Seismic

Seismic Site Class Calculation Summary

Project: Johnson Road Bridge over I-295 **Project No.:** 09.0026024.00

Location: Falmouth, ME

Evaluated By/Date: ENT **Date** 11/8/2021

Checked By/Date: ARB **Date** 2/22/2022

Objective:

Determine seismic site class by performing calculations in accordance with the MaineDOT Bridge Manual 2003 Edition with updates in 2018, which references the AASHTO LRFD Seismic Bridge Design Specifications, 9th Edition.

Subsurface Data: Cone Penetration Tests (CPTs) CPT-FJR-101 through -105 were conducted by Summit between May 28 and June 3, 2019. Downhole seismic testing was conducted at approximately 1-meter intervals in each CPT.

Approach:

1) Evaluate if the procedure in AASHTO LRFD Seismic Section 3.10.2.1 for classifying a site is appropriate for the site. Sites with highly variable subsurface conditions or very large sites may require multiple site class determinations or a site-specific seismic response analysis. Furthermore, classifying a site based on the 100 feet of soil and rock beneath the ground surface may be inappropriate if deep deposits of weak soils are present below 100 feet, or if foundation structures are supported on firm soil or rock below soft soils which can be justified as having little effect on the structure's seismic response.

2) Evaluate if soil properties are known in sufficient detail to determine site class. If data is not known in sufficient detail, AASHTO permits the use of Site Class D, unless conditions for Site Class E or Site Class F are likely to be present.

3) Check for the four categories of Site Class F requiring site-specific evaluation:
- Soils vulnerable to potential failure (liquefiable soils, sensitive clays, weakly cemented soils)
- Peats or highly organic clays greater than 10 feet in thickness
- Thick layers (greater than 25 feet) of highly plastic clay (PI > 75)
- Very thick soft/medium stiff clays (greater than 125 feet)

4) Check for existence of greater than 10 feet of soft clay (where s_u < 500 psf, *w* > 40%, and PI > 20). If these conditions are met, classify as Site Class E.

5) Categorize the site using one of the following three methods in AASHTO C3.10.3.1-1:
- $\overline{v}_s$ (Method A) - $\overline{N}$ (Method B) - $\overline{N}_{ch}$ *and* $\overline{s}_u$ (Method C)

If shear wave velocity data are available, they should be used to classify the site. The $\overline{N}$ and $\overline{s}_u$ methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with $\overline{v}_s$.

Results: Calculations of the Seismic Site Class based on Method A as described in section 3.10.3.1 of the LRFD Seismic Bridge Design Specifications are attached. Calculation results are summarized in the table below.

Boring ID	CPT-FJR-101	CPT-FJR-102	CPT-FJR-103	CPT-FJR-104	CPT-FJR-105
V_s	575	553	569	684	655

Conclusions: Based on the procedure outlined in section 3.10.3.1 and table 3.10.3.1-1 of the LRFD Seismic Bridge Design Specifications, since three of the deeper CPTs have an average shear wave velocity below 600 fps we recommend that Site Class "E" be used for design.

Johnson Road Bridge over I-295
Falmouth, ME

Calculated By: ENT Date: 11/8/2021
Checked By: ARB Date: 2/22/2022

INPUT

Exploration ID: **CPT-FJR-101** Ground Surface Elevation: 34.3 ft Depth of Boring: 59.5 ft

Depth to Bedrock: NE ARB

CALCULATION

$\overline{V}$ = 574.8

Soil Strata	CPT Interval Depth Top, ft	CPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Upper Marine Sand	10.2	11.2	23.6	960	12.4	0.01	
	13.6	14.6	20.2	675	15.8	0.02	
	17.0	18.0	16.8	980	19.3	0.02	
Marine Clay Crust	20.5	21.5	13.3	780	22.8	0.03	
	24.0	25.0	9.8	695	26.2	0.04	
Marine Clay	27.3	28.3	6.5	570	29.6	0.05	
	30.8	31.8	3.0	525	33.0	0.06	
	34.2	35.2	-0.4	485	36.4	0.07	
	37.5	38.5	-3.7	535	39.6	0.07	
	40.6	41.6	-6.8	500	42.9	0.09	
	44.1	45.1	-10.3	500	46.3	0.09	
	47.5	48.5	-13.7	625	49.5	0.08	
	50.5	51.5	-16.7	480	52.7	0.11	
	53.9	54.9	-20.1	630	56.1	0.09	
	57.2	58.2	-23.4	680	58.6	0.09	
Lower Marine Sand	58.9	59.9	-25.1	515	59.7	0.12	
Bottom of Boring	59.5						

Johnson Road Bridge over I-295
Falmouth, ME

Calculated By: ENT Date: 11/8/2021
Checked By: ARB Date: 2/22/2022

INPUT

Exploration ID: **CPT-FJR-102** Ground Surface Elevation: 29.0 ft Depth of Boring: 53.8 ft

Depth to Bedrock: NE ARB

CALCULATION

$\overline{V}$ = 552.8

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Upper Marine Sand	10.3	11.3	18.2	520	12.5	0.02	
Marine Clay Crust	13.6	14.6	14.9	540	15.9	0.03	
	17.1	18.1	11.4	485	19.3	0.04	
Marine Clay	20.5	21.5	8.0	400	22.7	0.06	
	23.9	24.9	4.6	425	26.3	0.06	
	27.7	28.7	0.8	570	29.9	0.05	
	31.0	32.0	-2.5	450	33.2	0.07	
	34.3	35.3	-5.8	505	36.5	0.07	
	37.7	38.7	-9.2	515	39.9	0.08	
	41.0	42.0	-12.5	625	43.2	0.07	
	44.3	45.3	-15.8	500	46.5	0.09	
	47.7	48.7	-19.2	570	49.8	0.09	
	50.9	51.9	-22.4	555	52.6	0.09	
Lower Marine Sand	53.2	54.2	-24.7	1340	54.0	0.04	
Bottom of Boring	53.8						

Johnson Road Bridge over I-295
Falmouth, ME

Calculated By: ENT Date: 11/8/2021
Checked By: ARB Date: 2/22/2022

INPUT

Exploration ID: CPT-FJR-103 Ground Surface Elevation: 36.8 ft Depth of Boring: 65.1 ft

Depth to Bedrock: NE ARB

CALCULATION

$\overline{V}$ = 568.8

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Fill	10.1	11.1	26.2	770	12.4	0.02	
Upper Marine Sand	13.6	14.6	22.7	590	16.4	0.03	
	18.2	19.2	18.1	400	20.4	0.05	
Marine Clay Crust	21.6	22.6	14.7	690	23.9	0.03	
	25.1	26.1	11.2	550	27.3	0.05	
Marine Clay	28.5	29.5	7.8	475	30.7	0.06	
	31.9	32.9	4.4	515	34.1	0.07	
	35.2	36.2	1.1	465	37.4	0.08	
	38.6	39.6	-2.3	580	40.8	0.07	
	42.0	43.0	-5.7	490	44.2	0.09	
	45.4	46.4	-9.1	595	47.6	0.08	
	48.7	49.7	-12.4	595	50.6	0.08	
	51.4	52.4	-15.1	450	53.5	0.12	
	54.6	55.6	-18.3	555	57.1	0.10	
Lower Marine Sand	58.5	59.5	-22.2	600	60.9	0.10	
	62.3	63.3	-26.0	910	64.0	0.07	
	64.6	65.6	-28.3	670	65.35	0.10	
Bottom of Boring	65.1						

Johnson Road Bridge over I-295
Falmouth, ME

Calculated By: ENT   Date: 11/8/2021
Checked By: ARB   Date: 2/22/2022

INPUT

Exploration ID: **CPT-FJR-104**   Ground Surface Elevation: 47.1 ft   Depth of Boring: 105.4 ft

Depth to Bedrock: NE ARB

CALCULATION

$\overline{V}$ = 684.0

Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Fill	10.0	11.0	36.6	995	12.3	0.01	
	13.6	14.6	33.0	510	17.2	0.03	
	19.8	20.8	26.8	750	22.0	0.03	
Upper Marine Sand	23.1	24.1	23.5	650	25.3	0.04	
	26.5	27.5	20.1	715	28.7	0.04	
	29.8	30.8	16.8	630	32.1	0.05	
Marine Clay Crust	33.3	34.3	13.3	725	35.5	0.05	
	36.6	37.6	10.0	615	38.9	0.06	
Marine Clay	40.1	41.1	6.5	550	42.3	0.08	
	43.4	44.4	3.2	485	45.6	0.09	
	46.7	47.7	-0.1	475	49.0	0.10	
	50.2	51.2	-3.6	505	52.4	0.10	
	53.6	54.6	-7.0	545	55.8	0.10	
	56.9	57.9	-10.3	535	59.2	0.11	
	60.4	61.4	-13.8	530	62.5	0.12	
	63.6	64.6	-17.0	610	65.8	0.11	
	67.0	68.0	-20.4	565	69.2	0.12	
	70.4	71.4	-23.8	565	72.6	0.13	
Lower Marine Sand	73.8	74.8	-27.2	620	76.0	0.12	
	77.2	78.2	-30.6	650	79.4	0.12	
	80.6	81.6	-34.0	790	82.8	0.10	
	84.0	85.0	-37.4	745	86.2	0.12	
	87.4	88.4	-40.8	760	89.5	0.12	
	90.5	91.5	-43.9	715	92.7	0.13	
	93.9	94.9	-47.3	880	96.0	0.11	
	97.0	98.0	-50.4	1105	99.3	0.09	
	100.5	101.5	-53.9	850	102.6	0.12	
	103.7	104.7	-57.1	880	104.8	0.12	
	104.9	105.9	-58.3	1092	105.7	0.10	
Bottom of Boring	105.4						

Johnson Road Bridge over I-295
Falmouth, ME

Calculated By: ENT Date: 11/8/2021
Checked By: ARB Date: 2/22/2022

INPUT

Exploration ID: **CPT-FJR-105** Ground Surface Elevation: 46.1 ft Depth of Boring: 21.3 ft

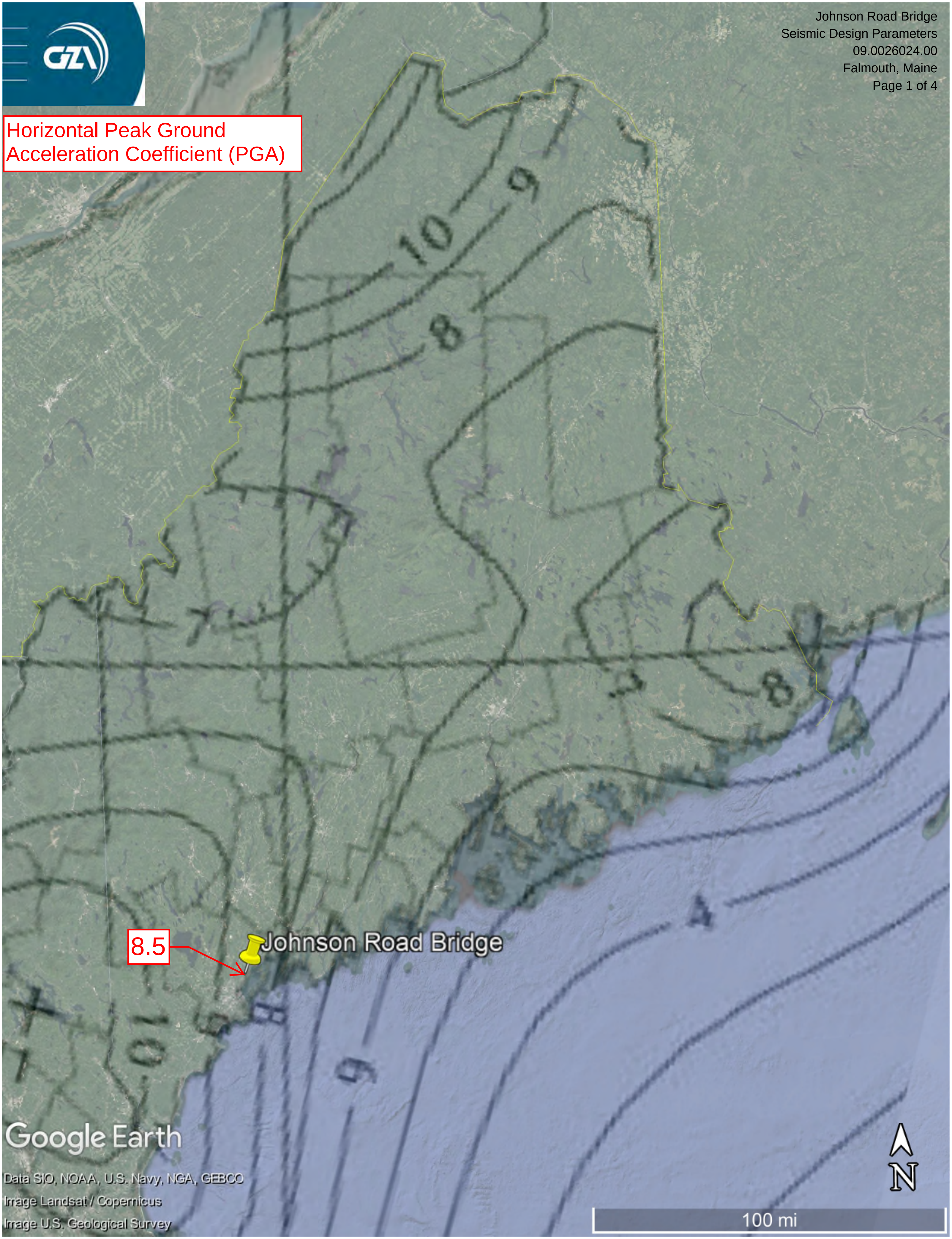
Depth to Bedrock: NE ARB

CALCULATION

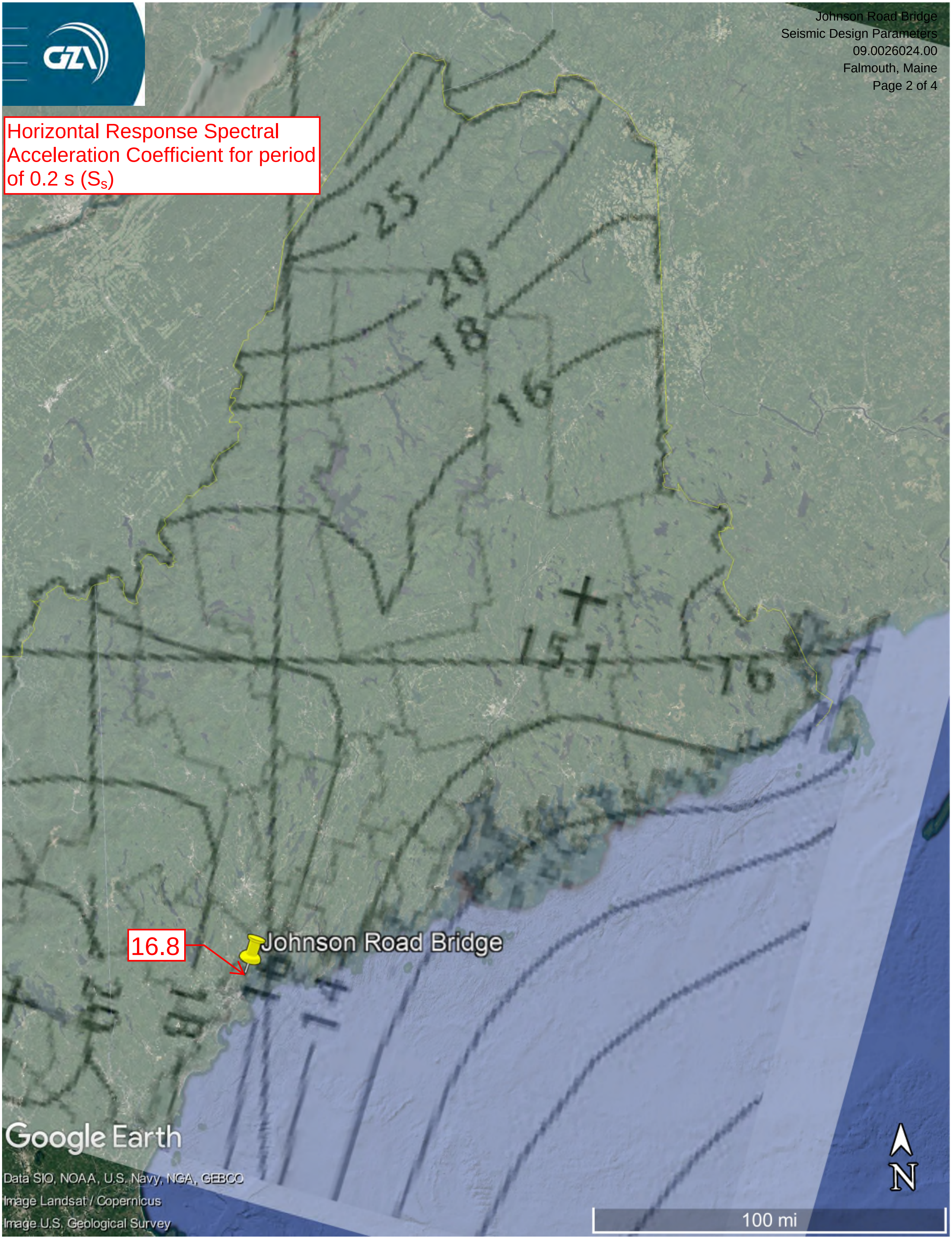
$\overline{V}$ = 654.7

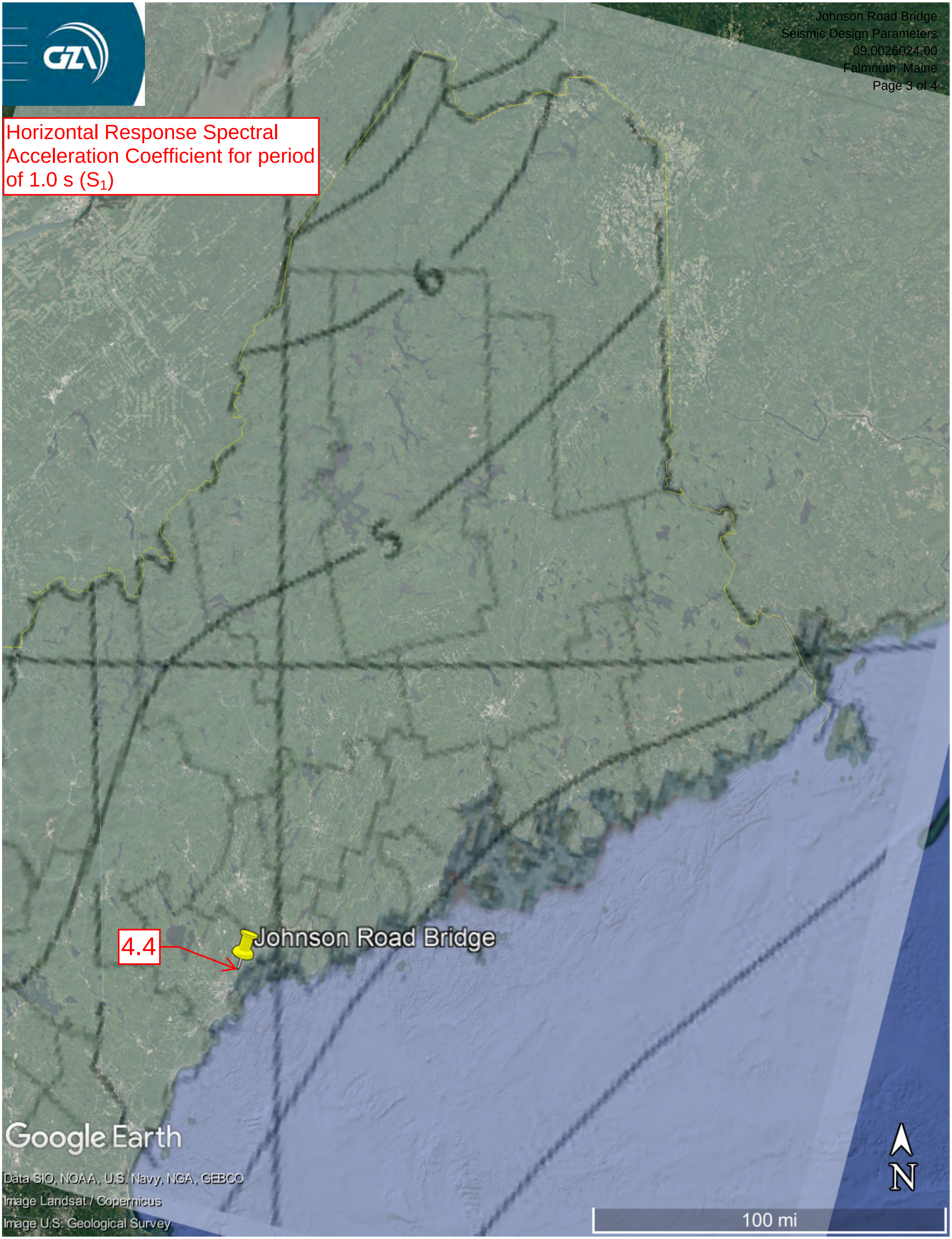
Soil Strata	CPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	V_s	d_i	d_i / V_s	Comment
Marine Clay Crust	10.5	11.5	35.1	670	12.7	0.02	
Marine Clay	13.9	14.9	31.7	485	16.1	0.03	
	17.3	18.3	28.3	590	19.5	0.03	
Lower Marine Sand	20.6	21.6	25.0	1005	21.5	0.02	
Bottom of Boring	21.3						

Horizontal Peak Ground Acceleration Coefficient (PGA)



Horizontal Response Spectral Acceleration Coefficient for period of 0.2 s (S_S)




Horizontal Response Spectral Acceleration Coefficient for period of 1.0 s (S_1)
6
5
4.4
Johnson Road Bridge
Google Earth
Data SIO, NOAA, U.S. Navy, NGA, GEBCO
Image Landsat / Copernicus
Image U.S. Geological Survey
100 mi
N

Johnson Road Bridge Seismic Interpolation for Coefficients		
Seismic Parameter	**Interpolated Value from Maps[1]**	**Design Parameter**
Horizontal Peak ground Acceleration Coefficient	8.5	$PGA = .085$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	16.8	$S_s = 0.168$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.4	$S_1 = .044$

Notes: 1. AASHTO Figures 3.10.2.1-1,-2, and -3 were overlaid within the Google Earth software. Coefficients were interpolated between lines on these figures as presented in pages 1 through 3 of this calculation.

For Class E, values of F_{PGA} and F_a = 2.5, and F_v = 3.5

Therefore:

$$A_s = F_{PGA} \times PGA = 2.5 \times 0.085 = 0.21\ g$$

$$S_{DS} = F_a \times S_s = 2.5 \times 0.168 = 0.42\ g$$

$$S_{D1} = F_v \times S_1 = 3.5 \times 0.044 = 0.15\ g$$

Summary:

SITE CLASS E SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	2.5
Fa	2.5
Fv	3.5
As (Period = 0.0 sec)	0.21 g
SDs (Period = 0.2 sec)	0.42 g
SD1 (Period = 1.0 sec)	0.15 g

Downdrag

Objective

Evaluate the Downdrag under the proposed grade raises and alignment configura<on at Abutment 1, using finite element modeling and the methodology developed by Fellenius (2016).

References

1. Tan, S. A., &; Fellenius, B. H. (2016). Nega<ve skin fric<on pile concepts with soil structure interac<on. Geotechnical Research, 3(4), 137147. hE ps://doi.org/10.1680/jgere.16.00006

Methodology

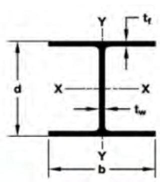
GZA used finite element methods (Plaxis 2D) to evaluate downdrag using the methodology developed by Fellenius[1] to calculate downdrag load applied to the pile by seE ling soil around its perimeter using FEM considering the neutral plane concept. The neutral plane occurs along a deep founda<on element at the depth where soil and pile seE lements are equal, resul<ng in zero mobilized side resistance at that loca<on. Above the neutral plane, seE lement of the surrounding soil develops nega<ve side resistance and transfer of downdrag axial compression load to the pile. The primary difference between the FEM approach and the conven<onal β-method approach is that the FEM method is able to consider load displacement behavior, and only generate downdrag consistent with the level of strain between the pile and the soil.

GZA developed an axisymmetric model in Plaxis2D with the design soil profile described previously for Abutment 1. In the axisymmetric model, the model space corresponds to one quadrant of the actual geometry.

1. Pile Properties

The axisymmetric model in Plaxis2D cannot effec<vely model the complex shape of an H-pile. Therefore, a pile equivalent to an HP14x117 was developed using a perimeter length equal to an HP14x117 perimeter and a cylindrical beam element with a reduced elas<c modulus (E) so that the product of pile area and modulus (EA) would be equal to an HP14x117 pile, which is required for representa<ve elas<c pile deforma<on behavior in the model. The soil-pile interface u<lized Mohr-Coulomb soil proper<es that would allow development of typical ul<mate shear strength values with sufficient shear displacement, which is consistent with assump<ons when calcula<ng nominal geotechnical resistance.

HP14x117 Properties



SECTION	Weight	Area	Depth d	Flange Width b	THICKNESS Flange (t_f)	Web (t_w)	Coating Area	PROPERTIES AXIS X-X I	S	Z	r	AXIS Y-Y I	S	Z	r
	lb/ft	in²	in	in	in	in	ft²/ft	in⁴	in³	in³	in	in⁴	in³	in³	in
HP 14 (HP 360)	73 (108)	21.4 (139)	13.60 (346)	14.60 (370)	0.505 (12.8)	0.505 (12.8)	6.96 (2.15)	729 (30300)	107 (1750)	118 (1940)	5.84 (14.8)	261 (10800)	35.8 (585)	54.6 (891)	3.49 (8.86)
	89 (132)	26.1 (168)	13.80 (351)	14.70 (373)	0.615 (15.6)	0.615 (15.6)	7.02 (2.16)	904 (37500)	131 (2140)	146 (2390)	5.88 (14.9)	326 (13500)	44.3 (724)	67.7 (1110)	3.53 (8.96)
	102 (152)	30.1 (194)	14.00 (356)	14.80 (376)	0.705 (17.9)	0.705 (17.9)	7.06 (2.18)	1050 (43900)	150 (2470)	169 (2770)	5.92 (15.0)	380 (15900)	51.4 (845)	78.8 (1290)	3.56 (9.05)
	117 (174)	34.4 (222)	14.20 ([illegible])	14.90 ([illegible])	0.805 (20.4)	0.805 (20.4)	7.12 ([illegible])	1220 ([illegible])	172 ([illegible])	194 ([illegible])	5.96 ([illegible])	443 ([illegible])	59.5 (973)	91.4 ([illegible])	3.59 (9.11)

$$E = 29000\ ksi = 4.176 \times 10^9 psf$$

$$\sigma_y = 50ksi$$

$$EA = (4.176 \times 10^9)\left(\frac{34.4}{144}\right) = 997.6 \times 10^6 \text{ lb}$$

$$EI = (4.176 \times 10^9)\left(\frac{1220}{144 \times 144}\right) = 245.69 \times 10^6\ lb.ft^2$$

$$Mp = S\sigma_y = \frac{172}{144 \times 12} \times (60000 \times 144) = 860 \times 10^3 lb.ft$$

$$Np = 60000 \times 34.4 = 2.064 \times 10^6 lb$$

- Use a normalized cylindrical section to represent HP14x117 pile
- New section to have the same perimeter $=2b + 2d = (2)(14.9) + (2)(14.2) = 58.2\ in$
- Equivalent Section: $2\pi R = 58.2\ \Rightarrow R = 9.26\ in = \mathbf{0.772\ ft}$

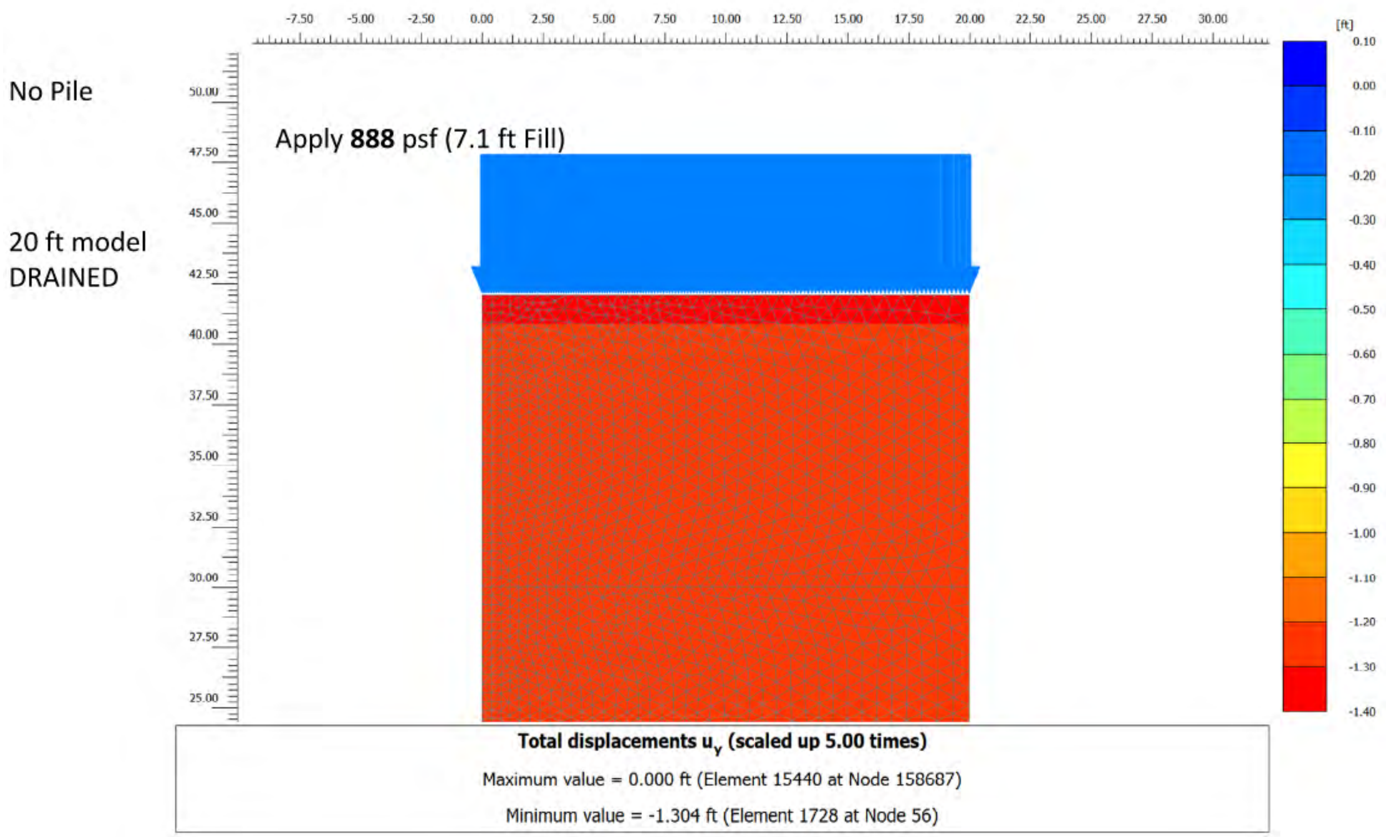
- To keep the axial stiffness (EA) the same, E in Plaxis model is reduced
- Equivalent Section: $A_{eq} = \pi R^2 = \pi \times 9.26^2 = 269.38\ in^2 = 1.87\ ft^2$
- E reduction ratio $= \frac{A}{A_{eq}} = \frac{34.4}{269.38} = 0.1277$
- $E_{eq} = 29000 ksi \times 0.1277 = 3703.3\ ksi = \mathbf{533.28 \times 10^3 ksf}$
- $\gamma_{eq} = 490\ pcf \times 0.1277 = \mathbf{62.57\ pcf}$

2. Analysis

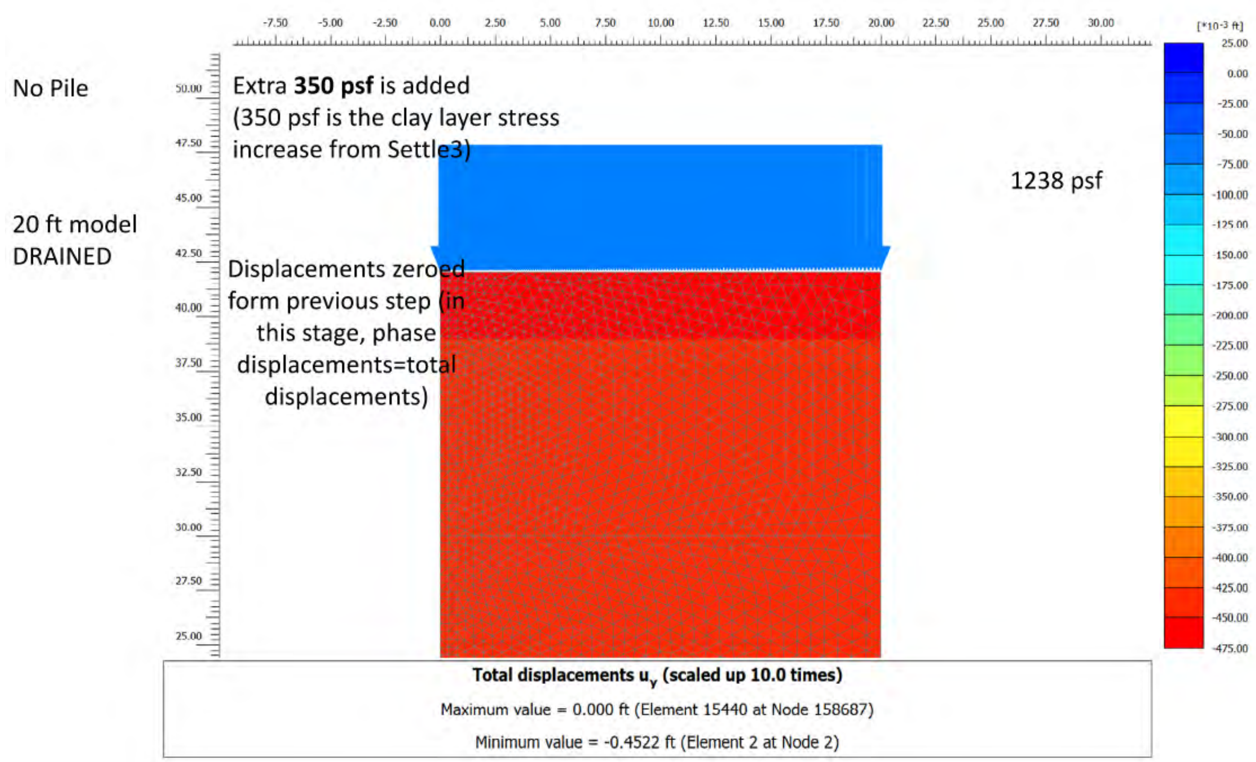
Two pile models were developed for the analysis. The first model was evaluated excluding the pile ("No-Pile model") in order to calibrate the an<cipated seE lements to achieve 5.4 inches in 75 years as predicted by the seE lement evalua<ons for the project. The Plaxis soil cons<tui<ve model did not include secondary compression, which is the primary cause of downdrag for the project. Therefore, the values of CR and RR were increased in the no pile model un<l the es<mated seE lement was shown. This produced appropriate soil-pile differen<al movement and neutral plane loca<on that would develop downdrag under an applied load. The second model ("Pile Model") includes the proposed pile and incorporates the modified primary consolida<on proper<es from the first model. The Plaxis modeling procedure is as follows.

No-Pile Model:

Step 1. Apply exis<ng fill material (from top of pile to exis<ng grade) as 888 psf of surcharge load on the pile top eleva<on.



Step 2. Add addi<onal 350-psf surcharge pressure, equivalent to the stress increase in the marine clay from SeE le3 analyses.



Step 3. Modify CR/RR values in Plaxis2D to match with clay seE lement over the design life of the bridge. The figure above shows the final itera<on of the model with 0.4522 L (5.4 inches) of seE lement. The ini<al and final consolida<on proper<es used to calibrate the model are shown below. The primary consolida<on values used to generate the total seE lement (including secondary) are increased to CR = 0.78 and RR = 0.065.

Pile Model:

Step 1. Apply exis<ng fill material (from pile top eleva<on to exis<ng ground surface) as 888 psf of surcharge load on the ground surface at the pile top eleva<on. Add pile element to the model and apply the maximum factored load (410 kips) to the pile beam element. Reset displacement in the model.

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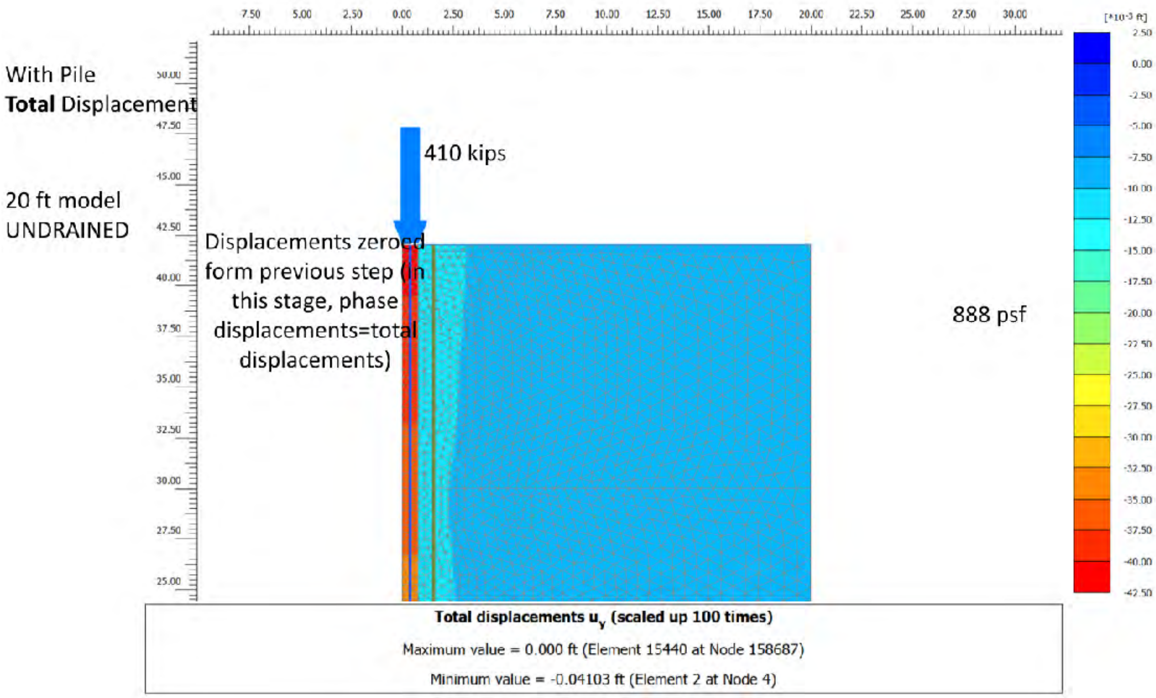
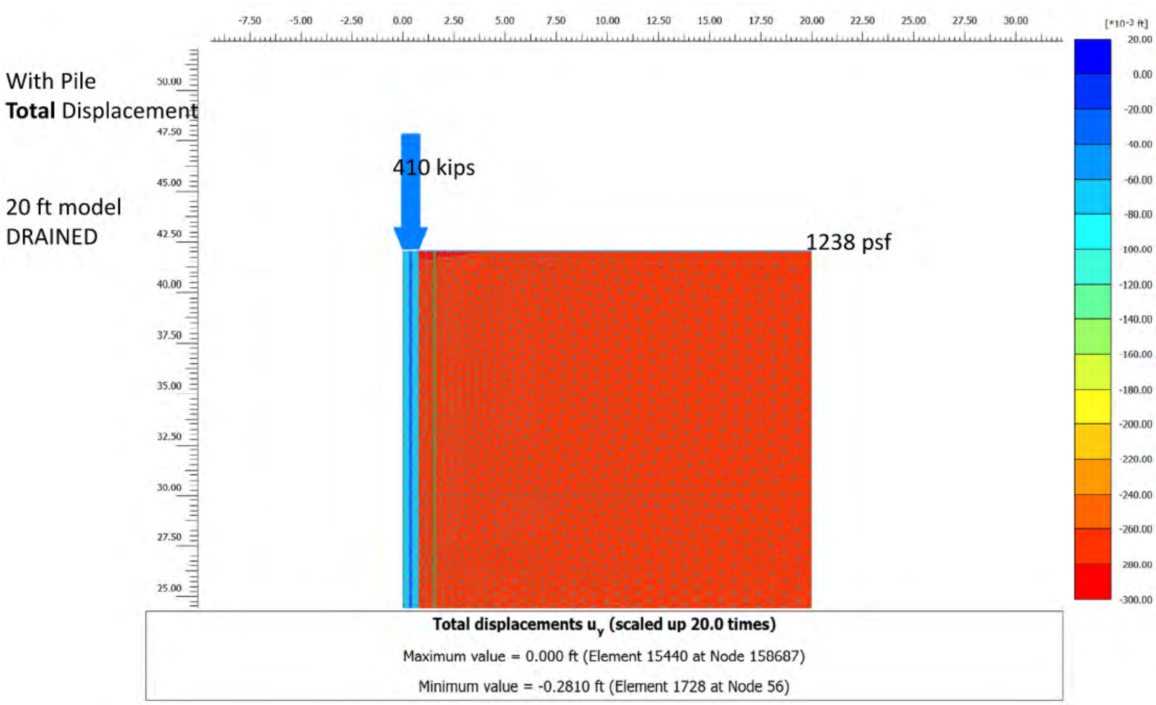
Engineers and Scientists

JOB: 09.0026024.00 Johnson Road Bridge
SUBJECT: Finite Element-Based Downdrag
SHEET: 4 OF 6
CALCULATED BY B. Cardali, 2/28/22
REVIEWED BY A. Blaisdell, 3/1/22

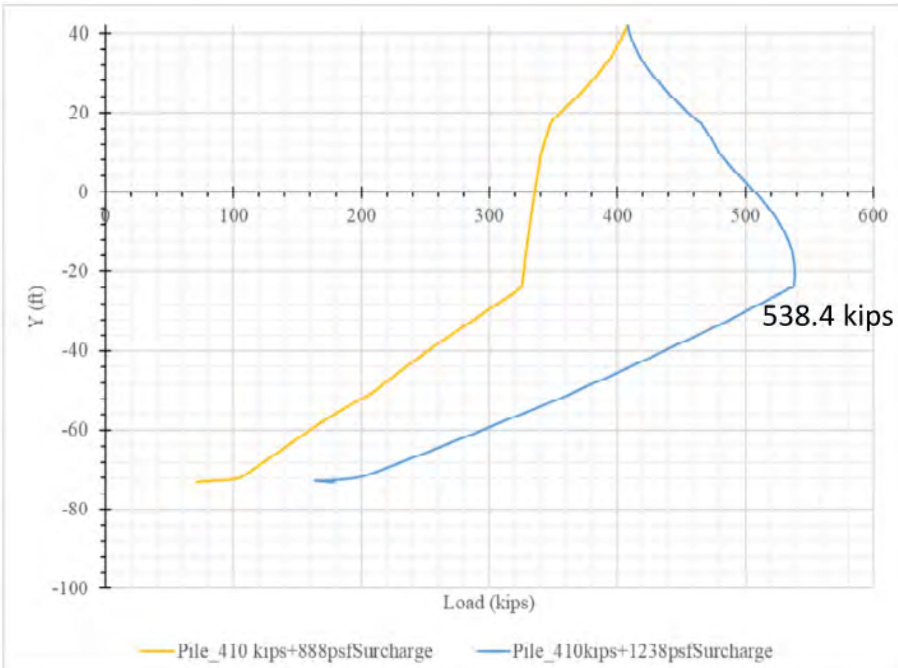


Step 2. Add 350 psf (total 1238 psf) of addi<onal surcharge load to represent the new fill. Note that the downdrag on the pile reduces the seE lement to 3.4" due to model effects, which is likely conserva<ve and results in more downdrag than actual.

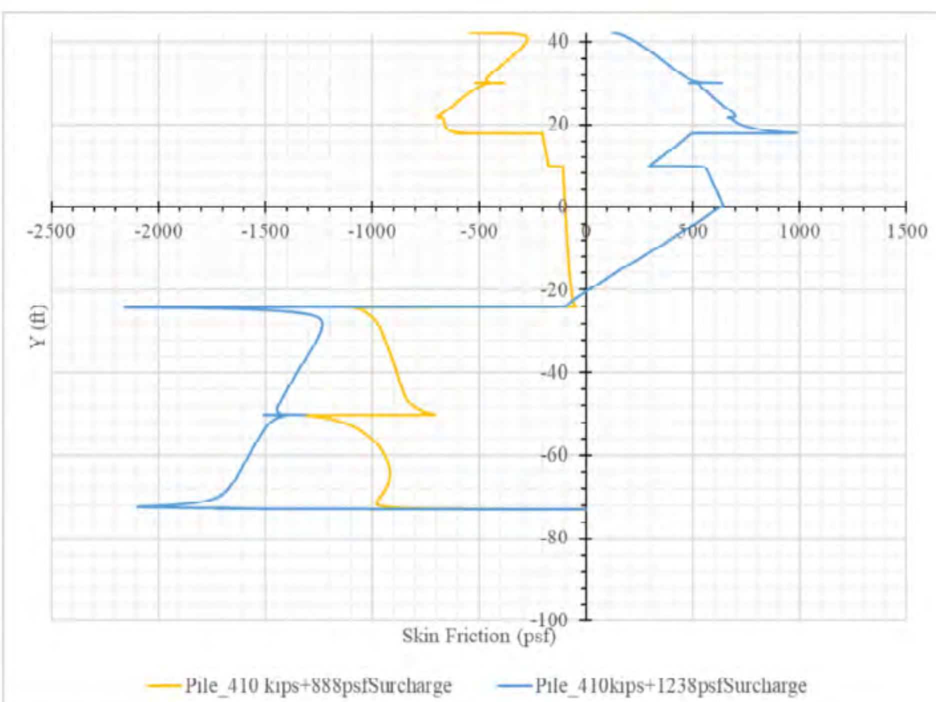


Step 3. Use load distribu<on in the pile from the modeling results to calculate downdrag.

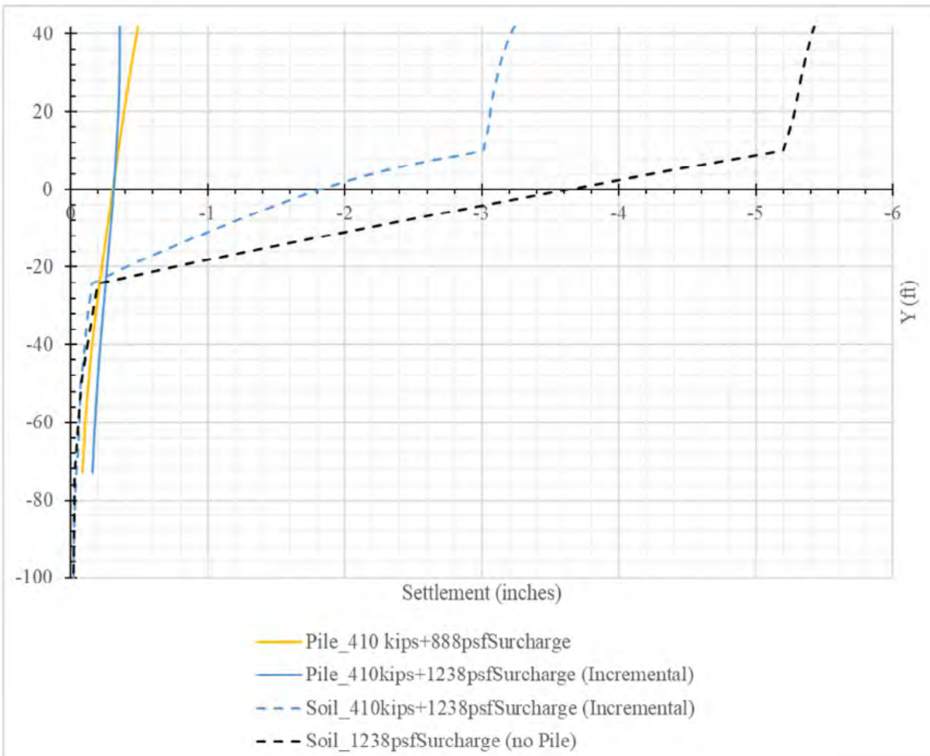
Axial Load Profile :



Skin Fric<on Profile: (Note: Nega<ve skin fric<on resists load, posi<ve skin fric<on is downdrag)



SeE lement Profile :



3. Results

The results indicate that the maximum load in the pile, occurring at the neutral plane which is at the boE om of the clay, is approxi mately 539 kips. The recommended downdrag load considered for pile design is equal to the difference between the maximum load in the pile and the maximum factored pile load of 410 kips, equal to 539 kips. Based on past prac<ce, a load factor of 1.0 was applied to the calculated downdrag resistance, which results in a maximum factored load equal to the maximum load calculated in Plaxis.

Under the maximum factored design load, the predicted seE lement would be approximately 1/2 inch, with an addi<onal approxi mately 1/2 inch occurring under maximum downdrag load. However, it is noted that pile seE lement is a service condi<on and would therefore be less.

Lateral Pile Analysis
Abutments

LPile Input Parameters
MaineDOT - Johnson Road Bridge No. 5792
Falmouth, Maine

GZA FILE NO. 09.0026024.00
CALCULATED BY B.Cardali 11/5/2021
CHECKED BY A.Blaisdell 12/13/2021

Objective: To estimate the horizontal modulus of subgrade reaction (k) or E50 of subsurface strata for use in lateral analyses. K values are estimated using strata internal friction angles (ϕ') or shear strength.

Methods Correlations between the horizontal modulus of subgrade reaction and the soil internal friction angle of a given stratum are based on Figure 3-34 presented in the 2019 LPile Technical Manual.

Given Information: SPT measurements, In-situ vanes, and subsurface conditions in borings BB-FJR-103, -105, -106 performed by New England Borings Contractors between May 22, 2019 and June 25, 2019.

Abutment 1, Pile length = 115'

Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Upper Existing Fill	Reese Sand	42	250	40	130
Lower Existing Fill	Reese Sand	30	165	36	130
Upper Marine Sand**	Reese Sand	21.6	35	30	58
Marine Clay Crust	Stiff Clay w/o free water	17.8	E_{50}=0.007	600	53
Marine Clay	Soft Clay	9.8	E_{50}=0.008	450	53
Lower Marine Sand	Reese Sand	-24.2	55	32	73
Glacial Till	Reese Sand	-50.2	120	38	73
Top of Rock	--	-73	--	--	--

Pier 1, Pile length = 47'

Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
New Fill	Reese Sand	29.8	45	32	63
Upper Marine Sand**	Reese Sand	25.8	35	30	58
Marine Clay Crust	Stiff Clay w/o free water	22.9	E_{50}=0.007	600	53
Marine Clay	Soft Clay	14.4	E_{50}=0.008	450	53
Lower Marine Sand	Reese Sand	-15.9	55	32	73
Top of Rock	--	-19.9	--	--	--

Abutment 2, Pile length = 30'

Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Existing Fill	Reese Sand	53	250	40	130
Upper Marine Sand**	Reese Sand	39.1	35	30	58
Marine Clay Crust	Stiff Clay w/o free water	34.6	E_{50}=0.007	1000	53
Top of Rock	--	23	--	--	--

Notes: 1. Pile tip elevation should be assumed to be top of Rock.
2. ** indicates the top of layer is the approximate ground water elevation based on the boring logs.
3. pci = pounds per cubic inch, deg = degrees, psi =pounds per square inch, γe = effective unit weight, pcf = pounds per square foot.
4. Both abutments were evaluated in Lpile considering two different subsurface profiles. Initial evaluations indicated the upper soil profiles are similar, therefore GZA evaluated the controlling Abutment 2 profile for design.

Table 2 - LPile Output Summary
MaineDOT - Johnson Road Bridge No. 5792
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0026024.00
CALCULATED BY B.Cardali 11/5/2021
CHECKED BY A.Blaisdell 12/13/2021

Abutment 1, Pile Length = 115 ft							
Pile Section	**Axial Load (kips)**	**Shear Force for Lateral deflection of 1.375 in. (kips)**	**Moment at Pile Head (kip-ft)**	**Depth below Pile Head to Fixity (ft)**	**Total Stress at Pile Head (ksi)**	**Bending Stress at Pile Head (ksi)**	**Axial Stress at Pile Head (ksi)**
HP 14x117 (Weak axis)	410	101.6	-356.6	19.5	83.7	71.7	12.0

Abutment 2, Pile Length = 30 ft							
Pile Section	**Axial Load (kips)**	**Shear Force for Lateral deflection of 1.5 in. (kips)**	**Moment at Pile Head (kip-ft)**	**Depth below Pile Head to Fixity (ft)**	**Total Stress at Pile Head (ksi)**	**Bending Stress at Pile Head (ksi)**	**Axial Stress at Pile Head (ksi)**
HP 14x117 (Weak axis)	410	99.2	-361	17.5	84.7	72.7	12.0

Notes:
1. Soil layering and properties are presented in Table 1.
2. The axial load is the maximum Factored Axial Load, excluding factored downdrag load, for the 5 Pile configuration.
3. Lpile model included imposed lateral deflection of 1.375 inches and zero rotation applied at the pile head.
4. The bending stress at pile head is calculated as the maximum moment divided by the pile section modulus.

Conclusion:
Initial lateral pile analyses results presented above indicate the pile stresses are beyond the elastic range. Therefore, an additional plastic hinge evaluation was conducted on the HP14x117 section in the weak axis.

Integral Abutment LRFD Pile Design

Subject: Pile Design for the Johnson Road Bridge Replacement in Falmouth, Maine.

Reference:

- AASHTO LRFD Bridge Design Specifica; ons, 9th Edi; on, 2020
- Maine BDG Chapter 5 - Substructures 2014
- VTRANS Integral Abutment Bridge Design Guidelines 2008

Design Steps - Maine BDG

Step 1 - Determine the founda; on displacements and the load effects (P_u and M_u) from the superstructure and substructure designs.

$P_u := 410.0 \cdot kip$ Maximum Factored Axial Load from Hoyle Tanner (HTA)

Ini; al LPile analyses were run based on a total deflec; on of 1.375 inches with zero end rota; on for these analyses.

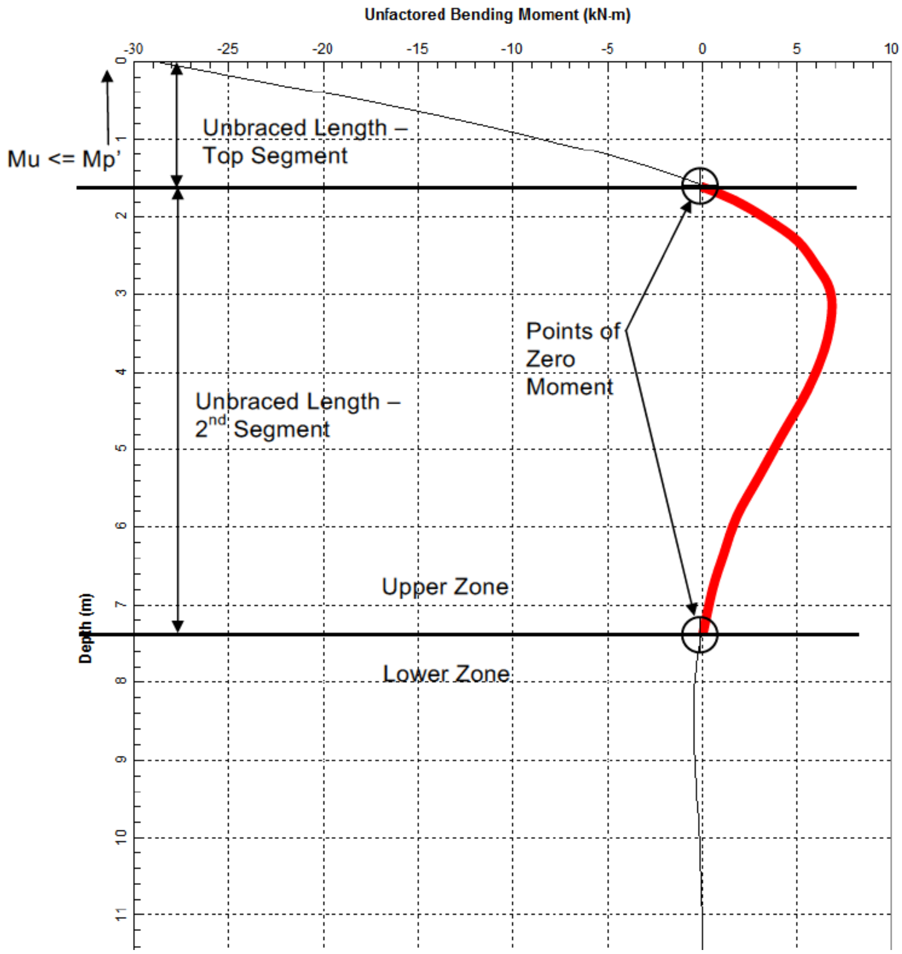
Step 2 - If applicable, determine the magnitude of scour. N / A

Step 3 - Select preliminary pile size.

HP14 x 117, Weak Axis Proper; es

Steel yield strength	$F_{y50} := 50ksi$
Modulus of elas; city for steel	$E := 29000ksi$
Cross sec; onal area of pile	$A_g := 34.4in^2$
Radius of gyra; on	$r_y := 3.59in$
Width of Flange	$b_f := 14.9 \cdot in$
Thickness of Flange	$t_f := 0.805 \cdot in$
Elas; c Sec; on Modulus	$S_y := 59.5 \cdot in^3$
Plas; c Sec; on Modulus	$Z_y := 91.4 \cdot in^3$

Step 4 - Determine the pile unbraced length and maximum moment at the top of the pile by running LPile soFware for top transla; on = 1.375 inches, P_u = 410.0 kip, and Live Load Rota; on = 0 (Fixed against rota; on)



Maximum moment from LPile output (page 22) $M_u := 4336.4 kip \cdot in = 361.37 \cdot kip \cdot ft$

Unbraced lengths from LPile output (page 22)

Upper segment $L_1 := 3.7 \cdot ft$

Lower segment $L_2 := 13.8 \cdot ft$

Step 5 - Determine if the applied moment on the pile will cause pile head plas; c deforma; on considering the interac; on of combined axial and flexural load effects on a single pile (AASHTO LRFD 6.9.2.2)

a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment.

See above for unbraced lengths (cri; cal lengths L_1 and L_2).

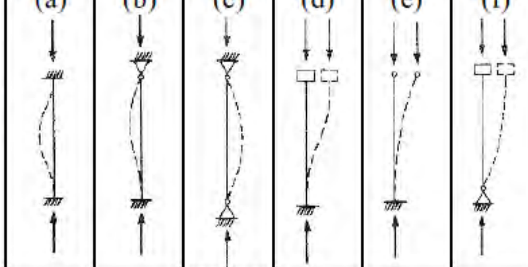
Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_y} = 12.368$$

Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_y} = 46.128$$

b. Determine K values for top and bol om of the pile per AASHTO LRFD Table C4.6.2.5-1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of *K* when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0
End condition code		Rotation fixed Translation fixed Rotation free Translation fixed Rotation fixed Translation free Rotation free Translation free				

Upper segment K value (Type d)

$$K_1 := 1.2$$

Lower segment K value (Type c)

$$K_2 := 1$$

c. Calculate the nominal and factored structural pile resistance P_n, per AASHTO LRFD 6.9.4.1 using the λ values

Elas; c cri; cal buckling resistance, P_e, based on flexural buckling (AASHTO LRFD Eq. 6.9.4.1.2-1)

Upper P_e

$$P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 44701 \cdot kip$$

Lower P_e

$$P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 4627 \cdot kip$$

Nominal yield resistance, P_o $\quad P_o := F_{y50} \cdot A_g = 1720 \cdot kip$

Check that the ra; o of Pe to Po is > 0.44

If $P_e/P_o > 0.44$ then use AASHTO LRFD Eq. 6.9.4.1.1-1
If $P_e/P_o < 0.44$ then use AASHTO LRFD Eq. 6.9.4.1.1-2

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad (6.9.4.1.1\text{-}1)$$

$$P_n = 0.877 P_e \qquad (6.9.4.1.1\text{-}2)$$

$$\frac{P_{e1}}{P_o} = 25.989 \qquad \frac{P_{e2}}{P_o} = 2.69$$

Both ra; os are greater than 0.44, therefore, use AASHTO LRFD Eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658^{\left(\frac{P_o}{P_{e1}}\right)}\right] \cdot P_o = 1693 \cdot kip$$

$$P_{n2} := \left[0.658^{\left(\frac{P_o}{P_{e2}}\right)}\right] \cdot P_o = 1472 \cdot kip$$

Factored structural Pile Resistance, $P_r = \varphi_c (P_n)$

$\phi_c := 0.7$ for axial resistance according to AASHTO LRFD 6.5.4.2

$$P_{r1} := \phi_c \cdot P_{n1} = 1184.8 \cdot kip$$

$$P_{r2} := \phi_c \cdot P_{n2} = 1030.5 \cdot kip$$

d. Compare the ra; o of P_u, the maximum factored axial load, to P_r, the structural resistance in the specified por; on of the pile - the pile size should be such that the ra; o is not less than 0.20.

Check for both segments

$\frac{P_u}{P_{r1}} = 0.346$ > 0.20, OK $\qquad$ $\frac{P_u}{P_{r2}} = 0.398$ > 0.20, OK

e. Determine the nominal and factored flexural resistance about H-Pile weak axis, (AASHTO LRFD Eq. 6.12.2.2)

Check slenderness ra; o for flange, limi; ng slenderness ra; o for compact flange, and limi; ng slenderness ra; o for a noncompact flange.

$$\lambda_f := \frac{b_f}{2 \cdot t_f} = 9.255$$

slenderness ra; o for flange (AASHTO LRFD Eq. 6.12.2.2.1-3)

$$\lambda_{pf} := 0.38 \cdot \left(\frac{E}{F_{y50}}\right)^{.5} = 9.152$$

$$\lambda_{rf} := 0.83 \cdot \left(\frac{E}{F_{y50}}\right)^{.5} = 19.989$$

If $\lambda_{pf} < \lambda_f < \lambda_{rf}$ Use ASSHTO LRFD Eq. 6.12.2.2.1-2 to find the nominal flexural resistance

$$M_n := \left[1 - \left(1 - \frac{S_y}{Z_y}\right) \cdot \left[\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \left(\frac{E}{F_{y50}}\right)^{.5}}\right]\right] \cdot F_{y50} \cdot Z_y = 379.57 \cdot ft \cdot kip$$

$\phi_f := 1.0$ for flexural resistance according to AASHTO LRFD 6.5.4.2

$M_r := M_n \cdot \phi_f = 379.6 \cdot ft \cdot kip$

$M_u = 361.4 \cdot ft \cdot kip$

$\frac{P_u}{P_{r1}} + \frac{8}{9} \cdot \frac{M_u}{M_r} = 1.2$ If less than 1, remains in elas; c zone. Since it exceeds 1, yielding is expected at the base of the pile cap.

j. Calculate the moment that will cause a plas; c hinge at the top of the pile, M_p',
Note: M_p' will be lower than M_n due to the inclusion of the axial load in the interac; on equa; on for pile overstresses

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

AASHTO LRFD Eq. 6.9.2.2 Interac; on equa; on

Use the interac; on equa; on to find the moment that will cause a plas; c hinge at the top of the pile. Assume M_{ux} and M_{rx} = 0 (out-of-plane), $M_{ry} = M_r$ and $M_u = M_p$', solve for Mp'

$M_p := \left(\frac{9}{8}\right) \cdot \left[1 - \left(\frac{P_u}{P_{r1}}\right)\right] \cdot M_r = 279.2 \cdot ft \cdot kip$ $M_p = 3350911 \cdot in \cdot lbf$

k. The calculated moment from LPile Run 1 (shown in step 4) exceeds the moment that would cause a plas; c hinge (above), therefore a plas; c hinge forms, and the moment (M_p') represents the limi; ng moment reac; on at the pile top for the subsequent analysis.

Step 6 - For fixed head piles, run a second LPILE analysis with end condi; ons 1) Top moment = M_p', top transla; on = 1.375 in; and axial load equal to P_u. Recalculate unbraced lengths from the moment vs. depth curve.

New unbraced lengths were determined from the second LPile analysis

Upper segment $L_{1p} := 3.1 \cdot ft$

Lower segment $L_{2p} := 14.2 \cdot ft$

6a. Repeat steps 5a through 5d above.

6b - If the pile size is such that the ra; o of Pu to structural resistance exceeds 0.2, check the upper segment of the pile with the interac; on equa; on of AASHTO LRFD Eq. 6.9.2.2. If a plas; c hinge forms at the top of the pile, the K value of the upper segment changes from 1.2, for a rota; on fixed head condi; on, to 2.1, for a rota; on free head condi; on. With the new K value and lengths repeat step 5.

5a.

Upper segment slenderness factor $\lambda_{1p} := \frac{(L_{1p})}{r_y} = 10.362$

Lower segment slenderness factor $\lambda_{2p} := \frac{(L_{2p})}{r_y} = 47.465$

5b.

Upper segment K value (Type e) $K_{1p} := 2.1$

Lower segment K value (Type c) $K_{2p} := 1$

5c.

Elas; c cri; cal buckling resistance, P_e, based on flexural buckling

Upper P_e $P_{ep1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{1p} \cdot \lambda_{1p})^2} = 20793 \cdot kip$

Lower P_e $P_{ep2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_{2p} \cdot \lambda_{2p})^2} = 4370 \cdot kip$

Nominal yield resistance, P_o $P_o := F_{y50} \cdot A_g = 1720 \cdot kip$

Check that the ra; o of Pe to Po is > 0.44

If $P_e/P_o > 0.44$ then use equa; on 6.9.4.1.1-1
If $P_e/P_o < 0.44$ then use equa; on 6.9.4.1.1-2

$$\frac{P_{ep1}}{P_o} = 12.089 \qquad \frac{P_{ep2}}{P_o} = 2.541$$

Both ra; os are greater than 0.44, therefore, use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{np1} := \left[0.658^{\left(\frac{P_o}{P_{ep1}}\right)}\right] \cdot P_o = 1661 \cdot kip$$

$$P_{np2} := \left[0.658^{\left(\frac{P_o}{P_{ep2}}\right)}\right] \cdot P_o = 1459 \cdot kip$$

Factored structural Pile Resistance, $P_r = \varphi_c (P_n)$

$$P_{rp1} := \phi_c \cdot P_{np1} = 1163 \cdot kip$$

$$P_{rp2} := \phi_c \cdot P_{np2} = 1021 \cdot kip$$

d. Compare the ra; o of P_u to the structural resistance in the upper por; on of the pile - the pile size should be such that the ra; o is not less than 0.20.

Check for both segments

$$\frac{P_u}{P_{rp1}} = 0.353$$ > 0.20, OK

$$\frac{P_u}{P_{rp2}} = 0.402$$ > 0.20, OK

From VTrans Integral Abutment Design Sec; on 4.5.2 - Check the axial capacity of the upper segment and the interac; on equa; on for the second segment to assess suitability of pile sec; on.

Upper Segment

Check that P_u/P_{rp1} is < 1

$$\frac{P_u}{P_{rp1}} = 0.353$$ < 1, OK

Lower Segment

Ul; mate moment along the lower segment from LPile output

$$M_{maxp2} := 2343387 \cdot in \cdot lbf$$

$$\frac{P_u}{P_{rp2}} + \frac{8}{9} \cdot \frac{M_{maxp2}}{M_r} = 0.859$$

< 1, OK.

Conclusion:
The ul; mate moment of the pile in the lower segment generates a demand ra; o (AASHTO LRFD 6.9.2.2) that is less than 1, therefore the structural capability of the lower segment is sufficient in resis; ng the moment.

Abutment 1 Initial analysis

```
===============================================================================

                    LPile for Windows, Version 2019-11.002

                Analysis of Individual Piles and Drilled Shafts
               Subjected to Lateral Loading Using the p-y Method
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===============================================================================

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-------------------------------------------------------------------------------
                             Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:
\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Lpile\Abutment 1\

Name of input data file:
Johnson AB 1_14x117 Weak axis.lp11

Name of output report file:
Johnson AB 1_14x117 Weak axis.lp11

Name of plot output file:
Johnson AB 1_14x117 Weak axis.lp11

Name of runtime message file:
Johnson AB 1_14x117 Weak axis.lp11


-------------------------------------------------------------------------------
                            Date and Time of Analysis
-------------------------------------------------------------------------------

               Date:  January 24, 2022           Time:  10:27:47


-------------------------------------------------------------------------------
                                  Problem Title
-------------------------------------------------------------------------------

Project Name: Johnson Road Bridge

Job Number: 09.0026024.00

Client: MaineDOT

Engineer: B.Cardali

Description: Abutment 1 - 14x89



-------------------------------------------------------------------------------
                           Program Options and Settings
-------------------------------------------------------------------------------

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed               =          500
 - Deflection tolerance for convergence               =   1.0000E-05 in
 - Maximum allowable deflection                       =     100.0000 in
 - Number of pile increments                          =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected
```

```
Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats



--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                      =           1
Total length of pile                                 =      115.000 ft
Depth of ground surface below top of pile            =       0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.

            Depth Below          Pile
Point       Pile Head          Diameter
 No.           feet             inches
-----      -------------     ------------
  1            0.000         14.9000
  2          115.000         14.9000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                 =   115.000000 ft
   Pile width                                        =    14.200000 in
   Shear capacity of section                         =       0.0000 lbs



--------------------------------------------------------------------------------
                       Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------

Ground Slope Angle                                   =        0.000 degrees
                                                     =        0.000 radians

Pile Batter Angle                                    =        0.000 degrees
                                                     =        0.000 radians


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 7 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =       0.0000 ft
   Distance from top of pile to bottom of layer      =    12.000000 ft
   Effective unit weight at top of layer             =   130.000000 pcf
   Effective unit weight at bottom of layer          =   130.000000 pcf
   Friction angle at top of layer                    =    40.000000 deg.
   Friction angle at bottom of layer                 =    40.000000 deg.
   Subgrade k at top of layer                        =   250.000000 pci
   Subgrade k at bottom of layer                     =   250.000000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =    12.000000 ft
   Distance from top of pile to bottom of layer      =    20.400000 ft
   Effective unit weight at top of layer             =   130.000000 pcf
   Effective unit weight at bottom of layer          =   130.000000 pcf
   Friction angle at top of layer                    =    36.000000 deg.
   Friction angle at bottom of layer                 =    36.000000 deg.
   Subgrade k at top of layer                        =   165.000000 pci
   Subgrade k at bottom of layer                     =   165.000000 pci


Layer 3 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =    20.400000 ft
   Distance from top of pile to bottom of layer      =    24.200000 ft
   Effective unit weight at top of layer             =    58.000000 pcf
   Effective unit weight at bottom of layer          =    58.000000 pcf
   Friction angle at top of layer                    =    30.000000 deg.
   Friction angle at bottom of layer                 =    30.000000 deg.
   Subgrade k at top of layer                        =    35.000000 pci
   Subgrade k at bottom of layer                     =    35.000000 pci
```

Layer 4 is stiff clay without free water

```
   Distance from top of pile to top of layer          =    24.200000 ft
   Distance from top of pile to bottom of layer       =    32.200000 ft
   Effective unit weight at top of layer              =    53.000000 pcf
   Effective unit weight at bottom of layer           =    53.000000 pcf
   Undrained cohesion at top of layer                 =   600.000000 psf
   Undrained cohesion at bottom of layer              =   600.000000 psf
   Epsilon-50 at top of layer                         =     0.007000
   Epsilon-50 at bottom of layer                      =     0.007000
```

Layer 5 is soft clay, p-y criteria by Matlock, 1970

```
   Distance from top of pile to top of layer          =    32.200000 ft
   Distance from top of pile to bottom of layer       =    66.200000 ft
   Effective unit weight at top of layer              =    53.000000 pcf
   Effective unit weight at bottom of layer           =    53.000000 pcf
   Undrained cohesion at top of layer                 =   450.000000 psf
   Undrained cohesion at bottom of layer              =   450.000000 psf
   Epsilon-50 at top of layer                         =     0.008000
   Epsilon-50 at bottom of layer                      =     0.008000
```

Layer 6 is sand, p-y criteria by Reese et al., 1974

```
   Distance from top of pile to top of layer          =    66.200000 ft
   Distance from top of pile to bottom of layer       =    92.200000 ft
   Effective unit weight at top of layer              =    73.000000 pcf
   Effective unit weight at bottom of layer           =    73.000000 pcf
   Friction angle at top of layer                     =    32.000000 deg.
   Friction angle at bottom of layer                  =    32.000000 deg.
   Subgrade k at top of layer                         =    55.000000 pci
   Subgrade k at bottom of layer                      =    55.000000 pci
```

Layer 7 is sand, p-y criteria by Reese et al., 1974

```
   Distance from top of pile to top of layer          =    92.200000 ft
   Distance from top of pile to bottom of layer       =   115.000000 ft
   Effective unit weight at top of layer              =    73.000000 pcf
   Effective unit weight at bottom of layer           =    73.000000 pcf
   Friction angle at top of layer                     =    38.000000 deg.
   Friction angle at bottom of layer                  =    38.000000 deg.
   Subgrade k at top of layer                         =   120.000000 pci
   Subgrade k at bottom of layer                      =   120.000000 pci
```

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	0.00	130.0000	--	40.0000	--	250.0000
	(Reese, et al.)	12.0000	130.0000	--	40.0000	--	250.0000
2	Sand	12.0000	130.0000	--	36.0000	--	165.0000
	(Reese, et al.)	20.4000	130.0000	--	36.0000	--	165.0000
3	Sand	20.4000	58.0000	--	30.0000	--	35.0000
	(Reese, et al.)	24.2000	58.0000	--	30.0000	--	35.0000
4	Stiff Clay	24.2000	53.0000	600.0000	--	0.00700	--
	w/o Free Water	32.2000	53.0000	600.0000	--	0.00700	--
5	Soft	32.2000	53.0000	450.0000	--	0.00800	--
	Clay	66.2000	53.0000	450.0000	--	0.00800	--
6	Sand	66.2000	73.0000	--	32.0000	--	55.0000
	(Reese, et al.)	92.2000	73.0000	--	32.0000	--	55.0000
7	Sand	92.2000	73.0000	--	38.0000	--	120.0000
	(Reese, et al.)	115.0000	73.0000	--	38.0000	--	120.0000

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 1.375000 in	S = 0.0000 in/in	410000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head

y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	115.000000 ft
Flange Width	=	14.900000 in
Section Depth	=	14.200000 in
Flange Thickness	=	0.805000 in
Web Thickness	=	0.805000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	34.123950 sq. in.
Moment of Inertia	=	444.363799 in^4
Elastic Bending Stiffness	=	12886550. kip-in^2
Plastic Modulus, Z	=	91.398684in^3
Plastic Moment Capacity = Fy Z	=	4570.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1706.197 kips
Nominal Axial Tensile Capacity	=	-1706.197 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	410.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 410.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000443	57.0960271	12886018.	100.9559856	12.9627342	
0.00000886	114.1920543	12886018.	54.2029928	13.9104468	
0.00001329	171.2880814	12886018.	38.6186619	14.8581592	
0.00001772	228.3841085	12886018.	30.8264964	15.8058717	
0.00002215	285.4801356	12886018.	26.1511971	16.7535843	
0.00002659	342.5761628	12886018.	23.0343309	17.7012968	
0.00003102	399.6721899	12886018.	20.8079979	18.6490093	
0.00003545	456.7682170	12886018.	19.1382482	19.5967219	
0.00003988	513.8642441	12886018.	17.8395540	20.5444344	
0.00004431	570.9602713	12886018.	16.8005986	21.4921470	
0.00004874	628.0562984	12886018.	15.9505441	22.4398594	
0.00005317	685.1523255	12886018.	15.2421655	23.3875720	
0.00005760	742.2483526	12886018.	14.6427681	24.3352845	
0.00006203	799.3443798	12886018.	14.1289990	25.2829970	
0.00006646	856.4404069	12886018.	13.6837324	26.2307096	
0.00007089	913.5364340	12886018.	13.2941241	27.1784221	
0.00007532	970.6324612	12886018.	12.9503521	28.1261346	
0.00007976	1028.	12886018.	12.6447770	29.0738471	
0.00008419	1085.	12886018.	12.3713677	30.0215597	
0.00008862	1142.	12886018.	12.1252993	30.9692722	
0.00009305	1199.	12886018.	11.9026660	31.9169847	
0.00009748	1256.	12886018.	11.7002721	32.8646972	
0.0001019	1313.	12886018.	11.5154776	33.8124098	
0.0001063	1370.	12886018.	11.3460827	34.7601223	
0.0001108	1427.	12886018.	11.1902394	35.7078348	
0.0001152	1484.	12886018.	11.0463841	36.6555473	
0.0001196	1542.	12886018.	10.9131847	37.6032599	
0.0001241	1599.	12886018.	10.7894995	38.5509724	
0.0001285	1656.	12886018.	10.6743443	39.4986849	
0.0001329	1713.	12886018.	10.5668662	40.4463974	
0.0001374	1770.	12886018.	10.4663221	41.3941100	
0.0001418	1827.	12886018.	10.3720621	42.3418225	
0.0001462	1884.	12886018.	10.2835147	43.2895350	
0.0001506	1941.	12886018.	10.2001760	44.2372475	
0.0001551	1998.	12886018.	10.1215996	45.1849601	

0.0001595	2055.	12886018.	10.0473885	46.1326726	
0.0001639	2113.	12886018.	9.9771888	47.0803851	
0.0001684	2170.	12886018.	9.9106838	48.0280977	
0.0001728	2227.	12886018.	9.8475894	48.9758102	
0.0001817	2339.	12876578.	9.7319297	50.0000000	Y
0.0001905	2444.	12828475.	9.6325790	50.0000000	Y
0.0001994	2542.	12750935.	9.5470130	50.0000000	Y
0.0002082	2635.	12651442.	9.4731172	50.0000000	Y
0.0002171	2722.	12536098.	9.4091017	50.0000000	Y
0.0002260	2804.	12409882.	9.3534361	50.0000000	Y
0.0002348	2883.	12274799.	9.3051750	50.0000000	Y
0.0002437	2957.	12135330.	9.2629997	50.0000000	Y
0.0002526	3029.	11992327.	9.2262858	50.0000000	Y
0.0002614	3097.	11848701.	9.1940689	50.0000000	Y
0.0002703	3163.	11703988.	9.1660813	50.0000000	Y
0.0002791	3227.	11560087.	9.1416188	50.0000000	Y
0.0002880	3288.	11417813.	9.1202192	50.0000000	Y
0.0002969	3348.	11277560.	9.1015349	50.0000000	Y
0.0003057	3404.	11135225.	9.0844038	50.0000000	Y
0.0003146	3456.	10986959.	9.0676771	50.0000000	Y
0.0003235	3504.	10834546.	9.0512930	50.0000000	Y
0.0003323	3549.	10679271.	9.0351400	50.0000000	Y
0.0003412	3590.	10522867.	9.0192571	50.0000000	Y
0.0003500	3628.	10365954.	9.0038004	50.0000000	Y
0.0003589	3664.	10208997.	8.9886718	50.0000000	Y
0.0003678	3697.	10053411.	8.9736823	50.0000000	Y
0.0003766	3728.	9899561.	8.9589475	50.0000000	Y
0.0003855	3757.	9747283.	8.9447210	50.0000000	Y
0.0003943	3785.	9598168.	8.9305960	50.0000000	Y
0.0004032	3811.	9451057.	8.9170017	50.0000000	Y
0.0004121	3835.	9306266.	8.9032592	50.0000000	Y
0.0004209	3858.	9164398.	8.8900200	50.0000000	Y
0.0004298	3879.	9026243.	8.8770717	50.0000000	Y
0.0004387	3899.	8889551.	8.8641553	50.0000000	Y
0.0004475	3919.	8756997.	8.8514541	50.0000000	Y
0.0004564	3937.	8626674.	8.8392739	50.0000000	Y
0.0004652	3954.	8499564.	8.8269396	50.0000000	Y
0.0004741	3971.	8375615.	8.8152008	50.0000000	Y
0.0004830	3986.	8253806.	8.8033506	50.0000000	Y
0.0004918	4002.	8136089.	8.7918169	50.0000000	Y
0.0005007	4015.	8019623.	8.7804579	50.0000000	Y
0.0005095	4029.	7907009.	8.7694761	50.0000000	Y
0.0005184	4042.	7796593.	8.7581957	50.0000000	Y
0.0005273	4054.	7688898.	8.7476372	50.0000000	Y
0.0005627	4098.	7282675.	8.7061484	50.0000000	Y
0.0005982	4136.	6913903.	8.6674959	50.0000000	Y
0.0006336	4167.	6577187.	8.6307083	50.0000000	Y
0.0006691	4195.	6269851.	8.5962337	50.0000000	Y
0.0007045	4219.	5988283.	8.5639449	50.0000000	Y
0.0007400	4240.	5729974.	8.5331459	50.0000000	Y
0.0007754	4258.	5491907.	8.5042310	50.0000000	Y
0.0008108	4275.	5272364.	8.4766239	50.0000000	Y
0.0008463	4290.	5069083.	8.4508029	50.0000000	Y
0.0008817	4303.	4880461.	8.4257503	50.0000000	Y
0.0009172	4315.	4704743.	8.4021622	50.0000000	Y
0.0009526	4326.	4541307.	8.3798809	50.0000000	Y
0.0009881	4336.	4388411.	8.3581407	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	410.0000000000	4336.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	358099.
2	12.0000	13.5063	Yes	No	358099.	1061064.
3	20.4000	27.9236	Yes	No	1419164.	330155.
4	24.2000	264.2282	No	No	1749319.	53618.

5	32.2000	361.6995	No	No	1802936.	171072.
6	66.2000	33.6987	No	No	1974009.	6307485.
7	92.2000	46.9597	Yes	No	8281494.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 1.375000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 410000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.3750	-4279110.	101583.	0.00	83757.	5.21E+09	0.00	0.00	0.00
1.1500	1.2969	-2883297.	96822.	-0.00948	60355.	5.21E+09	-290.1154	3087.	0.00
2.3000	1.1134	-1499556.	90502.	-0.01410	37156.	1.29E+10	-625.9300	7758.	0.00
3.4500	0.9078	-225930.	79919.	-0.01502	15803.	1.29E+10	-907.7522	13800.	0.00
4.6000	0.6988	876192.	64956.	-0.01467	26705.	1.29E+10	-1261.	24899.	0.00
5.7500	0.5028	1732884.	44974.	-0.01328	41068.	1.29E+10	-1635.	44877.	0.00
6.9000	0.3324	2267690.	22078.	-0.01113	50034.	1.29E+10	-1683.	69876.	0.00
8.0500	0.1955	2468224.	-634.6802	-0.00859	53396.	1.28E+10	-1609.	113537.	0.00
9.2000	0.09534	2347367.	-20953.	-0.00600	51370.	1.29E+10	-1336.	193382.	0.00
10.3500	0.02989	1957839.	-35862.	-0.00369	44839.	1.29E+10	-824.7416	380812.	0.00
11.5000	-0.00663	1399384.	-39974.	-0.00190	35476.	1.29E+10	228.7966	476100.	0.00
12.6500	-0.02247	876022.	-34512.	-6.79E-04	26702.	1.29E+10	562.7964	345649.	0.00
13.8000	-0.02536	454531.	-25847.	3.39E-05	19635.	1.29E+10	692.9607	377071.	0.00
14.9500	-0.02153	162253.	-16667.	3.64E-04	14735.	1.29E+10	637.4494	408494.	0.00
16.1000	-0.01531	-9613.	-8901.	4.46E-04	12176.	1.29E+10	488.0736	439916.	0.00
17.2500	-0.00923	-88471.	-3359.	3.93E-04	13498.	1.29E+10	315.2056	471339.	0.00
18.4000	-0.00445	-106766.	-64.1397	2.89E-04	13805.	1.29E+10	162.2751	502762.	0.00
19.5500	-0.00126	-93509.	1391.	1.82E-04	13583.	1.29E+10	48.6791	534184.	0.00
20.7000	5.57E-04	-70416.	1694.	9.38E-05	13196.	1.29E+10	-4.8434	119977.	0.00
21.8500	0.00133	-47819.	1576.	3.05E-05	12817.	1.29E+10	-12.2154	126643.	0.00
23.0000	0.00140	-27258.	1399.	-9.72E-06	12472.	1.29E+10	-13.5084	133308.	0.00
24.1500	0.00106	-9105.	1231.	-2.92E-05	12168.	1.29E+10	-10.7803	139973.	0.00
25.3000	5.93E-04	7051.	735.8245	-3.03E-05	12133.	1.29E+10	-61.0015	1420261.	0.00
26.4500	2.27E-04	11547.	45.1371	-2.03E-05	12209.	1.29E+10	-39.0982	2378748.	0.00
27.6000	3.16E-05	8527.	-262.1942	-9.58E-06	12158.	1.29E+10	-5.4426	2378748.	0.00
28.7500	-3.77E-05	4419.	-254.9628	-2.65E-06	12089.	1.29E+10	6.4907	2378748.	0.00
29.9000	-4.16E-05	1520.	-160.7251	5.30E-07	12041.	1.29E+10	7.1670	2378748.	0.00
31.0500	-2.30E-05	-23.1352	-83.8724	1.33E-06	12015.	1.29E+10	3.9711	2378748.	0.00
32.2000	-4.84E-06	-809.9087	-12.8289	8.85E-07	12029.	1.29E+10	6.3251	1.80E+07	0.00
33.3500	1.39E-06	-387.2295	25.9706	2.44E-07	12022.	1.29E+10	-0.7019	6968250.	0.00
34.5000	1.90E-06	-95.8830	14.5193	-1.46E-08	12017.	1.29E+10	-0.9577	6968250.	0.00
35.6500	9.86E-07	13.6680	4.4763	-5.87E-08	12015.	1.29E+10	-0.4978	6968250.	0.00
36.8000	2.77E-07	28.3269	0.07501	-3.62E-08	12015.	1.29E+10	-0.1400	6968250.	0.00
37.9500	-1.27E-08	16.1477	-0.8470	-1.24E-08	12015.	1.29E+10	0.00640	6968250.	0.00
39.1000	-6.40E-08	5.0898	-0.5798	-9.95E-10	12015.	1.29E+10	0.03233	6968250.	0.00
40.2500	-4.01E-08	0.1578	-0.2168	1.81E-09	12015.	1.29E+10	0.02027	6968250.	0.00
41.4000	-1.39E-08	-0.9148	-0.02838	1.41E-09	12015.	1.29E+10	0.00704	6968250.	0.00
42.5500	-1.25E-09	-0.6415	0.02452	5.76E-10	12015.	1.29E+10	6.30E-04	6968250.	0.00
43.7000	1.96E-09	-0.2444	0.02204	1.02E-10	12015.	1.29E+10	-9.90E-04	6968250.	0.00
44.8500	1.56E-09	-0.03441	0.00978	-4.77E-11	12015.	1.29E+10	-7.86E-04	6968250.	0.00
46.0000	6.45E-10	0.02604	0.00210	-5.21E-11	12015.	1.29E+10	-3.26E-04	6968250.	0.00
47.1500	1.18E-10	0.02426	-5.57E-04	-2.52E-11	12015.	1.29E+10	-5.97E-05	6968250.	0.00
48.3000	-5.05E-11	0.01095	-7.93E-04	-6.36E-12	12015.	1.29E+10	2.55E-05	6968250.	0.00
49.4500	-5.74E-11	0.00244	-4.17E-04	0.00	12015.	1.29E+10	2.90E-05	6968250.	0.00
50.6000	-2.81E-11	-5.66E-04	-1.19E-04	1.82E-12	12015.	1.29E+10	1.42E-05	6968250.	0.00
51.7500	-7.26E-12	-8.67E-04	4.11E-06	1.05E-12	12015.	1.29E+10	3.67E-06	6968250.	0.00
52.9000	0.00	-4.65E-04	2.66E-05	0.00	12015.	1.29E+10	-4.02E-07	6968250.	0.00
54.0500	1.99E-12	-1.36E-04	1.69E-05	0.00	12015.	1.29E+10	-1.00E-06	6968250.	0.00
55.2000	1.16E-12	2.41E-06	5.95E-06	0.00	12015.	1.29E+10	-5.88E-07	6968250.	0.00
56.3500	0.00	2.89E-05	5.76E-07	0.00	12015.	1.29E+10	-1.91E-07	6968250.	0.00
57.5000	0.00	1.88E-05	-8.09E-07	0.00	12015.	1.29E+10	-9.57E-09	6968250.	0.00
58.6500	0.00	6.73E-06	-6.55E-07	0.00	12015.	1.29E+10	3.19E-08	6968250.	0.00
59.8000	0.00	7.24E-07	-2.74E-07	0.00	12015.	1.29E+10	2.32E-08	6968250.	0.00
60.9500	0.00	-8.65E-07	-5.17E-08	0.00	12015.	1.29E+10	9.07E-09	6968250.	0.00
62.1000	0.00	-7.21E-07	2.05E-08	0.00	12015.	1.29E+10	1.40E-09	6968250.	0.00
63.2500	0.00	-3.07E-07	2.41E-08	0.00	12015.	1.29E+10	-8.86E-10	6968250.	0.00
64.4000	0.00	-5.93E-08	1.19E-08	0.00	12015.	1.29E+10	-8.82E-10	6968250.	0.00
65.5500	0.00	2.08E-08	2.78E-09	0.00	12015.	1.29E+10	-4.35E-10	6968250.	0.00
66.7000	0.00	1.79E-08	-3.12E-10	0.00	12015.	1.29E+10	-1.25E-11	607504.	0.00
67.8500	0.00	1.26E-08	-3.91E-10	0.00	12015.	1.29E+10	1.18E-12	617978.	0.00
69.0000	0.00	7.34E-09	-3.35E-10	0.00	12015.	1.29E+10	6.92E-12	628452.	0.00
70.1500	0.00	3.39E-09	-2.33E-10	0.00	12015.	1.29E+10	7.83E-12	638926.	0.00
71.3000	0.00	9.04E-10	-1.35E-10	0.00	12015.	1.29E+10	6.40E-12	649400.	0.00
72.4500	0.00	-3.67E-10	-6.11E-11	0.00	12015.	1.29E+10	4.29E-12	659875.	0.00
73.6000	0.00	-8.18E-10	-1.51E-11	0.00	12015.	1.29E+10	2.37E-12	670349.	0.00
74.7500	0.00	-8.13E-10	8.03E-12	0.00	12015.	1.29E+10	0.00	680823.	0.00
75.9000	0.00	-6.15E-10	1.59E-11	0.00	12015.	1.29E+10	0.00	691297.	0.00
77.0500	0.00	-3.83E-10	1.54E-11	0.00	12015.	1.29E+10	0.00	701771.	0.00
78.2000	0.00	-1.93E-10	1.15E-11	0.00	12015.	1.29E+10	0.00	712246.	0.00
79.3500	0.00	-6.64E-11	7.04E-12	0.00	12015.	1.29E+10	0.00	722720.	0.00
80.5000	0.00	2.43E-12	3.45E-12	0.00	12015.	1.29E+10	0.00	733194.	0.00

81.6500	0.00	3.03E-11	1.11E-12	0.00	12015.	1.29E+10	0.00	743668.	0.00
82.8000	0.00	3.42E-11	0.00	0.00	12015.	1.29E+10	0.00	754142.	0.00
83.9500	0.00	2.73E-11	0.00	0.00	12015.	1.29E+10	0.00	764617.	0.00
85.1000	0.00	1.76E-11	0.00	0.00	12015.	1.29E+10	0.00	775091.	0.00
86.2500	0.00	9.14E-12	0.00	0.00	12015.	1.29E+10	0.00	785565.	0.00
87.4000	0.00	3.39E-12	0.00	0.00	12015.	1.29E+10	0.00	796039.	0.00
88.5500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	806513.	0.00
89.7000	0.00	-1.16E-12	0.00	0.00	12015.	1.29E+10	0.00	816988.	0.00
90.8500	0.00	-1.41E-12	0.00	0.00	12015.	1.29E+10	0.00	827462.	0.00
92.0000	0.00	-1.14E-12	0.00	0.00	12015.	1.29E+10	0.00	837936.	0.00
93.1500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1851077.	0.00
94.3000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1873930.	0.00
95.4500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1896782.	0.00
96.6000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1919635.	0.00
97.7500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1942488.	0.00
98.9000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1965341.	0.00
100.0500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1988194.	0.00
101.2000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2011046.	0.00
102.3500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2033899.	0.00
103.5000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2056752.	0.00
104.6500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2079605.	0.00
105.8000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2102458.	0.00
106.9500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2125310.	0.00
108.1000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2148163.	0.00
109.2500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2171016.	0.00
110.4000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2193869.	0.00
111.5500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2216722.	0.00
112.7000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2239574.	0.00
113.8500	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	2262427.	0.00
115.0000	0.00	0.00	0.00	0.00	12015.	1.29E+10	0.00	1142640.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection            =     1.37500000 inches
Computed slope at pile head     =       0.000000 radians
Maximum bending moment          =      -4279110. inch-lbs
Maximum shear force             =        101583. lbs
Depth of maximum bending moment =       0.000000 feet below pile head
Depth of maximum shear force    =       0.000000 feet below pile head
Number of iterations            =             13
Number of zero deflection points =            17
```

Summary of Pile-head Responses for Conventional Analyses

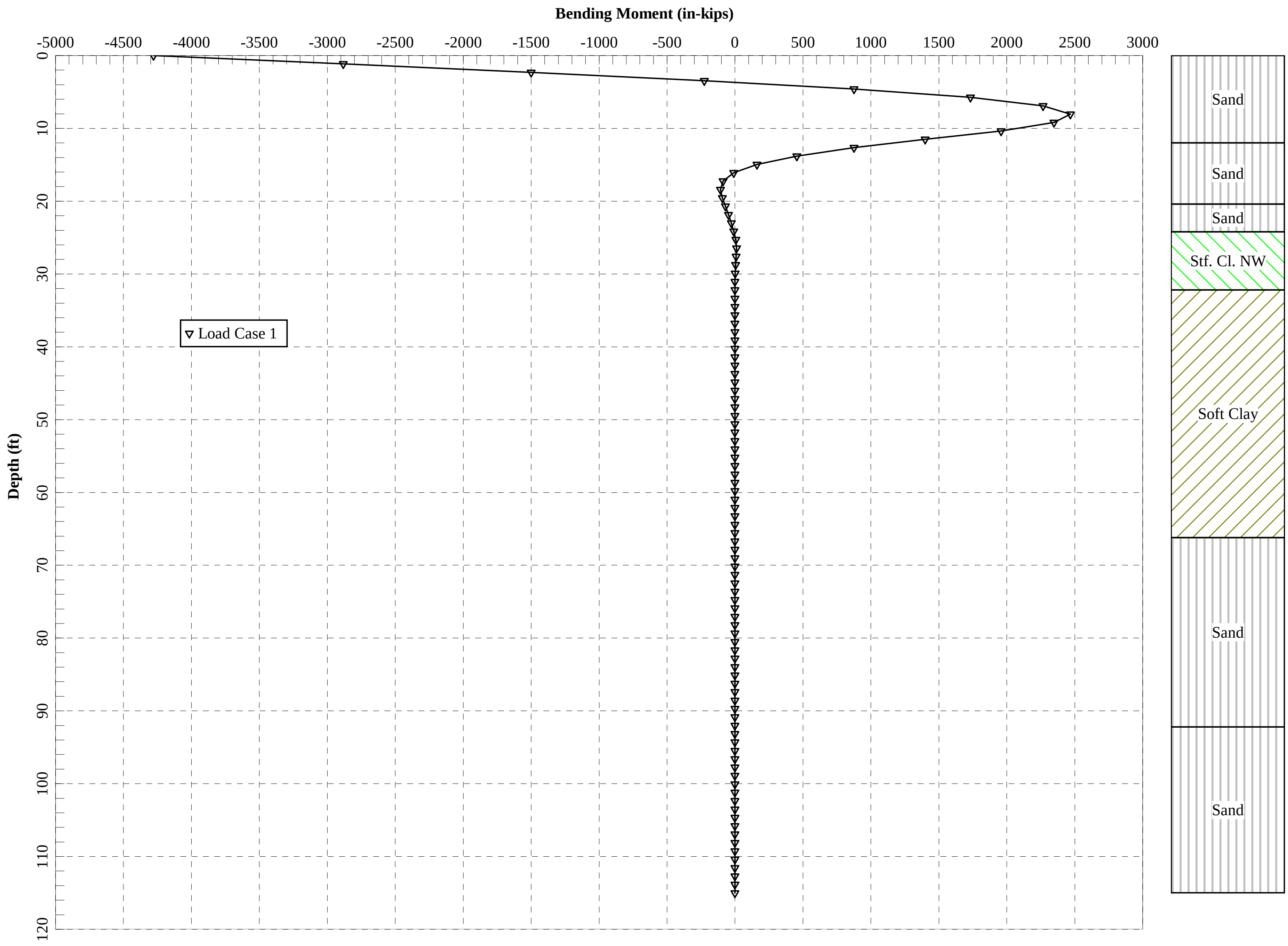
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.3750	S, rad	0.00	410000.	1.3750	0.00	101583.	-4279110.

Maximum pile-head deflection = 1.3750000000 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.


Bending Moment (in-kips)
-5000
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
Load Case 1
Sand
Sand
Sand
Stf. Cl. NW
Soft Clay
Sand
Sand

Abutment 2 Initial & Plastic Hinge analyses

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                     LPile for Windows, Version 2019-11.002

                 Analysis of Individual Piles and Drilled Shafts
                Subjected to Lateral Loading Using the p-y Method
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-------------------------------------------------------------------------------
                             Files Used for Analysis
-------------------------------------------------------------------------------

Path to file locations:
\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Lpile\Abutment 2\14x117\

Name of input data file:
Johnson AB 2_14x117 Weak axis.lp11

Name of output report file:
Johnson AB 2_14x117 Weak axis.lp11

Name of plot output file:
Johnson AB 2_14x117 Weak axis.lp11

Name of runtime message file:
Johnson AB 2_14x117 Weak axis.lp11


-------------------------------------------------------------------------------
                            Date and Time of Analysis
-------------------------------------------------------------------------------

               Date:  January 24, 2022           Time:  10:30:38


-------------------------------------------------------------------------------
                                  Problem Title
-------------------------------------------------------------------------------

Project Name: Johnson Road Bridge

Job Number: 09.0026024.00

Client: MaineDOT

Engineer: B.Cardali

Description: Abutment 2 - 14x117 weak axis



-------------------------------------------------------------------------------
                           Program Options and Settings
-------------------------------------------------------------------------------

Computational Options:
 - Conventional Analysis
Engineering Units Used for Data Input and Computations:
 - US Customary System Units (pounds, feet, inches)

Analysis Control Options:
 - Maximum number of iterations allowed               =          500
 - Deflection tolerance for convergence               =   1.0000E-05 in
 - Maximum allowable deflection                       =     100.0000 in
 - Number of pile increments                          =          100

Loading Type and Number of Cycles of Loading:
 - Static loading specified

 - Use of p-y modification factors for p-y curves not selected
 - Analysis uses layering correction (Method of Georgiadis)
 - No distributed lateral loads are entered
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Input of moment resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected
```

```
Output Options:
 - Output files use decimal points to denote decimal symbols.
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1
 - No p-y curves to be computed and reported for user-specified depths
 - Print using wide report formats



--------------------------------------------------------------------------------
                     Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Number of pile sections defined                      =            1
Total length of pile                                 =       30.000 ft
Depth of ground surface below top of pile            =       0.0000 ft

Pile diameters used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile. A summary of values of pile diameter vs. depth follows.

            Depth Below         Pile
Point        Pile Head        Diameter
 No.           feet            inches
-----      -------------    -------------
  1            0.000          14.9000
  2           30.000          14.9000


Input Structural Properties for Pile Sections:
----------------------------------------------

Pile Section No. 1:

   Section 1 is a H weak axis steel pile
   Length of section                                 =    30.000000 ft
   Pile width                                        =    14.200000 in
   Shear capacity of section                         =       0.0000 lbs



--------------------------------------------------------------------------------
                       Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------




Ground Slope Angle                                   =        0.000 degrees
                                                     =        0.000 radians

Pile Batter Angle                                    =        0.000 degrees
                                                     =        0.000 radians


--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------

The soil profile is modelled using 3 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =       0.0000 ft
   Distance from top of pile to bottom of layer      =    13.900000 ft
   Effective unit weight at top of layer             =   130.000000 pcf
   Effective unit weight at bottom of layer          =   130.000000 pcf
   Friction angle at top of layer                    =    40.000000 deg.
   Friction angle at bottom of layer                 =    40.000000 deg.
   Subgrade k at top of layer                        =   250.000000 pci
   Subgrade k at bottom of layer                     =   250.000000 pci


Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer         =    13.900000 ft
   Distance from top of pile to bottom of layer      =    18.400000 ft
   Effective unit weight at top of layer             =    58.000000 pcf
   Effective unit weight at bottom of layer          =    58.000000 pcf
   Friction angle at top of layer                    =    30.000000 deg.
   Friction angle at bottom of layer                 =    30.000000 deg.
   Subgrade k at top of layer                        =    35.000000 pci
   Subgrade k at bottom of layer                     =    35.000000 pci


Layer 3 is stiff clay without free water

   Distance from top of pile to top of layer         =    18.400000 ft
   Distance from top of pile to bottom of layer      =    30.000000 ft
   Effective unit weight at top of layer             =    53.000000 pcf
   Effective unit weight at bottom of layer          =    53.000000 pcf
   Undrained cohesion at top of layer                =  1000.000000 psf
   Undrained cohesion at bottom of layer             =  1000.000000 psf
   Epsilon-50 at top of layer                        =     0.007000
   Epsilon-50 at bottom of layer                     =     0.007000
```

(Depth of the lowest soil layer extends 0.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Angle of Friction deg.	E50 or krm	kpy pci
1	Sand	0.00	130.0000	--	40.0000	--	250.0000
	(Reese, et al.)	13.9000	130.0000	--	40.0000	--	250.0000
2	Sand	13.9000	58.0000	--	30.0000	--	35.0000
	(Reese, et al.)	18.4000	58.0000	--	30.0000	--	35.0000
3	Stiff Clay	18.4000	53.0000	1000.0000	--	0.00700	--
	w/o Free Water	30.0000	53.0000	1000.0000	--	0.00700	--

Static Loading Type

Static loading criteria were used when computing p-y curves for all analyses.

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 2

Load No.	Load Type	Condition 1	Condition 2	Axial Thrust Force, lbs	Compute Top y vs. Pile Length	Run Analysis
1	5	y = 1.375000 in	S = 0.0000 in/in	410000.	N.A.	Yes
2	4	y = 1.375000 in	M = -3350911. in-lbs	410000.	N.A.	Yes

V = shear force applied normal to pile axis
M = bending moment applied to pile head
y = lateral deflection normal to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applied to pile head
Values of top y vs. pile lengths can be computed only for load types with specified shear loading (Load Types 1, 2, and 3).
Thrust force is assumed to be acting axially for all pile batter angles.

Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness

---

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Dimensions and Properties of Steel H Weak Axis:

Length of Section	=	30.000000 ft
Flange Width	=	14.900000 in
Section Depth	=	14.200000 in
Flange Thickness	=	0.805000 in
Web Thickness	=	0.805000 in
Yield Stress of Pipe	=	50.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	34.123950 sq. in.
Moment of Inertia	=	444.363799 in^4
Elastic Bending Stiffness	=	12886550. kip-in^2
Plastic Modulus, Z	=	91.398684in^3
Plastic Moment Capacity = Fy Z	=	4570.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1706.197 kips
Nominal Axial Tensile Capacity	=	-1706.197 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	410.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 410.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000443	57.0960271	12886018.	100.9559856	12.9627342	
0.00000886	114.1920543	12886018.	54.2029928	13.9104468	
0.00001329	171.2880814	12886018.	38.6186619	14.8581592	
0.00001772	228.3841085	12886018.	30.8264964	15.8058717	
0.00002215	285.4801356	12886018.	26.1511971	16.7535843	
0.00002659	342.5761628	12886018.	23.0343309	17.7012968	
0.00003102	399.6721899	12886018.	20.8079979	18.6490093	
0.00003545	456.7682170	12886018.	19.1382482	19.5967219	
0.00003988	513.8642441	12886018.	17.8395540	20.5444344	
0.00004431	570.9602713	12886018.	16.8005986	21.4921470	
0.00004874	628.0562984	12886018.	15.9505441	22.4398594	
0.00005317	685.1523255	12886018.	15.2421655	23.3875720	
0.00005760	742.2483526	12886018.	14.6427681	24.3352845	
0.00006203	799.3443798	12886018.	14.1289990	25.2829970	
0.00006646	856.4404069	12886018.	13.6837324	26.2307096	
0.00007089	913.5364340	12886018.	13.2941241	27.1784221	
0.00007532	970.6324612	12886018.	12.9503521	28.1261346	
0.00007976	1028.	12886018.	12.6447770	29.0738471	
0.00008419	1085.	12886018.	12.3713677	30.0215597	
0.00008862	1142.	12886018.	12.1252993	30.9692722	
0.00009305	1199.	12886018.	11.9026660	31.9169847	
0.00009748	1256.	12886018.	11.7002721	32.8646972	
0.0001019	1313.	12886018.	11.5154776	33.8124098	
0.0001063	1370.	12886018.	11.3460827	34.7601223	
0.0001108	1427.	12886018.	11.1902394	35.7078348	
0.0001152	1484.	12886018.	11.0463841	36.6555473	
0.0001196	1542.	12886018.	10.9131847	37.6032599	
0.0001241	1599.	12886018.	10.7894995	38.5509724	
0.0001285	1656.	12886018.	10.6743443	39.4986849	
0.0001329	1713.	12886018.	10.5668662	40.4463974	
0.0001374	1770.	12886018.	10.4663221	41.3941100	
0.0001418	1827.	12886018.	10.3720621	42.3418225	
0.0001462	1884.	12886018.	10.2835147	43.2895350	
0.0001506	1941.	12886018.	10.2001760	44.2372475	
0.0001551	1998.	12886018.	10.1215996	45.1849601	
0.0001595	2055.	12886018.	10.0473885	46.1326726	
0.0001639	2113.	12886018.	9.9771888	47.0803851	
0.0001684	2170.	12886018.	9.9106838	48.0280977	
0.0001728	2227.	12886018.	9.8475894	48.9758102	
0.0001817	2339.	12876578.	9.7319297	50.0000000	Y
0.0001905	2444.	12828475.	9.6325790	50.0000000	Y
0.0001994	2542.	12750935.	9.5470130	50.0000000	Y
0.0002082	2635.	12651442.	9.4731172	50.0000000	Y
0.0002171	2722.	12536098.	9.4091017	50.0000000	Y
0.0002260	2804.	12409882.	9.3534361	50.0000000	Y
0.0002348	2883.	12274799.	9.3051750	50.0000000	Y
0.0002437	2957.	12135330.	9.2629997	50.0000000	Y
0.0002526	3029.	11992327.	9.2262858	50.0000000	Y
0.0002614	3097.	11848701.	9.1940689	50.0000000	Y
0.0002703	3163.	11703988.	9.1660813	50.0000000	Y
0.0002791	3227.	11560087.	9.1416188	50.0000000	Y
0.0002880	3288.	11417813.	9.1202192	50.0000000	Y
0.0002969	3348.	11277560.	9.1015349	50.0000000	Y
0.0003057	3404.	11135225.	9.0844038	50.0000000	Y
0.0003146	3456.	10986959.	9.0676771	50.0000000	Y
0.0003235	3504.	10834546.	9.0512930	50.0000000	Y
0.0003323	3549.	10679271.	9.0351400	50.0000000	Y
0.0003412	3590.	10522867.	9.0192571	50.0000000	Y
0.0003500	3628.	10365954.	9.0038004	50.0000000	Y
0.0003589	3664.	10208997.	8.9886718	50.0000000	Y
0.0003678	3697.	10053411.	8.9736823	50.0000000	Y
0.0003766	3728.	9899561.	8.9589475	50.0000000	Y
0.0003855	3757.	9747283.	8.9447210	50.0000000	Y
0.0003943	3785.	9598168.	8.9305960	50.0000000	Y
0.0004032	3811.	9451057.	8.9170017	50.0000000	Y
0.0004121	3835.	9306266.	8.9032592	50.0000000	Y
0.0004209	3858.	9164398.	8.8900200	50.0000000	Y
0.0004298	3879.	9026243.	8.8770717	50.0000000	Y
0.0004387	3899.	8889551.	8.8641553	50.0000000	Y
0.0004475	3919.	8756997.	8.8514541	50.0000000	Y
0.0004564	3937.	8626674.	8.8392739	50.0000000	Y
0.0004652	3954.	8499564.	8.8269396	50.0000000	Y
0.0004741	3971.	8375615.	8.8152008	50.0000000	Y
0.0004830	3986.	8253806.	8.8033506	50.0000000	Y
0.0004918	4002.	8136089.	8.7918169	50.0000000	Y
0.0005007	4015.	8019623.	8.7804579	50.0000000	Y
0.0005095	4029.	7907009.	8.7694761	50.0000000	Y
0.0005184	4042.	7796593.	8.7581957	50.0000000	Y
0.0005273	4054.	7688898.	8.7476372	50.0000000	Y
0.0005627	4098.	7282675.	8.7061484	50.0000000	Y
0.0005982	4136.	6913903.	8.6674959	50.0000000	Y
0.0006336	4167.	6577187.	8.6307083	50.0000000	Y
0.0006691	4195.	6269851.	8.5962337	50.0000000	Y
0.0007045	4219.	5988283.	8.5639449	50.0000000	Y
0.0007400	4240.	5729974.	8.5331459	50.0000000	Y
0.0007754	4258.	5491907.	8.5042310	50.0000000	Y
0.0008108	4275.	5272364.	8.4766239	50.0000000	Y
0.0008463	4290.	5069083.	8.4508029	50.0000000	Y
0.0008817	4303.	4880461.	8.4257503	50.0000000	Y
0.0009172	4315.	4704743.	8.4021622	50.0000000	Y

0.0009526	4326.	4541307.	8.3798809	50.0000000	Y
0.0009881	4336.	4388411.	8.3581407	50.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	410.0000000000	4336.

Note that the values in the above table are not factored by a strength reduction factor for LRFD.

The value of the strength reduction factor depends on the provisions of the LRFD code being followed.

The above values should be multiplied by the appropriate strength reduction factor to compute ultimate moment capacity according to the LRFD structural design standard being followed.

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	541013.
2	13.9000	18.6975	Yes	No	541013.	274588.
3	18.4000	76.8673	No	No	815601.	N.A.

Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Max moment

Computed Values of Pile Loading and Deflection for Lateral Loading for Load Case Number 1

Beyond yield stress, conduct plastic hinge analysis

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 1.375000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 410000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.3750	-4336415.	99225.	0.00	84717.	2.53E+09	0.00	0.00	0.00
0.3000	1.3639	-3975412.	98908.	-0.00591	78665.	2.53E+09	-59.0431	155.8444	0.00
0.6000	1.3324	-3606826.	98562.	-0.00936	72485.	1.05E+10	-133.0477	359.4732	0.00
0.9000	1.2965	-3238130.	97932.	-0.01049	66304.	1.15E+10	-216.5970	601.4325	0.00
1.2000	1.2569	-2870749.	96994.	-0.01141	60145.	1.23E+10	-304.9764	873.5051	0.00
1.5000	1.2143	-2506080.	95734.	-0.01219	54031.	1.28E+10	-395.0591	1171.	0.00
1.8000	1.1692	-2145489.	94155.	-0.01284	47985.	1.29E+10	-481.8152	1484.	0.00
2.1000	1.1218	-1790258.	92262.	-0.01339	42030.	1.29E+10	-569.7077	1828.	0.00
2.4000	1.0727	-1441672.	90062.	-0.01384	36185.	1.29E+10	-652.5309	2190.	0.00
2.7000	1.0222	-1100948.	87573.	-0.01420	30473.	1.29E+10	-730.2227	2572.	0.00
3.0000	0.9705	-769234.	84822.	-0.01446	24912.	1.29E+10	-798.3252	2961.	0.00
3.3000	0.9181	-447549.	81818.	-0.01463	19518.	1.29E+10	-870.5130	3413.	0.00
3.6000	0.8652	-136961.	78555.	-0.01471	14311.	1.29E+10	-942.5322	3922.	0.00
3.9000	0.8122	161468.	75006.	-0.01471	14722.	1.29E+10	-1029.	4561.	0.00
4.2000	0.7593	446496.	71120.	-0.01462	19501.	1.29E+10	-1130.	5356.	0.00
4.5000	0.7069	716698.	66875.	-0.01446	24031.	1.29E+10	-1229.	6257.	0.00
4.8000	0.6552	970682.	62280.	-0.01422	28289.	1.29E+10	-1324.	7276.	0.00
5.1000	0.6045	1207104.	57305.	-0.01392	32253.	1.29E+10	-1440.	8574.	0.00
5.4000	0.5550	1424370.	51869.	-0.01355	35895.	1.29E+10	-1581.	10253.	0.00
5.7000	0.5069	1620565.	46107.	-0.01313	39185.	1.29E+10	-1621.	11510.	0.00
6.0000	0.4605	1795087.	40225.	-0.01265	42111.	1.29E+10	-1647.	12876.	0.00
6.3000	0.4159	1947523.	34266.	-0.01213	44666.	1.29E+10	-1664.	14402.	0.00
6.6000	0.3732	2077595.	28250.	-0.01156	46847.	1.29E+10	-1678.	16192.	0.00
6.9000	0.3326	2185059.	22198.	-0.01097	48649.	1.29E+10	-1684.	18225.	0.00
7.2000	0.2942	2269800.	16143.	-0.01035	50069.	1.29E+10	-1680.	20560.	0.00
7.5000	0.2581	2331830.	10115.	-0.00970	51109.	1.29E+10	-1669.	23277.	0.00
7.8000	0.2243	2371269.	4143.	-0.00905	51771.	1.29E+10	-1649.	26455.	0.00
8.1000	0.1930	2388363.	-1730.	-0.00838	52057.	1.29E+10	-1614.	30118.	0.00
8.4000	0.1640	2383546.	-7454.	-0.00771	51976.	1.29E+10	-1566.	34365.	0.00
8.7000	0.1375	2357454.	-12976.	-0.00705	51539.	1.29E+10	-1502.	39339.	0.00
9.0000	0.1133	2310921.	-18242.	-0.00639	50759.	1.29E+10	-1423.	45236.	0.00
9.3000	0.09142	2244988.	-23196.	-0.00576	49653.	1.29E+10	-1329.	52341.	0.00

L1=3.7'

9.6000	0.07182	2160904.	-27782.	-0.00514	48244.	1.29E+10	-1219.	61090.	0.00
9.9000	0.05439	2060135.	-31940.	-0.00455	46554.	1.29E+10	-1091.	72206.	0.00
10.2000	0.03904	1944377.	-35602.	-0.00399	44614.	1.29E+10	-943.5904	87018.	0.00
10.5000	0.02564	1815589.	-38690.	-0.00347	42454.	1.29E+10	-772.0823	108411.	0.00
10.8000	0.01407	1676047.	-40900.	-0.00298	40115.	1.29E+10	-455.7441	116640.	0.00
11.1000	0.00418	1529906.	-41971.	-0.00253	37665.	1.29E+10	-139.1750	119880.	0.00
11.4000	-0.00417	1381331.	-41965.	-0.00213	35174.	1.29E+10	142.5670	123120.	0.00
11.7000	-0.01113	1234034.	-41005.	-0.00176	32704.	1.29E+10	390.5725	126360.	0.00
12.0000	-0.01685	1091290.	-39211.	-0.00144	30311.	1.29E+10	606.4235	129600.	0.00
12.3000	-0.02147	955956.	-36693.	-0.00115	28042.	1.29E+10	792.0664	132840.	0.00
12.6000	-0.02512	830492.	-33558.	-9.00E-04	25939.	1.29E+10	949.6831	136080.	0.00
12.9000	-0.02795	716994.	-29902.	-6.84E-04	24036.	1.29E+10	1082.	139320.	0.00
13.2000	-0.03005	617217.	-25813.	-4.98E-04	22363.	1.29E+10	1190.	142560.	0.00
13.5000	-0.03153	532608.	-21373.	-3.37E-04	20944.	1.29E+10	1277.	145800.	0.00
13.8000	-0.03248	464329.	-16654.	-1.98E-04	19800.	1.29E+10	1345.	149040.	0.00
14.1000	-0.03296	413283.	-13882.	-7.53E-05	18944.	1.29E+10	195.1649	21319.	0.00
14.4000	-0.03302	364597.	-13172.	3.34E-05	18128.	1.29E+10	199.6998	21773.	0.00
14.7000	-0.03272	318349.	-12449.	1.29E-04	17352.	1.29E+10	201.9866	22226.	0.00
15.0000	-0.03209	274587.	-11721.	2.12E-04	16619.	1.29E+10	202.1799	22680.	0.00
15.3000	-0.03119	233332.	-10996.	2.83E-04	15927.	1.29E+10	200.4413	23134.	0.00
15.6000	-0.03006	194578.	-10281.	3.42E-04	15277.	1.29E+10	196.9385	23587.	0.00
15.9000	-0.02873	158297.	-9581.	3.92E-04	14669.	1.29E+10	191.8429	24041.	0.00
16.2000	-0.02724	124437.	-8902.	4.31E-04	14101.	1.29E+10	185.3289	24494.	0.00
16.5000	-0.02562	92927.	-8249.	4.61E-04	13573.	1.29E+10	177.5722	24948.	0.00
16.8000	-0.02392	63680.	-7626.	4.83E-04	13083.	1.29E+10	168.7493	25402.	0.00
17.1000	-0.02214	36594.	-7036.	4.97E-04	12629.	1.29E+10	159.0359	25855.	0.00
17.4000	-0.02033	11554.	-6482.	5.04E-04	12209.	1.29E+10	148.6070	26309.	0.00

L2=17.5'-3.7'=13.8'

17.7000	-0.01851	-11565.	-5967.	5.04E-04	12209.	1.29E+10	137.6359	26762.	0.00
18.0000	-0.01671	-32895.	-5492.	4.98E-04	12567.	1.29E+10	126.2940	27216.	0.00
18.3000	-0.01493	-52575.	-5058.	4.86E-04	12896.	1.29E+10	114.7506	27670.	0.00
18.6000	-0.01321	-70746.	-4454.	4.69E-04	13201.	1.29E+10	220.8858	60210.	0.00
18.9000	-0.01156	-86025.	-3672.	4.47E-04	13457.	1.29E+10	213.6293	66556.	0.00
19.2000	-0.00999	-98500.	-2916.	4.21E-04	13666.	1.29E+10	205.9951	74232.	0.00
19.5000	-0.00852	-108265.	-2189.	3.92E-04	13830.	1.29E+10	197.9810	83616.	0.00
19.8000	-0.00717	-115420.	-1492.	3.61E-04	13950.	1.29E+10	189.5796	95231.	0.00
20.1000	-0.00593	-120069.	-824.8652	3.28E-04	14028.	1.29E+10	180.7765	109829.	0.00
20.4000	-0.00481	-122327.	-190.6819	2.94E-04	14066.	1.29E+10	171.5475	128524.	0.00
20.7000	-0.00381	-122311.	409.4403	2.60E-04	14066.	1.29E+10	161.8538	153024.	0.00
21.0000	-0.00293	-120146.	973.7173	2.26E-04	14029.	1.29E+10	151.6334	186093.	0.00
21.3000	-0.00218	-115967.	1500.	1.93E-04	13959.	1.29E+10	140.7848	232504.	0.00
21.6000	-0.00154	-109916.	1986.	1.62E-04	13858.	1.29E+10	129.1315	301285.	0.00
21.9000	-0.00102	-102146.	2428.	1.32E-04	13728.	1.29E+10	116.3387	411970.	0.00
22.2000	-5.93E-04	-92825.	2820.	1.05E-04	13571.	1.29E+10	101.6680	617191.	0.00
22.5000	-2.63E-04	-82149.	3139.	8.03E-05	13392.	1.29E+10	75.4895	1034238.	0.00
22.8000	-1.51E-05	-70461.	3283.	5.89E-05	13196.	1.29E+10	4.3481	1034238.	0.00
23.1000	1.62E-04	-58688.	3207.	4.09E-05	12999.	1.29E+10	-46.4343	1034238.	0.00
23.4000	2.79E-04	-47491.	2979.	2.61E-05	12811.	1.29E+10	-80.2598	1034238.	0.00
23.7000	3.49E-04	-37316.	2674.	1.42E-05	12641.	1.29E+10	-89.0856	918024.	0.00
24.0000	3.82E-04	-28280.	2350.	5.06E-06	12489.	1.29E+10	-91.0836	858847.	0.00

24.3000	3.86E-04	-20412.	2022.	-1.74E-06	12357.	1.29E+10	-91.3201	852140.	0.00
24.6000	3.69E-04	-13720.	1695.	-6.51E-06	12245.	1.29E+10	-90.3251	880573.	0.00
24.9000	3.39E-04	-8192.	1373.	-9.57E-06	12152.	1.29E+10	-88.4100	939011.	0.00
25.2000	3.00E-04	-3807.	1059.	-1.12E-05	12079.	1.29E+10	-85.7796	1028035.	0.00
25.5000	2.58E-04	-532.0221	771.4689	-1.18E-05	12024.	1.29E+10	-74.1186	1034238.	0.00
25.8000	2.15E-04	1783.	526.8399	-1.17E-05	12045.	1.29E+10	-61.7864	1034238.	0.00
26.1000	1.74E-04	3296.	325.6799	-1.10E-05	12070.	1.29E+10	-49.9691	1034238.	0.00
26.4000	1.36E-04	4160.	165.3480	-9.92E-06	12085.	1.29E+10	-39.1042	1034238.	0.00
26.7000	1.02E-04	4515.	41.9665	-8.71E-06	12091.	1.29E+10	-29.4411	1034238.	0.00
27.0000	7.34E-05	4488.	-48.9766	-7.45E-06	12090.	1.29E+10	-21.0828	1034238.	0.00
27.3000	4.88E-05	4185.	-112.1636	-6.24E-06	12085.	1.29E+10	-14.0211	1034238.	0.00
27.6000	2.84E-05	3699.	-152.1051	-5.14E-06	12077.	1.29E+10	-8.1686	1034238.	0.00
27.9000	1.18E-05	3105.	-172.9011	-4.19E-06	12067.	1.29E+10	-3.3848	1034238.	0.00
28.2000	-1.75E-06	2466.	-178.0902	-3.41E-06	12056.	1.29E+10	0.5020	1034238.	0.00
28.5000	-1.28E-05	1833.	-170.5695	-2.81E-06	12046.	1.29E+10	3.6762	1034238.	0.00
28.8000	-2.20E-05	1246.	-152.5750	-2.38E-06	12036.	1.29E+10	6.3208	1034238.	0.00
29.1000	-3.00E-05	741.2121	-125.7079	-2.11E-06	12027.	1.29E+10	8.6054	1034238.	0.00
29.4000	-3.72E-05	347.3812	-91.0019	-1.95E-06	12021.	1.29E+10	10.6758	1034238.	0.00
29.7000	-4.40E-05	91.7649	-49.0231	-1.89E-06	12017.	1.29E+10	12.6458	1034238.	0.00
30.0000	-5.08E-05	0.00	0.00	-1.88E-06	12015.	1.29E+10	14.5893	517119.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.37500000 inches
Computed slope at pile head      =       0.000000 radians
Maximum bending moment           =      -4336415. inch-lbs
Maximum shear force              =         99225. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =             27
Number of zero deflection points =              3
```

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head = 1.375000 inches

plastic moment

Moment at pile head = -3350911.0 in-lbs
Axial load at pile head = 410000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.3750	-3350911.	87294.	-0.00971	68195.	1.12E+10	0.00	0.00	0.00
0.3000	1.3381	-3021535.	87188.	-0.01073	62673.	1.12E+10	-59.0428	158.8445	0.00
0.6000	1.2978	-2691497.	86842.	-0.01159	57139.	1.26E+10	-133.0472	369.0705	0.00
0.9000	1.2546	-2362046.	86213.	-0.01231	51616.	1.29E+10	-216.5963	621.4876	0.00
1.2000	1.2091	-2034426.	85274.	-0.01293	46123.	1.29E+10	-304.9758	908.0122	0.00
1.5000	1.1616	-1709921.	84014.	-0.01345	40683.	1.29E+10	-395.0586	1224.	0.00
1.8000	1.1123	-1389830.	82435.	-0.01388	35316.	1.29E+10	-481.8149	1559.	0.00
2.1000	1.0616	-1075410.	80543.	-0.01423	30045.	1.29E+10	-569.7078	1932.	0.00
2.4000	1.0099	-767930.	78343.	-0.01448	24890.	1.29E+10	-652.5314	2326.	0.00
2.7000	0.9574	-468590.	75854.	-0.01466	19871.	1.29E+10	-730.2237	2746.	0.00
3.0000	0.9044	-178521.	73102.	-0.01475	15008.	1.29E+10	-798.3269	3178.	0.00
3.3000	0.8512	101276.	70098.	-0.01476	13713.	1.29E+10	-870.5153	3682.	0.00
3.6000	0.7981	369748.	66835.	-0.01469	18214.	1.29E+10	-942.5352	4251.	0.00
3.9000	0.7454	625854.	63286.	-0.01455	22508.	1.29E+10	-1029.	4969.	0.00
4.2000	0.6933	868366.	59401.	-0.01434	26574.	1.29E+10	-1130.	5866.	0.00
4.5000	0.6421	1095878.	55155.	-0.01407	30388.	1.29E+10	-1229.	6888.	0.00
4.8000	0.5921	1307017.	50560.	-0.01373	33928.	1.29E+10	-1324.	8052.	0.00
5.1000	0.5433	1500454.	45639.	-0.01334	37171.	1.29E+10	-1410.	9344.	0.00
5.4000	0.4960	1674999.	40483.	-0.01290	40097.	1.29E+10	-1454.	10556.	0.00
5.7000	0.4504	1830006.	35190.	-0.01241	42696.	1.29E+10	-1486.	11880.	0.00
6.0000	0.4067	1964995.	29805.	-0.01188	44959.	1.29E+10	-1506.	13328.	0.00
6.3000	0.3649	2079662.	24365.	-0.01131	46882.	1.29E+10	-1516.	14961.	0.00
6.6000	0.3252	2173819.	18886.	-0.01072	48460.	1.29E+10	-1527.	16909.	0.00
6.9000	0.2877	2247284.	13382.	-0.01010	49692.	1.29E+10	-1530.	19149.	0.00
7.2000	0.2525	2299989.	7881.	-0.00947	50576.	1.29E+10	-1526.	21757.	0.00
7.5000	0.2196	2331969.	2412.	-0.00882	51112.	1.29E+10	-1512.	24796.	0.00
7.8000	0.1890	2343387.	-2984.	-0.00816	51303.	1.29E+10	-1485.	28295.	0.00
8.1000	0.1608	2334588.	-8258.	-0.00751	51156.	1.29E+10	-1445.	32352.	0.00
8.4000	0.1349	2306100.	-13361.	-0.00686	50678.	1.29E+10	-1390.	37100.	0.00
8.7000	0.1114	2258642.	-18243.	-0.00622	49882.	1.29E+10	-1322.	42723.	0.00
9.0000	0.09010	2193124.	-22851.	-0.00560	48784.	1.29E+10	-1238.	49483.	0.00
9.3000	0.07103	2110652.	-27132.	-0.00500	47401.	1.29E+10	-1140.	57782.	0.00
9.6000	0.05409	2012534.	-31031.	-0.00443	45756.	1.29E+10	-1026.	68270.	0.00
9.9000	0.03917	1900291.	-34486.	-0.00388	43874.	1.29E+10	-893.5691	82122.	0.00
10.2000	0.02616	1775685.	-37426.	-0.00337	41785.	1.29E+10	-739.8466	101795.	0.00
10.5000	0.01494	1640758.	-39605.	-0.00289	39523.	1.29E+10	-470.7212	113400.	0.00
10.8000	0.00537	1499053.	-40766.	-0.00245	37147.	1.29E+10	-174.0696	116640.	0.00
11.1000	-0.00269	1354475.	-40918.	-0.00205	34724.	1.29E+10	89.6048	119880.	0.00
11.4000	-0.00939	1210499.	-40178.	-0.00169	32310.	1.29E+10	321.2043	123120.	0.00
11.7000	-0.01488	1070187.	-38660.	-0.00137	29957.	1.29E+10	522.1333	126360.	0.00
12.0000	-0.01928	936201.	-36471.	-0.00109	27711.	1.29E+10	694.1849	129600.	0.00
12.3000	-0.02275	810825.	-33710.	-8.49E-04	25609.	1.29E+10	839.4256	132840.	0.00

L1=3.1'

Max moment for lower segment check

12.6000	-0.02540	695994.	-30471.	-6.39E-04	23684.	1.29E+10	960.0795	136080.	0.00
12.9000	-0.02735	593318.	-26838.	-4.59E-04	21962.	1.29E+10	1058.	139320.	0.00
13.2000	-0.02870	504115.	-22887.	-3.06E-04	20467.	1.29E+10	1137.	142560.	0.00
13.5000	-0.02955	429435.	-18687.	-1.75E-04	19215.	1.29E+10	1197.	145800.	0.00
13.8000	-0.02996	370087.	-14300.	-6.35E-05	18220.	1.29E+10	1241.	149040.	0.00
14.1000	-0.03001	326664.	-11747.	3.38E-05	17492.	1.29E+10	177.6977	21319.	0.00
14.4000	-0.02972	285409.	-11104.	1.19E-04	16800.	1.29E+10	179.7482	21773.	0.00
14.7000	-0.02915	246366.	-10456.	1.94E-04	16145.	1.29E+10	179.9543	22226.	0.00
15.0000	-0.02833	209553.	-9811.	2.57E-04	15528.	1.29E+10	178.4549	22680.	0.00
15.3000	-0.02729	174967.	-9174.	3.11E-04	14948.	1.29E+10	175.3944	23134.	0.00
15.6000	-0.02609	142582.	-8551.	3.55E-04	14405.	1.29E+10	170.9209	23587.	0.00
15.9000	-0.02474	112353.	-7946.	3.91E-04	13899.	1.29E+10	165.1854	24041.	0.00
16.2000	-0.02327	84219.	-7363.	4.18E-04	13427.	1.29E+10	158.3406	24494.	0.00
16.5000	-0.02172	58102.	-6807.	4.38E-04	12989.	1.29E+10	150.5399	24948.	0.00
16.8000	-0.02012	33912.	-6281.	4.51E-04	12584.	1.29E+10	141.9366	25402.	0.00
17.1000	-0.01847	11547.	-5787.	4.58E-04	12209.	1.29E+10	132.6834	25855.	0.00
17.4000	-0.01682	-9103.	-5326.	4.58E-04	12168.	1.29E+10	122.9316	26309.	0.00
17.7000	-0.01518	-28155.	-4902.	4.53E-04	12487.	1.29E+10	112.8314	26762.	0.00
18.0000	-0.01356	-45734.	-4514.	4.42E-04	12782.	1.29E+10	102.5310	27216.	0.00
18.3000	-0.01199	-61965.	-4164.	4.27E-04	13054.	1.29E+10	92.1771	27670.	0.00
18.6000	-0.01049	-76976.	-3623.	4.08E-04	13306.	1.29E+10	208.5064	71585.	0.00
18.9000	-0.00906	-89253.	-2886.	3.85E-04	13511.	1.29E+10	201.0032	79904.	0.00
19.2000	-0.00772	-98888.	-2176.	3.58E-04	13673.	1.29E+10	193.1152	90100.	0.00
19.5000	-0.00648	-105980.	-1496.	3.30E-04	13792.	1.29E+10	184.8355	102756.	0.00
19.8000	-0.00534	-110632.	-846.1370	3.00E-04	13870.	1.29E+10	176.1502	118716.	0.00
20.1000	-0.00432	-112956.	-228.4030	2.68E-04	13909.	1.29E+10	167.0353	139227.	0.00
20.4000	-0.00341	-113069.	355.6738	2.37E-04	13911.	1.29E+10	157.4518	166224.	0.00
20.7000	-0.00261	-111094.	904.2926	2.05E-04	13878.	1.29E+10	147.3365	202857.	0.00
21.0000	-0.00193	-107164.	1415.	1.75E-04	13812.	1.29E+10	136.5848	254620.	0.00
21.3000	-0.00136	-101420.	1886.	1.46E-04	13715.	1.29E+10	125.0124	332055.	0.00
21.6000	-8.82E-04	-94014.	2313.	1.18E-04	13591.	1.29E+10	112.2638	458461.	0.00
21.9000	-5.02E-04	-85114.	2691.	9.35E-05	13442.	1.29E+10	97.5317	699024.	0.00
22.2000	-2.09E-04	-74915.	2974.	7.11E-05	13271.	1.29E+10	59.9429	1034238.	0.00
22.5000	9.65E-06	-63908.	3077.	5.17E-05	13086.	1.29E+10	-2.7711	1034238.	0.00
22.8000	1.64E-04	-52911.	2988.	3.54E-05	12902.	1.29E+10	-47.0197	1034238.	0.00
23.1000	2.64E-04	-42501.	2766.	2.21E-05	12728.	1.29E+10	-75.9805	1034238.	0.00
23.4000	3.23E-04	-33058.	2472.	1.15E-05	12569.	1.29E+10	-87.3280	974713.	0.00
23.7000	3.47E-04	-24734.	2155.	3.44E-06	12430.	1.29E+10	-88.9589	921985.	0.00
24.0000	3.47E-04	-17552.	1835.	-2.47E-06	12309.	1.29E+10	-88.9534	922093.	0.00
24.3000	3.30E-04	-11516.	1517.	-6.53E-06	12208.	1.29E+10	-87.7955	959010.	0.00
24.6000	3.00E-04	-6613.	1204.	-9.06E-06	12126.	1.29E+10	-85.7748	1028355.	0.00
24.9000	2.64E-04	-2819.	913.1238	-1.04E-05	12062.	1.29E+10	-75.9376	1034238.	0.00
25.2000	2.26E-04	-8.3140	659.8051	-1.08E-05	12015.	1.29E+10	-64.7951	1034238.	0.00
25.5000	1.87E-04	1963.	446.6036	-1.05E-05	12048.	1.29E+10	-53.6502	1034238.	0.00
25.8000	1.50E-04	3238.	272.5028	-9.78E-06	12069.	1.29E+10	-43.0725	1034238.	0.00
26.1000	1.16E-04	3954.	134.7976	-8.77E-06	12081.	1.29E+10	-33.4304	1034238.	0.00
26.4000	8.68E-05	4235.	29.7475	-7.63E-06	12086.	1.29E+10	-24.9308	1034238.	0.00
26.7000	6.15E-05	4191.	-46.9065	-6.45E-06	12085.	1.29E+10	-17.6547	1034238.	0.00
27.0000	4.03E-05	3916.	-99.5460	-5.32E-06	12081.	1.29E+10	-11.5895	1034238.	0.00

L2=17.3'-3.1'=14.2'

27.3000	2.32E-05	3490.	-132.3872	-4.28E-06	12074.	1.29E+10	-6.6557	1034238.	0.00
27.6000	9.50E-06	2975.	-149.2818	-3.38E-06	12065.	1.29E+10	-2.7302	1034238.	0.00
27.9000	-1.17E-06	2425.	-153.5919	-2.63E-06	12056.	1.29E+10	0.3356	1034238.	0.00
28.2000	-9.40E-06	1877.	-148.1263	-2.02E-06	12046.	1.29E+10	2.7008	1034238.	0.00
28.5000	-1.57E-05	1364.	-135.1222	-1.57E-06	12038.	1.29E+10	4.5236	1034238.	0.00
28.8000	-2.07E-05	909.0843	-116.2657	-1.25E-06	12030.	1.29E+10	5.9522	1034238.	0.00
29.1000	-2.48E-05	530.7623	-92.7389	-1.05E-06	12024.	1.29E+10	7.1182	1034238.	0.00
29.4000	-2.83E-05	244.4732	-65.2907	-9.45E-07	12019.	1.29E+10	8.1308	1034238.	0.00
29.7000	-3.16E-05	63.4583	-34.3244	-9.02E-07	12016.	1.29E+10	9.0727	1034238.	0.00
30.0000	-3.48E-05	0.00	0.00	-8.93E-07	12015.	1.29E+10	9.9964	517119.	0.00

* This analysis computed pile response using nonlinear moment-curvature relationships. Values of total stress due to combined axial and bending stresses are computed only for elastic sections only and do not equal the actual stresses in concrete and steel. Stresses in concrete and steel may be interpolated from the output for nonlinear bending properties relative to the magnitude of bending moment developed in the pile.

Output Summary for Load Case No. 2:

```
Pile-head deflection             =     1.37500000 inches
Computed slope at pile head      =    -0.00970621 radians
Maximum bending moment           =      -3350911. inch-lbs
Maximum shear force              =         87294. lbs
Depth of maximum bending moment  =       0.000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =             29
Number of zero deflection points =              3
```

Summary of Pile-head Responses for Conventional Analyses

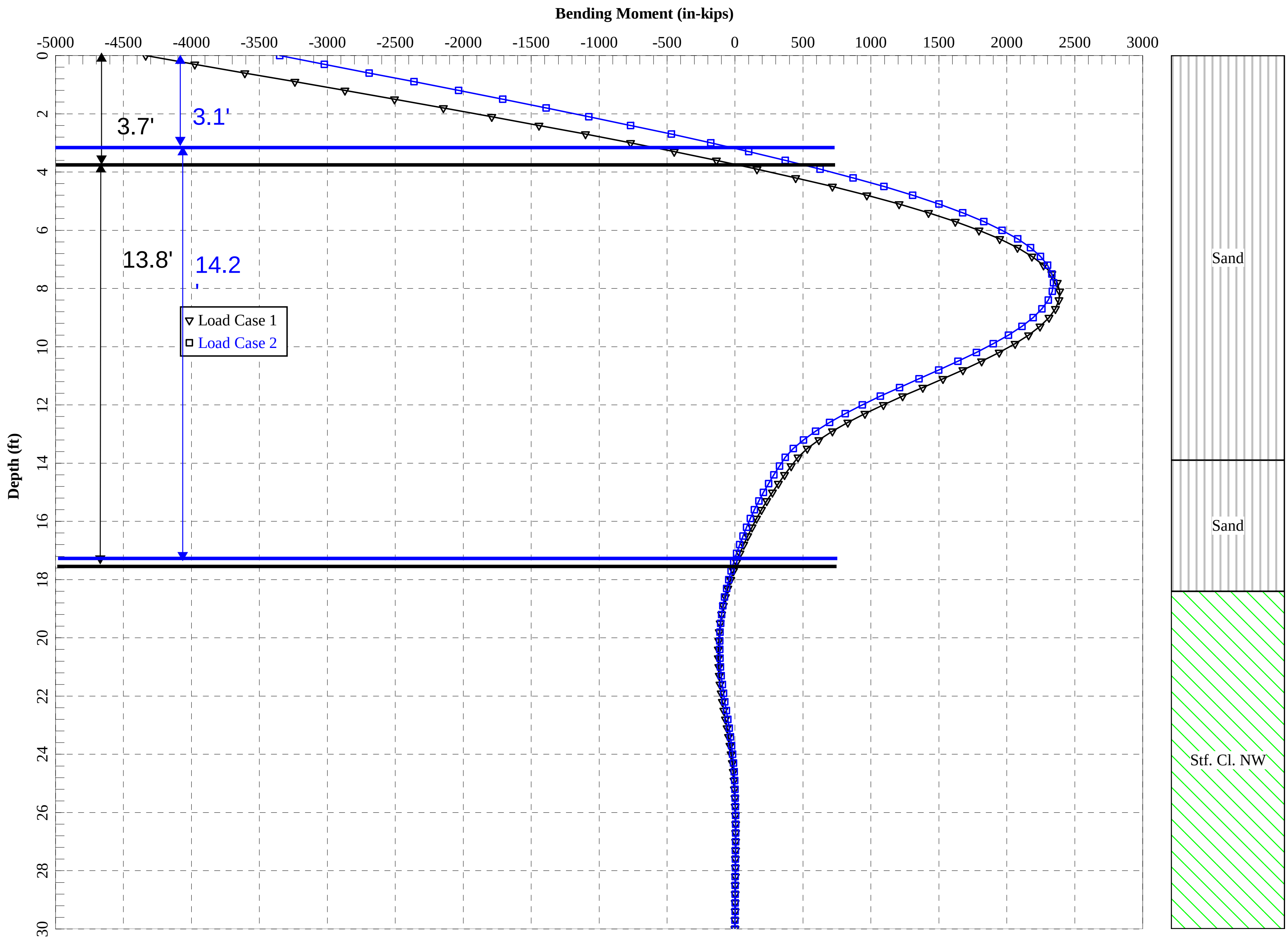
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

Load Case No.	Load Type 1	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.3750	S, rad	0.00	410000.	1.3750	0.00	99225.	-4336415.
2	y, in	1.3750	M, in-lb	-3350911.	410000.	1.3750	-0.00971	87294.	-3350911.

Maximum pile-head deflection = 1.3750000000 inches
Maximum pile-head rotation   = -0.0097062110 radians = -0.556125 deg.

The analysis ended normally.



Bending Moment (in-kips)
-5000
-4500
-4000
-3500
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
2500
3000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
3.7'
3.1'
13.8'
14.2'
Load Case 1
Load Case 2
Sand
Sand
Stf. Cl. NW

Pile Group Analysis
Pier

		Sheet #	3
		Job #	
Program:	LEAP® Bridge Concrete CONNECT Edition	Hoyle Tanner & Associates Inc	Designed	
Module:	Substructure	Copyright © Bentley Systems, Inc. 2016	Date	Jan/18/2022
Version:	21.00.00.28	www.bentley.com Phone: 1-800-778-4277	Checked	
File Name:	**project.lbcx**		Date	

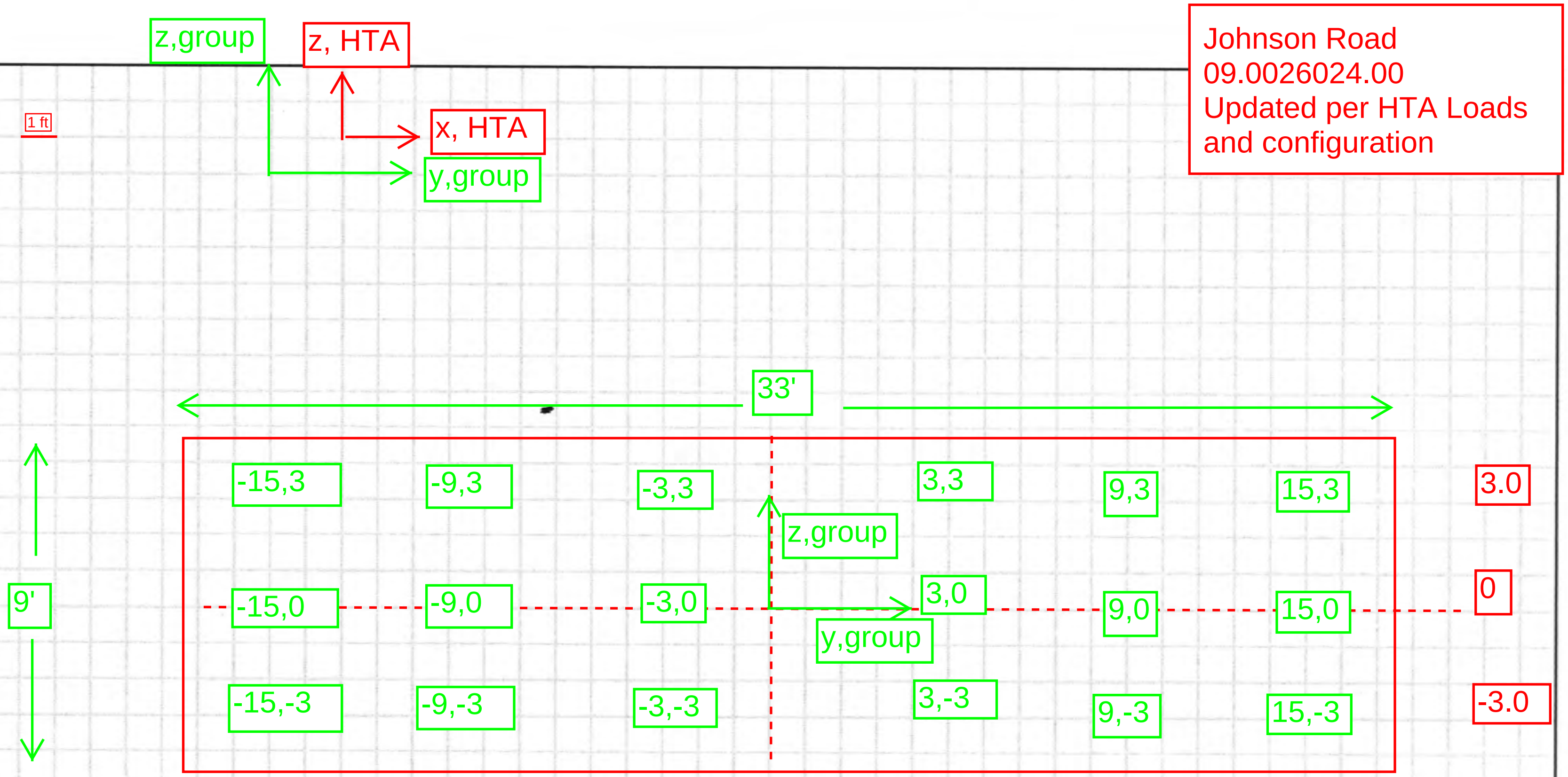
Pile Reactions, Factored **Load Effect @ Footing Bot.**

Pile	X in	Z in	Batter X degree	Batter Z degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	234.0	-0.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	190.88
					202	---	-1581.05	1405.50	2863.13	83.29
2	306.0	-0.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	216.74
					202	---	-1581.05	1405.50	2863.13	74.20
3	378.0	-0.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	242.60
					1471	---	-2079.19	3184.61	8162.44	50.73
4	162.0	-0.0	-0	-0	16	---	-3439.97	4590.47	5643.71	200.07
					209	---	-1680.66	1347.55	-1030.22	91.73
5	90.0	-0.0	-0	-0	18	---	-3203.19	4590.47	9076.71	221.18
					1517	---	-2079.19	-30.05	-7491.95	79.83
6	18.0	-0.0	-0	-0	18	---	-3203.19	4590.47	9076.71	249.99
					1517	---	-2079.19	-30.05	-7491.95	56.05
7	234.0	-36.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	196.69
					1671	---	-2079.19	4107.27	6140.99	-8.33
8	306.0	-36.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	222.55
					79	---	-2197.27	3977.08	8418.18	-28.49
9	378.0	-36.0	-0	-0	2	---	-3203.19	-209.14	-8145.46	248.41
					79	---	-2197.27	3977.08	8418.18	-55.21
10	162.0	-36.0	-0	-0	1	---	-3439.97	-209.14	4266.20	203.69
					1667	---	-2079.19	4097.80	-5804.02	-7.53
11	90.0	-36.0	-0	-0	3	---	-3203.19	-209.14	7699.20	220.43
					1667	---	-2079.19	4097.80	-5804.02	-25.96
12	18.0	-36.0	-0	-0	3	---	-3203.19	-209.14	7699.20	244.87
					1667	---	-2079.19	4097.80	-5804.02	-44.38
13	234.0	36.0	-0	-0	17	---	-3203.19	4590.47	-6767.95	316.21
					1351	---	-2079.19	-943.24	6474.51	79.03
14	306.0	36.0	-0	-0	17	---	-3203.19	4590.47	-6767.95	337.70
					1351	---	-2079.19	-943.24	6474.51	58.48
15	378.0	36.0	-0	-0	17	---	-3203.19	4590.47	-6767.95	359.18
					1351	---	-2079.19	-943.24	6474.51	37.92
16	162.0	36.0	-0	-0	16	---	-3439.97	4590.47	5643.71	327.58
					1357	---	-2079.19	-952.71	-5470.49	80.36
17	90.0	36.0	-0	-0	18	---	-3203.19	4590.47	9076.71	348.69
					65	---	-2197.27	-822.52	-7747.68	62.33
18	18.0	36.0	-0	-0	18	---	-3203.19	4590.47	9076.71	377.51
					65	---	-2197.27	-822.52	-7747.68	37.73

Pile Reactions: Notes

Load effects on pile are calculated at centroid of the bottom of the footing.
Both the max. and min. pile reaction are reported for each individual pile.
Positive pile reaction represents pile subject to compression load; negative pile reaction represents pile subject to uplift.
Coordinate system of pile layout see Geometry Tab>Footing Pile>Edit Pile.

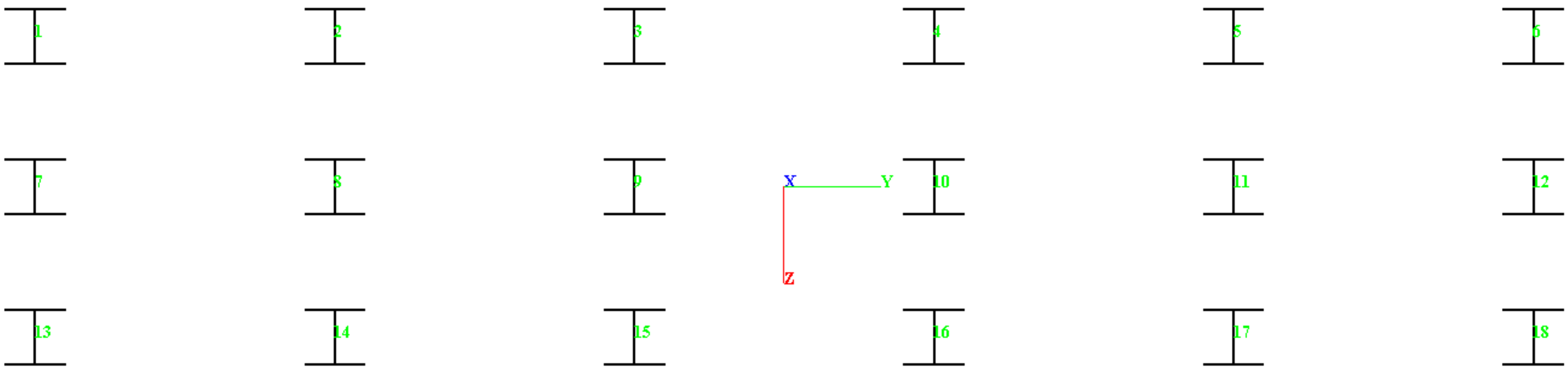
Johnson Road
09.0026024.00
Updated per HTA Loads and configuration



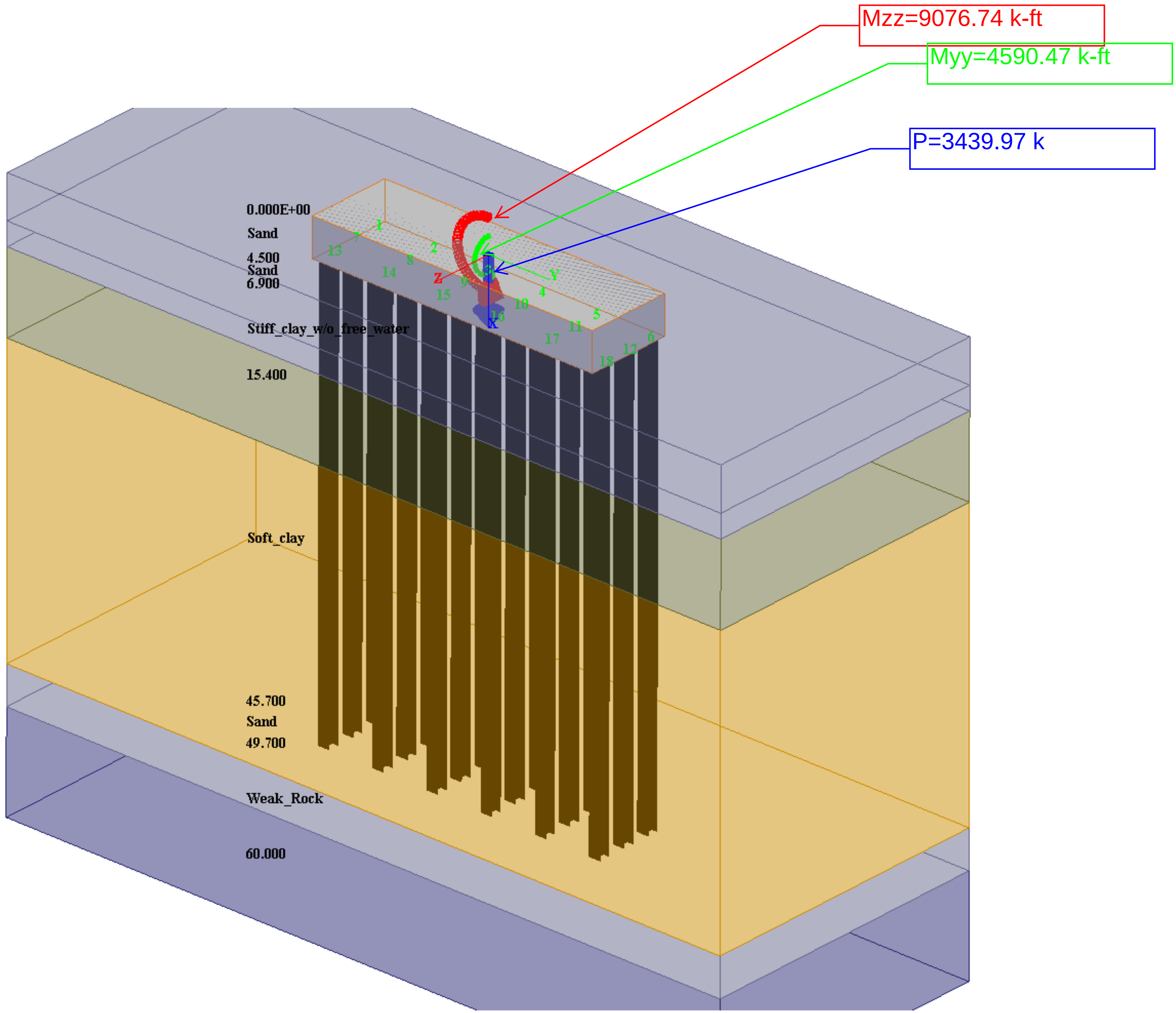
Applied loading in GZA Group Coordinate. system
P(Group)x = 3439.97
Myy = 4590.47 k-ft = 55085.64 k-in
Mzz = 9076.71 k-ft = 108920.52 k-in

GZA
GeoEnvironmental, Inc.
707 Sable Oaks Drive, Suite 150
South Portland, ME 04106
(207) 879-9190
http://www.gza.com
Engineers and Scientists

JOB
SHEET NO ____ OF ____
Calculated By ____ Date ____
Checked By ____ Date ____
Scale



1
2
3
4
5
6
7
8
9
X
Y
Z
10
11
12
13
14
15
16
17
18



Mzz=9076.74 k-ft
Myy=4590.47 k-ft
P=3439.97 k
0.000E+00
Sand
4.500
Sand
6.900
Stiff_clay_w/o_free_water
15.400
Soft_clay
45.700
Sand
49.700
Weak_Rock
60.000

```
========================================================================

                GROUP for Windows, Version 2016.10.13

                 Serial Number : 364300562

                 Analysis of A Group of Piles
               Subjected to Axial and Lateral Loading

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========================================================================


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GZA GeoEnvironmental, Inc.
Portland, ME

Path to file locations      : P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd - Falmouth\Work\Calcs\Group\UpdatedProfile_02.10.2022\
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Name of output summary file : 26024.00_PierGroup_02.10.2022.gp10t
Name of plot output file    : 26024.00_PierGroup_02.10.2022.gp10p
Name of runtime file        : 26024.00_PierGroup_02.10.2022.gp10r


------------------------------------------------------------------------
                    Time and Date of Analysis
------------------------------------------------------------------------

          Date:  February 10, 2022     Time:  11:14:41




           *****     INPUT INFORMATION     *****


New Group

ANALYSIS TYPE = 3D ANALYSIS

ADJUST DEPTH FOR BATTER PILES

GENERATE LOAD-DISP (AND T-R) CURVES BASED ON SOIL PROFILE

EXTEND INTERPOLATION FOR L-DP (AND T-R) CURVES

UNITS SYSTEM  = ENGL
```

```
 * TABLE B *  PILE CAP OPTIONS

 LENGTH,YY    ( FT ) =    31.00
 WIDTH, ZZ    ( FT ) =    9.000
 THICKNESS,XX ( FT ) =    4.000

 * PILE CAP DIMENSIONS ARE NOT CONSIDERED
   FOR THE PILE GROUP ANALYSIS



 * TABLE C *  LOAD AND CONTROL PARAMETERS


 ** LOAD CASES **


 NUMBER OF LOAD CASES :    1


 LOAD CASE    :     1
 CASE NAME    : Max. Factored Load for Strength (HTA Output)
 LOAD TYPE    : Dead, DL
 SCALE FACTOR :       1.0000

 * CONCENTRATED LOADS *

NL  VERT.LOAD HR.LOAD Y HR.LOAD Z  MOMENT X  MOMENT Y  MOMENT Z   COORD X   COORD Y   COORD Z
       KIP       KIP       KIP      KIP-IN    KIP-IN    KIP-IN      FT        FT        FT
 1   3.44E+03      0.00      0.00      0.00  5.51E+04  1.09E+05      0.00      0.00      0.00


 * EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

       VER.LOAD X,KIP     HOR.LOAD Y,KIP     HOR.LOAD Z,KIP
              3439.97           0.00000           0.00000

       MOMENT X,KIP-IN    MOMENT Y,KIP-IN    MOMENT Z,KIP-IN
              0.00000           55085.6       1.08921E+05


 * THE LOADING IS STATIC *


 * CONTROL PARAMETERS *
     TOLERANCE ON CONVERGENCE OF PILE CAP MOVEMENT        =  1.00000E-04
     TOLERANCE ON DETERMINATION OF PILE DEFLECTIONS       =  1.00000E-03 IN
     MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =      100
     MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS   =      100
     FACTOR TO APPLY THE LOAD IN INCREMENTS               =   1.0000
     MINIMUM FACTOR FOR LOAD INCREMENTS                   =   1.0000
     PRINT RESULTS AT PILE CAP AND PILE HEADS
```

* TABLE D *   ARRANGEMENT OF PILE GROUPS

GROUP	CONN.Z-Z	CONN.Y-Y	PILE PROP	P-Y CURVE	L-S CURVE	T-R CURVE	R-F-L SET
1	FIX	FIX	1	0	1 G	1 G	0
2	FIX	FIX	1	0	1 G	1 G	0
3	FIX	FIX	1	0	1 G	1 G	0
4	FIX	FIX	1	0	1 G	1 G	0
5	FIX	FIX	1	0	1 G	1 G	0
6	FIX	FIX	1	0	1 G	1 G	0
7	FIX	FIX	1	0	1 G	1 G	0
8	FIX	FIX	1	0	1 G	1 G	0
9	FIX	FIX	1	0	1 G	1 G	0
10	FIX	FIX	1	0	1 G	1 G	0
11	FIX	FIX	1	0	1 G	1 G	0
12	FIX	FIX	1	0	1 G	1 G	0
13	FIX	FIX	1	0	1 G	1 G	0
14	FIX	FIX	1	0	1 G	1 G	0
15	FIX	FIX	1	0	1 G	1 G	0
16	FIX	FIX	1	0	1 G	1 G	0
17	FIX	FIX	1	0	1 G	1 G	0
18	FIX	FIX	1	0	1 G	1 G	0

GROUP	CorX,FT	CorY,FT	CorZ,FT	ALPHA,DEG	BETA,DEG	THETA,DEG	GROUND,FT	SPz,KIP-IN	SPy,KIP-IN
1	4.000	-15.00	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
2	4.000	-9.000	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
3	4.000	-3.000	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
4	4.000	3.000	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
5	4.000	9.000	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
6	4.000	15.00	-3.000	0.000	90.00	0.000	-4.000	0.000	0.000
7	4.000	-15.00	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
8	4.000	-9.000	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
9	4.000	-3.000	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
10	4.000	3.000	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
11	4.000	9.000	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
12	4.000	15.00	0.000	0.000	90.00	0.000	-4.000	0.000	0.000
13	4.000	-15.00	3.000	0.000	90.00	0.000	-4.000	0.000	0.000
14	4.000	-9.000	3.000	0.000	90.00	0.000	-4.000	0.000	0.000
15	4.000	-3.000	3.000	0.000	90.00	0.000	-4.000	0.000	0.000
16	4.000	3.000	3.000	0.000	90.00	0.000	-4.000	0.000	0.000
17	4.000	9.000	3.000	0.000	90.00	0.000	-4.000	0.000	0.000
18	4.000	15.00	3.000	0.000	90.00	0.000	-4.000	0.000	0.000

* TABLE E *   PILE GEOMETRY AND PROPERTIES
PILE TYPE = 1 - DRIVEN PILE
= 2 - DRILLED SHAFT

PROP	SECTS	INC	PILE TYPE	LENGTH, FT
1	1	100	1	45.700

* PILE SECTIONS *

PROP	SECT	FROM,FT	TO,FT	CROSS SECT
1	1	0.00000	45.7000	1

* PILE CROSS SECTIONS *

CROSS SECTION : 1
SECTION NAME : Section
TYPE : ELASTIC
CROSS SECTION TYPE : AISC SECTION (HP)
AISC SECTION NAME : HP14X89
EQUIVALENT DIAMETER : 14.2500 IN
EXTERNAL WIDTH : 14.7000 IN
EXTERNAL DEPTH : 13.8000 IN
FLANGE THICKNESS : 0.61500 IN
WEB THICKNESS : 0.61500 IN
YOUNG MODULUS : 29000.0 KIP/IN**2
SHEAR MODULUS : 11153.8 KIP/IN**2

* PILE CROSS SECTIONS PROPERTIES *

ELASTIC SECTIONS

SECT	DIAM,IN	AREA,IN**2	Iz,IN**4	Iy,IN**4	GJ,KIP-IN**2	Mn,KIP-IN	Vn,KIP
1	14.250	26.100	326.00	904.00	4.0043E+04	0.0000	0.0000

* TABLE F *   SOIL DATA

SOILS INFORMATION

GROUND SURFACE = 0.00000 FT

6 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A SAND

		TOP OF LAYER	BOTTOM OF LAYER
X COORDINATE	(FT)	0.00000	4.50000
EFFECTIVE UNIT WEIGHT	(KIP/FT**3)	0.0630000	0.0630000
FRICTION ANGLE	(DEGREES)	32.0000	32.0000
P-Y SUBGRADE MODULUS	(KIP/IN**3)	0.0450000	0.0450000
ULTIMATE UNIT SIDE FRICTION	(KIP/FT**2)	2.14142E-03 (P)	0.11069 (P)
ULTIMATE UNIT TIP RESISTANCE	(KIP/FT**2)	0.00000	0.00000

LAYER 2
THE SOIL IS A SAND

		TOP OF LAYER	BOTTOM OF LAYER
X COORDINATE	(FT)	4.50000	6.90000
EFFECTIVE UNIT WEIGHT	(KIP/FT**3)	0.0580000	0.0580000
FRICTION ANGLE	(DEGREES)	30.0000	30.0000
P-Y SUBGRADE MODULUS	(KIP/IN**3)	0.0350000	0.0350000
ULTIMATE UNIT SIDE FRICTION	(KIP/FT**2)	0.11069 (P)	0.15780 (P)

```
    ULTIMATE UNIT TIP RESISTANCE  (KIP/FT**2)       0.00000          0.00000

    LAYER  3
    THE SOIL IS A STIFF CLAY WITHOUT FREE WATER
                                                 TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                        (FT)        6.90000          15.4000
    EFFECTIVE UNIT WEIGHT         (KIP/FT**3)      0.0530000        0.0530000
    UNDRAINED COHESION, C         (KIP/FT**2)       0.60000          0.60000
    STRAIN AT 50% STRESS                         7.00000E-03      7.00000E-03
    ULTIMATE UNIT SIDE FRICTION   (KIP/FT**2)       0.27485 (P)      0.31342 (P)
    ULTIMATE UNIT TIP RESISTANCE  (KIP/FT**2)       0.00000          0.00000

    LAYER  4
    THE SOIL IS A SOFT CLAY
                                                 TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                        (FT)        15.4000          45.7000
    EFFECTIVE UNIT WEIGHT         (KIP/FT**3)      0.0530000        0.0530000
    UNDRAINED COHESION, C         (KIP/FT**2)       0.45000          0.45000
    STRAIN AT 50% STRESS                         8.00000E-03      8.00000E-03
    ULTIMATE UNIT SIDE FRICTION   (KIP/FT**2)       0.31342 (P)      0.52811 (P)
    ULTIMATE UNIT TIP RESISTANCE  (KIP/FT**2)       0.00000          0.00000

    LAYER  5
    THE SOIL IS A SAND
                                                 TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                        (FT)        45.7000          49.7000
    EFFECTIVE UNIT WEIGHT         (KIP/FT**3)      0.0730000        0.0730000
    FRICTION ANGLE                 (DEGREES)        32.0000          32.0000
    P-Y SUBGRADE MODULUS          (KIP/IN**3)      0.0550000        0.0550000
    ULTIMATE UNIT SIDE FRICTION   (KIP/FT**2)       1.01120 (P)      1.13030 (P)
    ULTIMATE UNIT TIP RESISTANCE  (KIP/FT**2)       69.4160 (R)      77.5922 (R)

    LAYER  6
    THE LAYER IS A WEAK ROCK
                                                 TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                        (FT)        49.7000          60.0000
    EFFECTIVE UNIT WEIGHT         (KIP/FT**3)       0.10800          0.10800
    UNIAXIAL COMPRESSIVE STRENGTH (KIP/IN**2)       10.0000          10.0000
    YOUNG MODULUS,ER              (KIP/IN**2)       0.00000          0.00000
    KRM                                             0.00000          0.00000
    RQD                                  (%)        0.50000          0.50000
    ULTIMATE UNIT SIDE FRICTION   (KIP/FT**2)       144.000 (P)      144.000 (P)
    ULTIMATE UNIT TIP RESISTANCE  (KIP/FT**2)       12960.0 (R)      12960.0 (R)

  Notes :  Program estimated values for listed parameters
           if zero input values were entered:
           (P) ULTIMATE UNIT SIDE FRICTION for Driven Piles
           (R) ULTIMATE UNIT TIP RESISTANCE for Driven Piles


* TABLE H *   AXIAL LOAD VS DISPLACEMENT

AXIAL LOAD-DISPLACEMENT CURVES GENERATED INTERNALLY

  NUM OF CURVES  1

   CURVE  1         NUM OF POINTS 19

  DISPLACEMENT, IN       AXIAL LOAD,KIP
        -2.03082            -70.6466
        -1.03008            -69.5191
        -0.52972            -68.9543
        -0.12712            -66.0405
      -0.0738229            -60.3994
      -0.0153553            -14.2527
    -7.65915E-03            -7.11379
    -1.53188E-03            -1.42286
    -1.53251E-04            -0.14240
         0.00000             0.00000
       0.0208117             37.3741
       0.0454158             80.3740
         0.15001             243.465
         0.21192             330.217
         0.48493             683.620
         0.70635             943.539
         1.81031             2011.25
         2.78840             2736.38
         4.37524             3626.44


* TABLE I *   TORS. MOM. VS ANGLE ROT.

TORQUE-ROTATION CURVES GENERATED INTERNALLY

  NUM OF CURVES  1

   CURVE  1         NUM OF POINTS 19

    ROT. ANGLE,Rad.     TORS.MOMEN,KIP-IN
        -4.51340            -504.529
        -4.26833            -496.533
        -4.14579            -492.535
        -3.71853            -464.420
        -3.52461            -450.257
        -3.11343            -419.260
        -2.98783            -409.632
        -2.65264            -382.850
        -2.14578            -339.401
         0.00000             0.00000
         2.14578             339.401
         2.65264             382.850
         2.98783             409.632
         3.11343             419.260
         3.52461             450.257
         3.71853             464.420
         4.14579             492.535
         4.26833             496.533
         4.51340             504.529
```

* TABLE J *   MOMENT CURVATURE SETS

USER DEFINED MOMENT CURVATURE

NUM OF SETS : 1

CURVE SET 1   NUM OF CURVES 1

CURVE 1   AXIAL LOAD   0.000E+00KIPS

POINT	MOMENT KIPS-IN	CURVATURE RADIAN/IN
1	0.00000	0.00000

* TABLE K *   REDUCTION FACTORS

AVERAGE DIAMETER IS USED TO GET RATIO S/B

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS ALONG Y-DIRECTION
ESTIMATED ASSUMING MOVEMENT IN THE DIRECTION OF Y-FORCE (+)

GROUP NO	P-FACTOR	Y-FACTOR
1	0.7339	1.0000
2	0.7339	1.0000
3	0.7339	1.0000
4	0.7339	1.0000
5	0.7339	1.0000
6	0.8771	1.0000
7	0.6062	1.0000
8	0.6062	1.0000
9	0.6062	1.0000
10	0.6062	1.0000
11	0.6062	1.0000
12	0.7692	1.0000
13	0.7339	1.0000
14	0.7339	1.0000
15	0.7339	1.0000
16	0.7339	1.0000
17	0.7339	1.0000
18	0.8771	1.0000

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS ALONG Z-DIRECTION
ESTIMATED ASSUMING MOVEMENT IN THE DIRECTION OF Z-FORCE (+)

GROUP NO	P-FACTOR	Y-FACTOR
1	0.5978	1.0000
2	0.5893	1.0000
3	0.5893	1.0000
4	0.5893	1.0000
5	0.5893	1.0000
6	0.5978	1.0000
7	0.5994	1.0000
8	0.5909	1.0000
9	0.5909	1.0000
10	0.5909	1.0000
11	0.5909	1.0000
12	0.5994	1.0000
13	0.8907	1.0000
14	0.8907	1.0000
15	0.8907	1.0000
16	0.8907	1.0000
17	0.8907	1.0000
18	0.8907	1.0000

GROUP for Windows, Version 2016.10.13

Serial Number : 364300562

Analysis of A Group of Piles
Subjected to Axial and Lateral Loading

Time and Date of Analysis

Date: February 10, 2022 Time: 11:14:41

***** COMPUTATION RESULTS *****

New Group

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : Max. Factored Load for Strength (HTA Output)
LOAD TYPE : Dead, DL

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	0.6031	1.0000
2	0.5918	1.0000
3	0.5918	1.0000
4	0.5918	1.0000
5	0.5918	1.0000
6	0.6001	1.0000
7	0.6024	1.0000
8	0.5911	1.0000
9	0.5911	1.0000
10	0.5911	1.0000
11	0.5911	1.0000
12	0.5995	1.0000
13	0.8905	1.0000
14	0.8885	1.0000
15	0.8885	1.0000
16	0.8885	1.0000
17	0.8885	1.0000
18	0.8885	1.0000

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

VERT. LOAD,KIPS	HOR. LOAD Y,KIPS	HOR. LOAD Z,KIPS
3439.97	0.00000	0.00000

MOMENT X ,KIP-IN	MOMENT Y,KIP-IN	MOMENT Z,KIP-IN
0.00000	55085.6	1.08921E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.11842	-0.0229626	0.18260

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-1.86592E-06	1.90195E-03	2.60521E-04

Pile cap reactions

* TABLE M * COMPUTATION ON INDIVIDUAL PILE

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.096846	-0.010525	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
2	0.078088	-0.010525	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
3	0.059331	-0.010525	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04
4	0.040573	-0.010525	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04
5	0.021816	-0.010525	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
6	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
7	0.1653	-0.010458	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
8	0.1466	-0.010458	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
9	0.1278	-0.010458	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04

10	0.1090	-0.010458	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04
11	0.090286	-0.010458	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
12	0.071528	-0.010458	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
13	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
14	0.2150	-0.010390	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
15	0.1963	-0.010390	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04
16	0.1775	-0.010390	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04
17	0.1588	-0.010390	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
18	0.1400	-0.010390	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
MINIMUM	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	6	1	6	1	1	1
MAXIMUM	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	13	13	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,KIP	FOR. Y,KIP	FOR. Z,KIP	MOM X,KIP-IN	MOM Y,KIP-IN	MOM Z,KIP-IN
1	160.57	0.030728	-0.4510	-2.9514E-04	554.96	33.699
2	131.32	0.032565	-0.5190	-2.9514E-04	558.08	33.835
3	102.07	0.029117	-0.5209	-2.9514E-04	560.15	33.901
4	71.910	0.025590	-0.5223	-2.9514E-04	562.24	33.970
5	39.128	0.021833	-0.5222	-2.9514E-04	564.42	34.049
6	5.4916	0.014049	-0.4735	-2.9514E-04	565.86	34.076
7	264.91	0.049459	-0.5209	-2.9514E-04	551.77	33.733
8	238.08	0.051452	-0.5895	-2.9514E-04	554.78	33.859
9	208.84	0.048069	-0.5911	-2.9514E-04	556.84	33.928
10	179.59	0.044687	-0.5928	-2.9514E-04	558.90	33.997
11	150.34	0.041306	-0.5946	-2.9514E-04	560.96	34.066
12	121.09	0.034032	-0.5483	-2.9514E-04	562.27	34.084
13	358.52	-0.062951	1.1021	-2.9514E-04	520.23	32.036
14	334.24	-0.065231	1.0823	-2.9514E-04	522.45	32.095
15	308.29	-0.068577	1.0753	-2.9514E-04	524.51	32.146
16	282.01	-0.071952	1.0685	-2.9514E-04	526.58	32.198
17	255.72	-0.075326	1.0617	-2.9514E-04	528.64	32.250
18	227.85	-0.078850	1.0560	-2.9514E-04	530.74	32.307
MINIMUM	5.4916	-0.078850	-0.5946	-2.9514E-04	520.23	32.036
Pile N.	6	18	11	1	13	13
MAXIMUM	358.52	0.051452	1.1021	-2.9514E-04	565.86	34.084
Pile N.	13	8	13	1	6	12

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	0.096846	-0.010525	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
2	0.078088	-0.010525	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
3	0.059331	-0.010525	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04
4	0.040573	-0.010525	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04

5	0.021816	-0.010525	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
6	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
7	0.1653	-0.010458	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
8	0.1466	-0.010458	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
9	0.1278	-0.010458	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04
10	0.1090	-0.010458	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04
11	0.090286	-0.010458	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
12	0.071528	-0.010458	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
13	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
14	0.2150	-0.010390	0.091511	-1.8659E-06	1.9019E-03	2.6052E-04
15	0.1963	-0.010390	0.091377	-1.8659E-06	1.9019E-03	2.6052E-04
16	0.1775	-0.010390	0.091243	-1.8659E-06	1.9019E-03	2.6052E-04
17	0.1588	-0.010390	0.091108	-1.8659E-06	1.9019E-03	2.6052E-04
18	0.1400	-0.010390	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
MINIMUM	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	6	1	6	1	1	1
MAXIMUM	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	13	13	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,KIP	LAT. y,KIP	LAT. z,KIP	MOM x,KIP-IN	MOM y,KIP-IN	MOM z,KIP-IN
1	160.57	0.030728	-0.4510	-2.9514E-04	554.96	33.699
2	131.32	0.032565	-0.5190	-2.9514E-04	558.08	33.835
3	102.07	0.029117	-0.5209	-2.9514E-04	560.15	33.901
4	71.910	0.025590	-0.5223	-2.9514E-04	562.24	33.970
5	39.128	0.021833	-0.5222	-2.9514E-04	564.42	34.049
6	5.4916	0.014049	-0.4735	-2.9514E-04	565.86	34.076
7	264.91	0.049459	-0.5209	-2.9514E-04	551.77	33.733
8	238.08	0.051452	-0.5895	-2.9514E-04	554.78	33.859
9	208.84	0.048069	-0.5911	-2.9514E-04	556.84	33.928
10	179.59	0.044687	-0.5928	-2.9514E-04	558.90	33.997
11	150.34	0.041306	-0.5946	-2.9514E-04	560.96	34.066
12	121.09	0.034032	-0.5483	-2.9514E-04	562.27	34.084
13	358.52	-0.062951	1.1021	-2.9514E-04	520.23	32.036
14	334.24	-0.065231	1.0823	-2.9514E-04	522.45	32.095
15	308.29	-0.068577	1.0753	-2.9514E-04	524.51	32.146
16	282.01	-0.071952	1.0685	-2.9514E-04	526.58	32.198
17	255.72	-0.075326	1.0617	-2.9514E-04	528.64	32.250
18	227.85	-0.078850	1.0560	-2.9514E-04	530.74	32.307
MINIMUM	5.4916	-0.078850	-0.5946	-2.9514E-04	520.23	32.036
Pile N.	6	18	11	1	13	13
MAXIMUM	358.52	0.051452	1.1021	-2.9514E-04	565.86	34.084
Pile N.	13	8	13	1	6	12

Max axial Pile demand = 358.5 kips

Required geotechnical resistance = 358.5/0.65 = 552 kips

PILE GROUP	STRESS,KIP/IN**2
1	10.588
2	9.4917
3	8.3874
4	7.2483
5	6.0095

6	4.7320
7	14.561
8	13.557
9	12.452
10	11.348
11	10.244
12	9.1332
13	17.896
14	16.983
15	16.005
16	15.014
17	14.023
18	12.972
MINIMUM	4.7320
Pile N.	6
MAXIMUM	17.896
Pile N.	13

Max Stress

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR KIP-IN	MOMENT y-DIR KIP-IN	SHEAR y-DIR KIP	SHEAR z-DIR KIP	SOIL REACT y-DIR KIP/IN	SOIL REACT z-DIR KIP/IN	TOTAL STRESS KIP/IN**2	FLEX. RIG. z-DIR KIP-IN**2	FLEX. RIG. y-DIR KIP-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-0.010525	-6.6529E-03	-33.699	-28.203	-0.016537	-5.4878	-7.7159E-03	-0.065735	6.1520	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	19.651	7.3120	0.0000	10.968	45.700	0.0000	0.0000
2	-0.010525	-6.7171E-03	-33.835	-28.227	-0.016547	-5.4473	-7.5773E-03	-0.064823	5.0314	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
3	-0.010525	-6.6974E-03	-33.901	-28.145	-0.016504	-5.4421	-7.5834E-03	-0.064765	3.9108	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
4	-0.010525	-6.6771E-03	-33.970	-28.060	-0.016457	-5.4365	-7.5896E-03	-0.064706	2.7552	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
5	-0.010525	-6.6546E-03	-34.049	-27.966	-0.016397	-5.4299	-7.5957E-03	-0.064639	1.4992	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
6	-0.010525	-6.5742E-03	-34.076	-27.789	-0.016299	-5.4490	-7.7088E-03	-0.065193	0.2104	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
7	-0.010458	-6.7527E-03	-33.733	-28.601	-0.016991	-5.5300	-7.6579E-03	-0.066034	10.150	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	19.651	7.3120	0.0000	10.968	45.700	0.0000	0.0000
8	-0.010458	-6.8195E-03	-33.859	-28.637	-0.016949	-5.4903	-7.5205E-03	-0.065137	9.1220	9.4540E+06	2.6216E+07
x(FT)	0.0000	11.425	0.0000	19.194	19.651	7.3120	0.0000	10.968	45.700	0.0000	0.0000
9	-0.010458	-6.8007E-03	-33.928	-28.554	-0.016895	-5.4849	-7.5266E-03	-0.065083	8.0013	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	19.651	7.3120	0.0000	10.968	45.700	0.0000	0.0000
10	-0.010458	-6.7828E-03	-33.997	-28.473	-0.016847	-5.4796	-7.5326E-03	-0.065029	6.8807	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
11	-0.010458	-6.7650E-03	-34.066	-28.391	-0.016799	-5.4744	-7.5387E-03	-0.064976	5.7601	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	20.108	7.3120	0.0000	10.968	45.700	0.0000	0.0000
12	-0.010458	-6.6868E-03	-34.084	-28.226	-0.016743	-5.4955	-7.6518E-03	-0.065550	4.6395	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.0000	19.194	19.651	7.3120	0.0000	10.968	45.700	0.0000	0.0000
13	-0.010390	-5.3066E-03	-33.057	-26.670	-0.028778	-6.4149	-0.011248	-0.086710	13.737	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	1.3710	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
14	-0.010390	-5.3021E-03	-33.062	-26.623	-0.031323	-6.4061	-0.011232	-0.086505	12.806	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	1.3710	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
15	-0.010390	-5.2888E-03	-33.067	-26.564	-0.034881	-6.4023	-0.011241	-0.086431	11.812	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	1.3710	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
16	-0.010390	-5.2755E-03	-33.081	-26.506	-0.038472	-6.3983	-0.011250	-0.086357	10.805	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.9140	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
17	-0.010390	-5.2624E-03	-33.100	-26.448	-0.042062	-6.3944	-0.011259	-0.086284	9.7977	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.9140	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
18	-0.010390	-5.2484E-03	-33.121	-26.386	-0.045818	-6.3898	-0.011268	-0.086208	8.7301	9.4540E+06	2.6216E+07
x(FT)	0.0000	10.968	0.9140	17.823	0.0000	7.3120	0.0000	10.968	45.700	0.0000	0.0000
Min.	-0.010525	-6.8195E-03	-34.084	-28.637	-0.045818	-6.4149	-0.011268	-0.086710	0.2104	9.4540E+06	2.6216E+07
Pile N.	1	8	12	8	18	13	18	13	6	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR KIP-IN	MOMENT y-DIR KIP-IN	SHEAR y-DIR KIP	SHEAR z-DIR KIP	SOIL REACT y-DIR KIP/IN	SOIL REACT z-DIR KIP/IN	TOTAL STRESS KIP/IN**2	FLEX. RIG. z-DIR KIP-IN**2	FLEX. RIG. y-DIR KIP-IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	4.8029E-04	0.091646	0.8920	554.96	0.4275	0.5281	0.014930	0.069379	10.588	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
2	4.8717E-04	0.091511	0.8965	558.08	0.4228	0.5227	0.014364	0.068030	9.4917	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
3	4.8636E-04	0.091377	0.8952	560.15	0.4205	0.5211	0.014024	0.067982	8.3874	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
4	4.8541E-04	0.091243	0.8936	562.24	0.4181	0.5195	0.013695	0.067934	7.2483	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
5	4.8408E-04	0.091108	0.8914	564.42	0.4157	0.5176	0.013387	0.067886	6.0095	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
6	4.7720E-04	0.090974	0.8851	565.86	0.4148	0.5183	0.013229	0.068792	4.7320	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
7	4.9317E-04	0.091646	0.9107	551.77	0.4354	0.5399	0.014608	0.069274	14.561	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	15.995	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
8	5.0046E-04	0.091511	0.9157	554.78	0.4310	0.5344	0.014073	0.067927	13.557	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
9	4.9944E-04	0.091377	0.9140	556.84	0.4288	0.5328	0.013766	0.067878	12.452	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
10	4.9840E-04	0.091243	0.9123	558.90	0.4266	0.5312	0.013464	0.067829	11.348	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
11	4.9735E-04	0.091108	0.9105	560.96	0.4244	0.5296	0.013169	0.067780	10.244	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
12	4.9057E-04	0.090974	0.9045	562.27	0.4239	0.5311	0.013015	0.068692	9.1332	9.4540E+06	2.6216E+07
x(FT)	9.5970	0.0000	16.452	0.0000	6.3980	21.936	7.3120	3.1990	0.0000	0.0000	0.0000
13	3.6598E-04	0.091646	0.8377	536.13	0.4953	0.8106	0.018556	0.1022	18.023	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.8280	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000
14	3.6680E-04	0.091511	0.8388	537.37	0.4931	0.7928	0.018123	0.1019	17.103	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.3710	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000
15	3.6678E-04	0.091377	0.8390	538.58	0.4911	0.7873	0.017735	0.1019	16.118	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.3710	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000
16	3.6669E-04	0.091243	0.8390	539.79	0.4891	0.7821	0.017359	0.1018	15.120	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.3710	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000
17	3.6658E-04	0.091108	0.8390	541.00	0.4872	0.7769	0.016992	0.1017	14.123	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.3710	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000
18	3.6629E-04	0.090974	0.8387	542.22	0.4852	0.7728	0.016643	0.1016	13.064	9.4540E+06	2.6216E+07
x(FT)	9.1400	0.0000	15.081	1.3710	6.3980	0.0000	7.3120	3.1990	1.3710	0.0000	0.0000

Max.	5.0046E-04	0.091646	0.9157	565.86	0.4953	0.8106	0.018556	0.1022	18.023	9.4540E+06	2.6216E+07
Pile N.	8	1	8	6	13	13	13	13	13	1	1

***** SUMMARY FOR LOAD CASES AND COMBINATIONS *****

***** LOAD CASES RESULTS *****

LOAD CASE : 1

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONCENTRATED LOAD AT ORIGIN *

LOAD X,KIP	LOAD Y,KIP	LOAD Z,KIP	MOM X,KIP-IN	MOM Y,KIP-IN	MOM Z,KIP-IN
3439.97	0.00000	0.00000	0.00000	55085.6	1.08921E+05

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

DISP X,IN	DISP Y,IN	DISP Z,IN	ROT X,RAD	ROT Y,RAD	ROT Z,RAD
0.11842	-0.0229626	0.18260	-1.86592E-06	1.90195E-03	2.60521E-04

* TABLE M * COMPUTATION ON INDIVIDUAL PILE

* PILE TOP DISPLACEMENTS, GLOBAL *

	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	6	1	6	1	1	1
MAXIMUM	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	13	13	1	1	1	1

* PILE TOP REACTIONS, GLOBAL *

	FOR. X,KIP	FOR. Y,KIP	FOR. Z,KIP	MOM X,KIP-IN	MOM Y,KIP-IN	MOM Z,KIP-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	5.4916	-0.078850	-0.5946	-2.9514E-04	520.23	32.036
Pile N.	6	18	11	1	13	13
MAXIMUM	358.52	0.051452	1.1021	-2.9514E-04	565.86	34.084
Pile N.	13	8	13	1	6	12

* PILE TOP DISPLACEMENTS, LOCAL *

	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
	***********	***********	***********	***********	***********	***********
MINIMUM	3.0580E-03	-0.010525	0.090974	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	6	1	6	1	1	1
MAXIMUM	0.2338	-0.010390	0.091646	-1.8659E-06	1.9019E-03	2.6052E-04
Pile N.	13	13	1	1	1	1

* PILE TOP REACTIONS, LOCAL *

	AXIAL,KIP	LAT. y,KIP	LAT. z,KIP	MOM x,KIP-IN	MOM y,KIP-IN	MOM z,KIP-IN
	***********	***********	***********	***********	***********	***********
MINIMUM	5.4916	-0.078850	-0.5946	-2.9514E-04	520.23	32.036
Pile N.	6	18	11	1	13	13
MAXIMUM	358.52	0.051452	1.1021	-2.9514E-04	565.86	34.084
Pile N.	13	8	13	1	6	12

* EFFECTS FOR LATERALLY LOADED PILE *

PILE	DISPL. y-DIR IN	DISPL. z-DIR IN	MOMENT z-DIR KIP-IN	MOMENT y-DIR KIP-IN	SHEAR y-DIR KIP	SHEAR z-DIR KIP	SOIL REACT y-DIR KIP/IN	SOIL REACT z-DIR KIP/IN	TOTAL STRESS KIP/IN**2
******	**********	**********	**********	**********	**********	**********	**********	**********	**********
Min.	-0.010525	-6.8195E-03	-34.084	-28.637	-0.045818	-6.4149	-0.011268	-0.086710	0.2104
Pile N.	1	8	12	8	18	13	18	13	6
Max.	5.0046E-04	0.091646	0.9157	565.86	0.4953	0.8106	0.018556	0.1022	18.023
Pile N.	8	1	8	6	13	13	13	13	13

Axial Pile Resistance

Objective

Evaluate the axial geotechnical resistance of the abutment and pier piles for the Johnson Road Bridge in Falmouth, ME. Evalua; ons were conducted to assess a suitable driving system to install a pile to the required geotechnical nominal resistance of 829 kips for the HP 14x117 abutment piles and 552 kips for HP 14x89 piles at the pier.

Methodology

Evaluate proposed pile sec; on for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Sta; c Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Associa; on of State Highway and Transporta; on Officials, AASHTO LRFD Bridge Design Specifica; ons: Customary U.S. Units, 9th edi; on. (AASHTO LRFD)

Soil Properties

Consider Johnson Road Bridge Interpre; ve Subsurface Profile (see Figure 3), subsurface layering and proper; es rela; ve to pile design are presented in the Apile outputs aE ached.

Structural Properties

ASTM A572, Gr. 50 piles

Yield Strength of Steel $F_y := 50ksi$

Young's Modulus of Steel $E_s := 30000 \cdot ksi$

Abutments: HP14x117

Radius of gyra; on (weak axis) $r_{y.1} := 3.59in$

Area of sec; on $A_{s.1} := 34.4in^2$

Pier: HP14x89

Radius of gyra; on (weak axis) $r_{y.2} := 3.53in$

Area of sec; on $A_{s.2} := 26.1in^2$

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.5.1-1

Determine normalized column slenderness factor λ

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi}\right)^2 \cdot \frac{F_y}{E}$$ AASHTO Eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the pile is fully embedded, AASHTO 10.7.3.13.1.

Giving:

$P_{n.1} := 0.66^{\lambda} \cdot F_y \cdot A_{s.1}$ $P_{n.1} = 1720 \cdot kip$ **Abutments: HP14x117**

$P_{n.2} := 0.66^{\lambda} \cdot F_y \cdot A_{s.2}$ $P_{n.2} = 1305 \cdot kip$ **Pier: HP14x89**

2. Factored Structural Compressive Resistance - Strength Limit State:

Factor for piles in compression under hard driving condi; ons:

From Ar; cle 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State per AASHTO Eq. 6.9.2.1-1:

$P_{r.1} := \phi_c \cdot P_{n.1} = 860 \cdot kip$ **Abutments: HP14x117**

$P_{r.2} := \phi_c \cdot P_{n.2} = 652.5 \cdot kip$ **Pier: HP14x89**

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Ar; cle 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State per AASHTO Eq. 6.9.2.1-1 :

$P_{r.1} := \phi \cdot P_{n.1} = 1720 \cdot kip$ **Abutments: HP14x117**

$P_{r.2} := \phi \cdot P_{n.2} = 1305 \cdot kip$ **Pier: HP14x89**

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Ar; cle 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state or poten; al for driving damage to occur during hard driving.

Abutments: Required nominal resistance of 829 kips for the pile configura; on based on a maximum factored pile loads of 539 kips (max factored 410 kips and 129 kips downdrag) and a 0.65 resistance factor.

Pier: Required nominal resistance of 552 kips for the pile configura; on based on a maximum factored pile loads of 358 kips and a 0.65 resistance factor.

The es; mated % skin fric; on resistance is 10-20% for the piles at the required nominal pile resistance, based on the es; mated fric; on resistance. A-pile is higher, but we an; cipate driving resistance in the Marine clay may be lower than the calculated sta; c resistance.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8.1

$f_y := 50ksi$ yield Strength of steel

$\phi_{da} := 1.0$ AASHTO Table 10.5.5.2.3-1 Refers to Ar; cle 6.5.4.2, Pg. 6-28

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ Driving Stress in pile cannot exceed 45 ksi

Abutment 1 Piles - Drive pile plumb through 115 feet of soil to rock with toe quake representa; ve of; p resistance in soH soil (0.1 in) and no plug. Model pile length as 120 feet (5 foot s; ckup at end of drive).

Drive Abutment 1 piles with a Delmag D46-32 open-ended diesel hammer with a maximum rated energy of 107,315 H-lbs (fuel sel ng 1, maximum).

Abutment 2 Piles - Drive pile plumb through 30 feet of soil to rock with toe quake representa; ve of; p resistance on rock (0.04 in) and no plug. Model pile length as 35 feet (5 foot s; ckup at end of drive).

Drive abutment piles with a Delmag D25-32 open-ended diesel hammer with a maximum rated energy of 58,245 H-lbs (fuel sel ng 4, 3 below maximum)

Pier Piles - Drive pile plumb through 48 feet of soil to rock with toe quake representa; ve of; p resistance on rock (0.04 in) and no plug. Model pile length as 53 feet (5 foot s; ckup at end of drive).

Drive pier piles with a Delmag D25-32 open-ended diesel hammer with a rated energy of 58,245 H-lbs (fuel sel ng 4, three below max).

GRLWEAP Output is aE ached for the piles.

$R_{ndr1} := 829kip$

Required nominal geotechnical resistance:
AB1 :Toe quake 0.1 in (fuel sel ng 1) - pile driving stress=39 ksi, final penetra; on resistance=7 bpi.
AB2 :Toe quake 0.04 in (fuel sel ng 4) - pile driving stress=40 ksi, final penetra; on resistance=13 bpi.

$R_{ndr2} := 552kip$

Required nominal geotechnical resistance:
Toe quake 0.04inch (fuel sel ng 4) - pile driving stress=38 ksi, final penetra; on resistance=8 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria $\qquad \phi_{dyn} := 0.65$ $\qquad$ AASHTO Table 10.5.5.2.3-1

$$R_{ndr1\_factored} := R_{ndr1} \cdot \phi_{dyn}$$

$$R_{ndr1\_factored} = 539 \cdot kip$$

129 kips downdrag and 410 kips Axial load

$$R_{ndr2\_factored} := R_{ndr2} \cdot \phi_{dyn}$$

$$R_{ndr2\_factored} = 359 \cdot kip$$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\qquad \phi_{serv\_ext} := 1$

From Ar; cle 10.5.5.1 and 10.5.5.3

Pier 1:

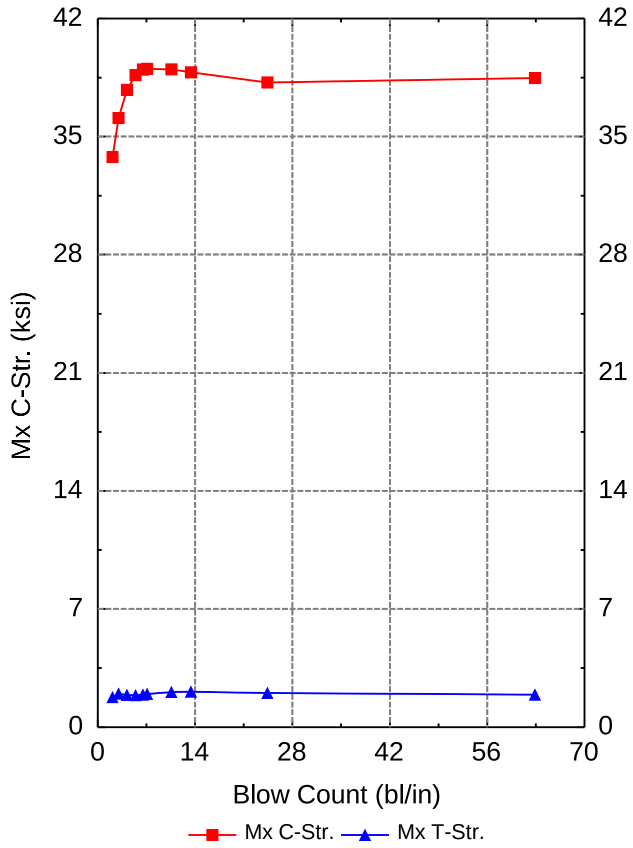
$$R_{ndr1\_serv\_ext} := R_{ndr1} \cdot \phi_{serv\_ext}$$

$$R_{ndr1\_serv\_ext} = 829 \cdot kip$$

$$R_{ndr2\_serv\_ext} := R_{ndr2} \cdot \phi_{serv\_ext}$$

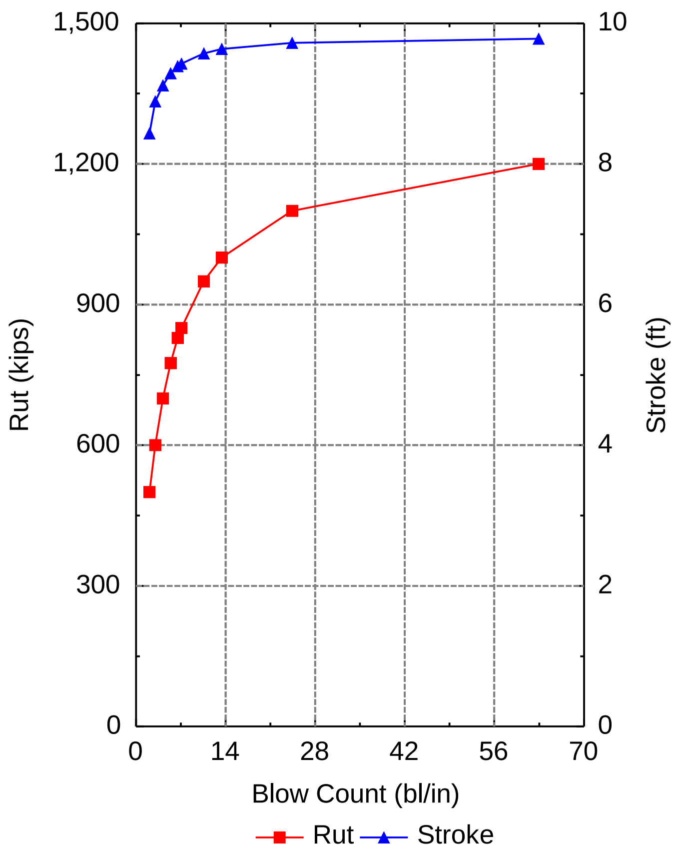
$$R_{ndr2\_serv\_ext} = 552 \cdot kip$$

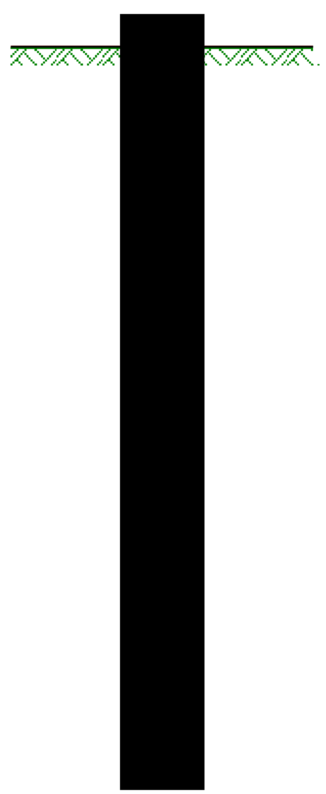
Since the driving stresses do not exceed the limi; ng driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetra; on resistance for both abutment and pier piles are within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer systems are judged acceptable to install the piles to the required nominal resistances. It should be noted that a smaller hammer could be used to acheive nominal resistance at the pier, but GZA's evalua; ons were conducted to limit two hammers for the project.



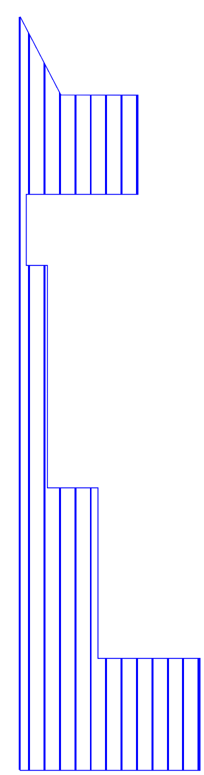
DELMAG D 46-32

Ram Weight	10.14	kips
Efficiency	0.800	
Pressure	1570.0 (100%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.200	s/ft
Toe Damping	0.150	s/ft
Pile Length	120.000	ft
Pile Penetration	115.000	ft
Pile Top Area	34.400	in^2
RSA	No	





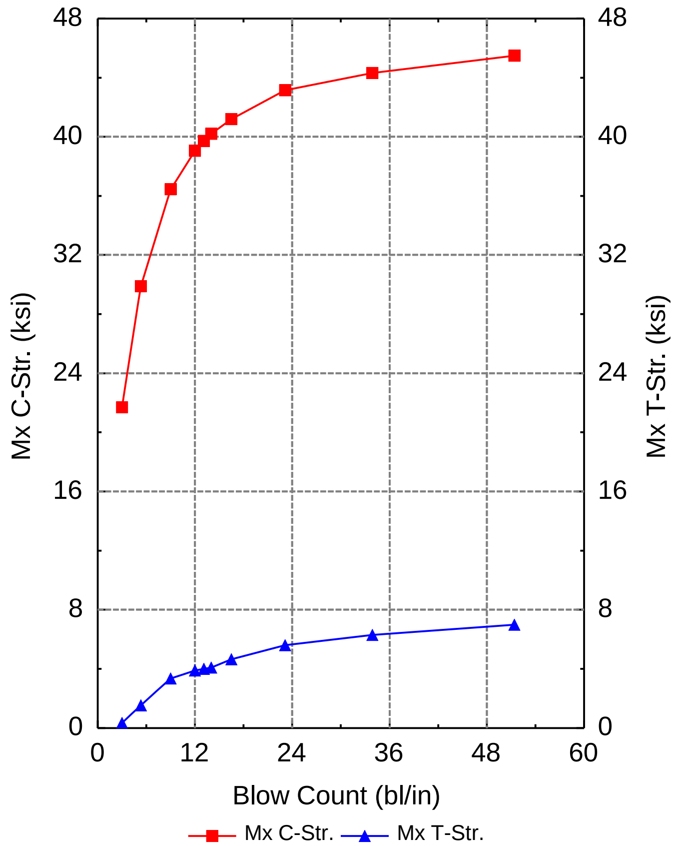
Pile Model



Shaft=20%
(Prop.)

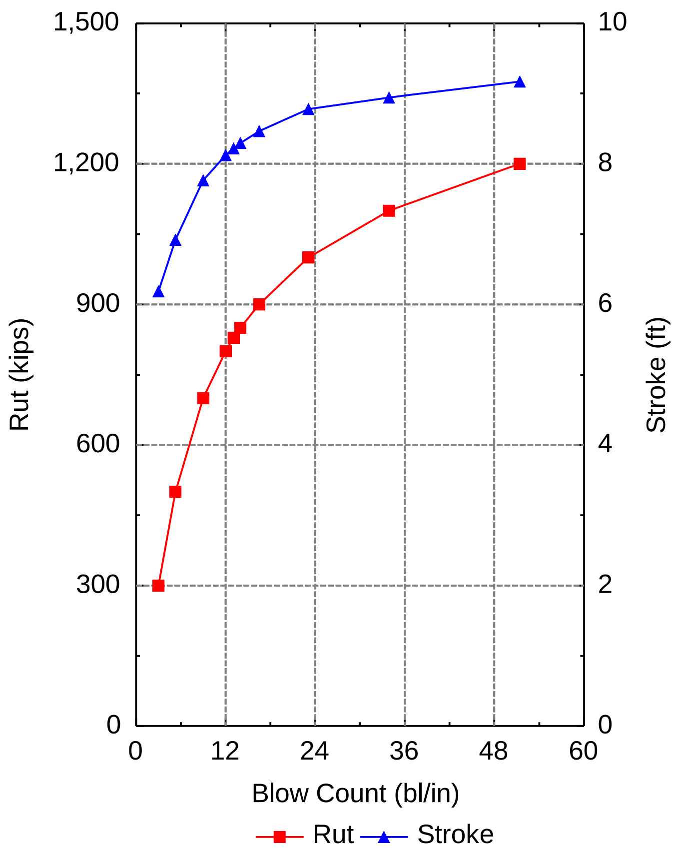
Bearing Graph Summary — DELMAG D 46-32

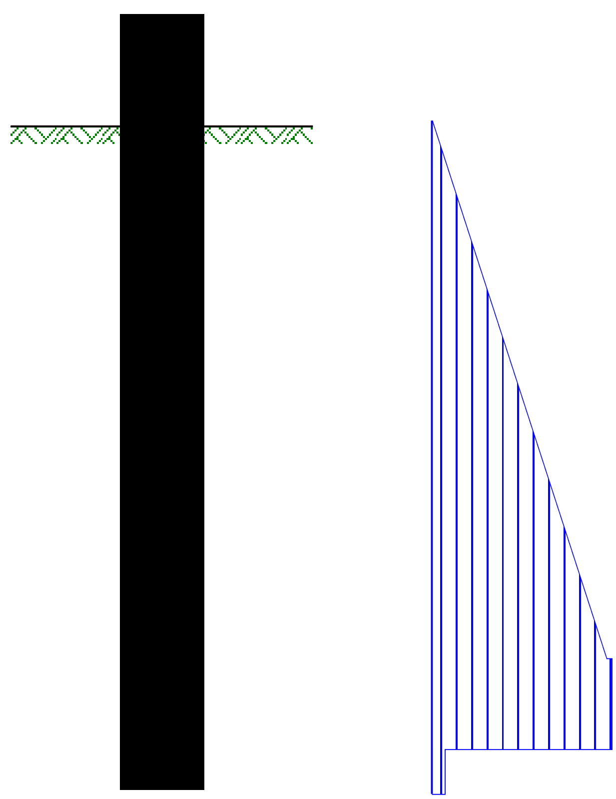
Rut	Mx C-Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer
kips	ksi	ksi	bl/in	ft	kip-ft	-
500.0	33.79	1.77	2.1	8.43	61.36	D 46-32
600.0	36.10	1.98	2.9	8.89	63.81	D 46-32
700.0	37.77	1.90	4.2	9.11	64.78	D 46-32
775.0	38.64	1.89	5.4	9.29	65.51	D 46-32
829.0	38.96	1.94	6.5	9.39	66.02	D 46-32
850.0	39.02	1.97	7.1	9.42	66.04	D 46-32
950.0	38.99	2.08	10.6	9.57	66.51	D 46-32
1000.0	38.81	2.10	13.4	9.64	66.71	D 46-32
1100.0	38.21	2.02	24.4	9.72	66.76	D 46-32
1200.0	38.46	1.93	62.9	9.78	66.63	D 46-32



DELMAG D 25-32

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1095.0 (73%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.152	s/ft
Pile Length	35.000	ft
Pile Penetration	30.000	ft
Pile Top Area	34.400	in²
RSA	No	



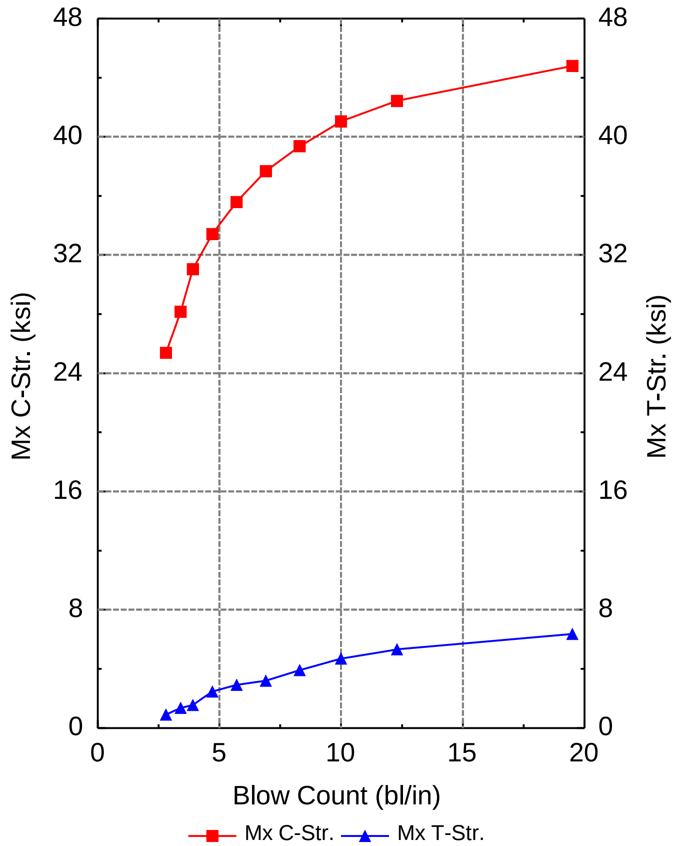


Pile Model

Shaft=10%
(Prop.)

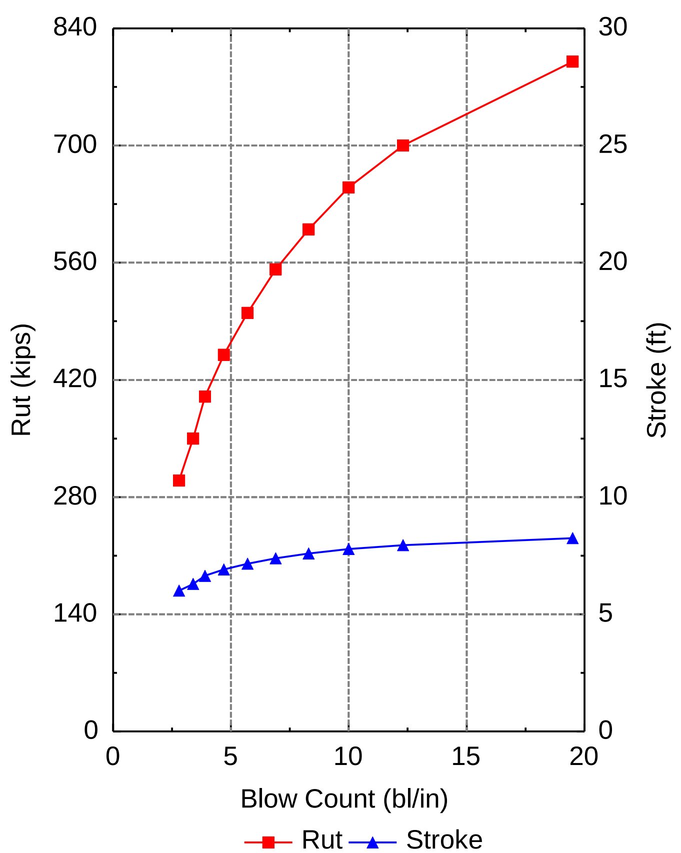
Bearing Graph Summary — DELMAG D 25-32

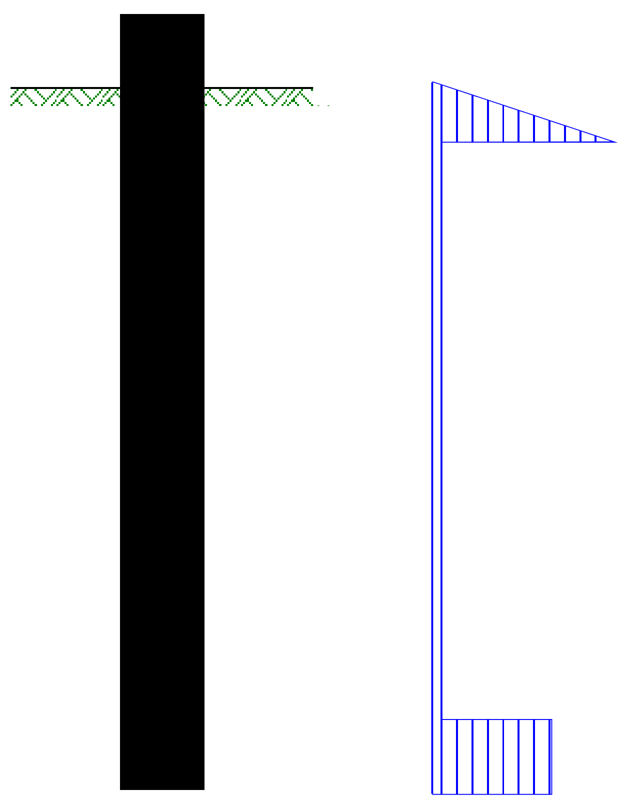
Rut	Mx C-Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer
kips	ksi	ksi	bl/in	ft	kip-ft	-
300.0	21.70	0.34	3.0	6.18	17.72	D 25-32
500.0	29.89	1.54	5.3	6.92	18.89	D 25-32
700.0	36.45	3.35	9.0	7.77	21.32	D 25-32
800.0	39.05	3.90	12.0	8.12	22.60	D 25-32
829.0	39.72	4.01	13.1	8.22	22.94	D 25-32
850.0	40.19	4.09	14.0	8.29	23.20	D 25-32
900.0	41.20	4.65	16.5	8.46	23.78	D 25-32
1000.0	43.14	5.60	23.1	8.78	24.89	D 25-32
1100.0	44.32	6.29	33.9	8.95	25.47	D 25-32
1200.0	45.49	7.00	51.4	9.17	26.27	D 25-32



DELMAG D 25-32

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1095.0 (73%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	53.000	ft
Pile Penetration	48.000	ft
Pile Top Area	26.100	in²
RSA	No	





Bearing Graph Summary — DELMAG D 25-32

Rut	Mx C-Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer
kips	ksi	ksi	bl/in	ft	kip-ft	-
300.0	25.38	0.91	2.8	6.01	18.57	D 25-32
350.0	28.15	1.35	3.4	6.29	19.35	D 25-32
400.0	31.04	1.54	3.9	6.65	20.43	D 25-32
450.0	33.41	2.46	4.7	6.91	21.12	D 25-32
500.0	35.59	2.91	5.7	7.15	21.91	D 25-32
552.0	37.67	3.19	6.9	7.38	22.78	D 25-32
600.0	39.36	3.92	8.3	7.59	23.60	D 25-32
650.0	41.03	4.70	10.0	7.78	24.33	D 25-32
700.0	42.44	5.31	12.3	7.95	24.96	D 25-32
800.0	44.79	6.36	19.5	8.25	26.02	D 25-32

```
     ==========================================================================

                      APILE for Windows, Version 2018.8.5

                       Serial Number :  653550831

                       A Program for Analyzing the Axial Capacity
                       and Short-term Settlement of Driven Piles
                                 under Axial Loading.
                     (c) Copyright ENSOFT, Inc., 1987-2015
                             All Rights Reserved

     ==========================================================================


     This program is licensed to :

     GZA GeoEnvironmental, Inc.
     Portland, OR

     Path to file locations      : P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd -
Falmouth\Work\Calcs\APile\
     Name of input data file     : 26024 Johnson Road Apile_AB1.ap8d
     Name of output file         : 26024 Johnson Road Apile_AB1.ap8o
     Name of plot output file    : 26024 Johnson Road Apile_AB1.ap8p

     --------------------------------------------------------------------------
                          Time and Date of Analysis
     --------------------------------------------------------------------------

               Date:  November 19, 2021     Time:  10:58:31



1
          ********************
          * INPUT INFORMATION *
          ********************



          Johnson Road Bridge 14x89

          DESIGNER   : E. Tome

          JOB NUMBER : 09.0026024.00


          METHOD FOR UNIT LOAD TRANSFERS :

          - USACE (U.S. Army Corps of Engineers)
            Unfactored Unit Side Friction and Unit Side Resistance are used.



          COMPUTATION METHOD(S) FOR PILE CAPACITY :

          - USACE (U.S. Army Corps of Engineers)
```

```
  # Critical Depth Method for Sand:
    10 to 20 Pile Diameter based on the Density
  Use Long Pile Option
- API RP 2A (American Petroleum Institute)
```

TYPE OF LOADING :
- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

```
- MODULUS OF ELASTICITY      = 0.290E+08 PSI
- CROSS SECTION AREA         =     26.10 IN2
```

NONCIRCULAR PILE PROPERTIES :

```
- TOTAL PILE LENGTH, TL      =    120.00 FT.
- BATTER ANGLE               =      0.00 DEG
- PILE STICKUP LENGTH, PSL   =      5.00 FT.
- ZERO FRICTION LENGTH, ZFL  =      0.00 FT.
- PERIMETER OF PILE          =     57.00 IN.
- TIP AREA OF PILE           =     26.10 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION        =      1.00 FT.
```

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.80	130.00	40.00	40.00
12.00	SAND	0.80	130.00	40.00	40.00
12.00	SAND	0.80	130.00	36.00	40.00
26.20	SAND	0.80	130.00	36.00	40.00
26.20	SAND	0.80	58.00	30.00	20.00
30.00	SAND	0.80	58.00	30.00	20.00
30.00	CLAY	0.00	53.00	0.00	0.00
38.00	CLAY	0.00	53.00	0.00	0.00
38.00	CLAY	0.00	53.00	0.00	0.00
72.00	CLAY	0.00	53.00	0.00	0.00
72.00	SAND	0.80	73.00	32.00	8.00
98.00	SAND	0.80	73.00	32.00	8.00
98.00	SAND	0.80	73.00	38.00	50.00
120.40	SAND	0.80	3.00	38.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.60	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.60	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.45	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.45	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
12.00	1.000	1.000
12.00	1.000	1.000
26.20	1.000	1.000
26.20	1.000	1.000
30.00	1.000	1.000
30.00	1.000	1.000
38.00	1.000	1.000
38.00	1.000	1.000
72.00	1.000	1.000
72.00	1.000	1.000
98.00	1.000	1.000
98.00	1.000	1.000
120.40	1.000	1.000

1

****
* COMPUTATION RESULT *
****

****
* ARMY CORPS METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.2	1.2
1.00	0.3	2.2	2.5
2.00	1.1	3.9	5.0
3.00	2.4	6.2	8.7
4.00	4.3	8.3	12.6
5.00	6.8	10.4	17.1
6.00	9.7	12.4	22.2
7.00	13.2	14.5	27.8
8.00	17.3	16.6	33.9
9.00	21.9	18.7	40.6
10.00	27.0	20.2	47.2
11.00	32.7	20.1	52.8
12.00	38.9	19.9	58.8
13.00	45.2	19.5	64.8
14.00	51.6	19.0	70.6
15.00	58.5	19.8	78.3
16.00	65.8	21.1	86.9
17.00	73.6	22.4	96.0
18.00	81.9	23.8	105.6
19.00	90.6	25.1	115.7
20.00	99.9	26.4	126.3
21.00	109.6	27.7	137.3
22.00	119.7	29.0	148.8
23.00	130.4	30.3	160.7
24.00	141.5	31.4	172.9
25.00	153.1	31.1	184.2
26.00	165.1	27.3	192.5
27.00	177.7	23.3	200.9
28.00	186.9	18.2	205.2
29.00	192.7	11.2	203.9
30.00	198.4	7.3	205.7
31.00	202.7	4.5	207.2
32.00	205.4	1.7	207.1
33.00	208.1	1.0	209.1
34.00	210.8	1.0	211.8
35.00	213.5	1.0	214.5
36.00	216.2	1.0	217.2
37.00	218.9	0.9	219.8
38.00	221.6	0.9	222.5
39.00	224.1	0.8	224.9
40.00	226.2	0.7	226.9
41.00	228.3	0.7	229.1
42.00	230.5	0.7	231.2
43.00	232.6	0.7	233.3
44.00	234.7	0.7	235.5
45.00	236.9	0.7	237.6
46.00	239.0	0.7	239.7
47.00	241.2	0.7	241.9
48.00	243.3	0.7	244.0
49.00	245.4	0.7	246.2
50.00	247.6	0.7	248.3
51.00	249.7	0.7	250.4
52.00	251.8	0.7	252.6
53.00	254.0	0.7	254.7
54.00	256.1	0.7	256.8

55.00	258.3	0.7	259.0
56.00	260.4	0.7	261.1
57.00	262.5	0.7	263.3
58.00	264.7	0.7	265.4
59.00	266.8	0.7	267.5
60.00	268.9	0.7	269.7
61.00	271.1	0.7	271.8
62.00	273.2	0.7	273.9
63.00	275.4	0.7	276.1
64.00	277.5	0.7	278.2
65.00	279.6	0.7	280.4
66.00	281.8	0.7	282.5
67.00	283.9	0.7	284.6
68.00	286.0	0.7	286.8
69.00	288.2	0.7	288.9
70.00	290.3	1.7	292.0
71.00	292.5	5.5	297.9
72.00	294.6	9.2	303.8
73.00	298.8	12.9	311.7
74.00	305.0	16.7	321.6
75.00	311.2	17.7	328.8
76.00	317.4	17.7	335.0
77.00	323.6	17.7	341.3
78.00	329.8	17.7	347.5
79.00	336.0	17.7	353.7
80.00	342.2	17.7	359.9
81.00	348.4	17.7	366.1
82.00	354.6	17.7	372.3
83.00	360.8	17.7	378.5
84.00	367.0	17.7	384.7
85.00	373.2	17.7	390.9
86.00	379.4	17.7	397.1
87.00	385.6	17.7	403.3
88.00	391.8	17.7	409.5
89.00	398.0	17.7	415.7
90.00	404.3	17.7	421.9
91.00	410.5	17.7	428.1
92.00	416.7	17.7	434.3
93.00	422.9	17.7	440.5
94.00	429.1	17.7	446.7
95.00	435.3	17.7	452.9
96.00	441.5	19.3	460.8
97.00	447.7	25.5	473.2
98.00	453.9	31.6	485.5
99.00	464.2	37.8	502.0
100.00	478.6	43.9	522.5
101.00	493.0	45.6	538.6
102.00	507.4	45.6	553.0
103.00	521.8	45.6	567.4
104.00	536.2	45.6	581.7
105.00	550.6	45.6	596.1
106.00	565.0	45.6	610.5
107.00	579.4	45.6	624.9
108.00	593.7	45.6	639.3
109.00	608.1	45.6	653.7
110.00	622.5	45.6	668.1
111.00	636.9	45.6	682.5
112.00	651.3	45.6	696.9

113.00	665.7	45.6	711.3
114.00	680.1	45.6	725.7
115.00	694.5	45.6	740.1

****
* API RP-2A (2010) *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.5	0.5
1.00	0.2	1.0	1.2
2.00	0.7	1.8	2.5
3.00	1.6	2.8	4.4
4.00	2.8	3.8	6.5
5.00	4.3	4.7	9.0
6.00	6.2	5.7	11.9
7.00	8.5	6.6	15.1
8.00	11.1	7.5	18.6
9.00	14.0	8.5	22.5
10.00	17.3	9.4	26.7
11.00	20.9	10.4	31.3
12.00	24.9	11.3	36.2
13.00	28.9	12.3	41.2
14.00	32.9	13.2	46.1
15.00	37.2	14.1	51.4
16.00	41.8	15.1	56.9
17.00	46.7	16.0	62.7
18.00	51.9	17.0	68.9
19.00	57.4	17.9	75.3
20.00	63.2	18.9	82.0
21.00	69.3	19.8	89.1
22.00	75.7	20.7	96.4
23.00	82.3	21.7	104.0
24.00	89.3	22.6	111.9
25.00	96.6	22.8	119.4
26.00	104.2	20.8	125.0
27.00	112.0	18.7	130.7
28.00	119.2	15.6	134.8
29.00	125.6	10.4	136.0
30.00	132.0	7.0	139.1
31.00	136.7	4.4	141.1
32.00	139.6	1.7	141.3
33.00	142.4	1.0	143.4
34.00	145.3	1.0	146.3
35.00	148.1	1.0	149.1
36.00	151.0	1.0	151.9
37.00	153.8	0.9	154.7
38.00	156.7	0.9	157.5
39.00	159.2	0.8	160.0
40.00	161.3	0.7	162.1
41.00	163.4	0.7	164.2
42.00	165.6	0.7	166.3
43.00	167.7	0.7	168.5
44.00	169.9	0.7	170.6

45.00	172.0	0.7	172.7
46.00	174.1	0.7	174.9
47.00	176.3	0.7	177.0
48.00	178.4	0.7	179.1
49.00	180.5	0.7	181.3
50.00	182.7	0.7	183.4
51.00	184.8	0.7	185.6
52.00	187.0	0.7	187.7
53.00	189.1	0.7	189.8
54.00	191.2	0.7	192.0
55.00	193.4	0.7	194.1
56.00	195.5	0.7	196.2
57.00	197.6	0.7	198.4
58.00	199.8	0.7	200.5
59.00	201.9	0.7	202.7
60.00	204.1	0.7	204.8
61.00	206.2	0.7	206.9
62.00	208.3	0.7	209.1
63.00	210.5	0.7	211.2
64.00	212.6	0.7	213.3
65.00	214.7	0.7	215.5
66.00	216.9	0.7	217.6
67.00	219.0	0.7	219.8
68.00	221.2	0.7	221.9
69.00	223.3	0.7	224.0
70.00	225.4	1.2	226.6
71.00	227.6	2.9	230.5
72.00	229.7	4.7	234.4
73.00	235.1	6.5	241.6
74.00	243.7	8.3	252.1
75.00	252.4	8.9	261.3
76.00	261.0	9.0	270.0
77.00	269.7	9.1	278.8
78.00	278.3	9.2	287.5
79.00	287.0	9.3	296.3
80.00	295.6	9.4	305.0
81.00	304.3	9.5	313.8
82.00	312.9	9.6	322.5
83.00	321.5	9.7	331.3
84.00	330.2	9.8	340.0
85.00	338.8	9.9	348.8
86.00	347.5	10.1	357.5
87.00	356.1	10.2	366.3
88.00	364.8	10.3	375.0
89.00	373.4	10.4	383.8
90.00	382.1	10.5	392.5
91.00	390.7	10.6	401.3
92.00	399.4	10.7	410.0
93.00	408.0	10.8	418.8
94.00	416.6	10.9	427.5
95.00	425.3	11.0	436.3
96.00	433.9	12.9	446.8
97.00	442.6	19.7	462.3
98.00	451.2	26.4	477.7
99.00	460.9	33.2	494.0
100.00	471.5	39.9	511.4
101.00	482.1	41.7	523.8
102.00	492.8	41.7	534.5

103.00	503.4	41.7	545.1
104.00	514.1	41.7	555.8
105.00	524.7	41.7	566.4
106.00	535.3	41.6	577.0
107.00	546.0	41.1	587.1
108.00	556.6	40.0	596.6
109.00	567.3	38.1	605.3
110.00	577.9	35.4	613.3
111.00	588.5	32.3	620.8
112.00	599.2	28.9	628.1
113.00	609.8	25.6	635.4
114.00	620.5	22.2	642.6
115.00	631.1	18.8	649.9

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

```
*************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
*************************************************
```

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
2	10	0.6025E+01		
			0.0000E+00	0.0000E+00
			0.9900E+00	0.1000E-01
			0.1639E+01	0.2000E-01
			0.2439E+01	0.4000E-01
			0.2912E+01	0.6000E-01
			0.3225E+01	0.8000E-01
			0.3614E+01	0.1200E+00
			0.3845E+01	0.1600E+00
			0.4424E+01	0.5000E+00
			0.4743E+01	0.1000E+02
3	10	0.1196E+02		
			0.0000E+00	0.0000E+00
			0.1104E+01	0.1000E-01
			0.1977E+01	0.2000E-01
			0.3270E+01	0.4000E-01

			0.4181E+01	0.6000E-01
			0.4858E+01	0.8000E-01
			0.5797E+01	0.1200E+00
			0.6417E+01	0.1600E+00
			0.8208E+01	0.5000E+00
			0.9378E+01	0.1000E+02
4	10	0.1200E+02		
			0.0000E+00	0.0000E+00
			0.9573E+00	0.1000E-01
			0.1739E+01	0.2000E-01
			0.2939E+01	0.4000E-01
			0.3817E+01	0.6000E-01
			0.4487E+01	0.8000E-01
			0.5443E+01	0.1200E+00
			0.6092E+01	0.1600E+00
			0.8049E+01	0.5000E+00
			0.9398E+01	0.1000E+02
5	10	0.1913E+02		
			0.0000E+00	0.0000E+00
			0.9855E+00	0.1000E-01
			0.1834E+01	0.2000E-01
			0.3222E+01	0.4000E-01
			0.4308E+01	0.6000E-01
			0.5181E+01	0.8000E-01
			0.6499E+01	0.1200E+00
			0.7446E+01	0.1600E+00
			0.1059E+02	0.5000E+00
			0.1306E+02	0.1000E+02
6	10	0.2616E+02		
			0.0000E+00	0.0000E+00
			0.1006E+01	0.1000E-01
			0.1905E+01	0.2000E-01
			0.3448E+01	0.4000E-01
			0.4721E+01	0.6000E-01
			0.5791E+01	0.8000E-01
			0.7488E+01	0.1200E+00
			0.8774E+01	0.1600E+00
			0.1350E+02	0.5000E+00
			0.1779E+02	0.1000E+02
7	10	0.2620E+02		
			0.0000E+00	0.0000E+00
			0.5825E+00	0.1000E-01
			0.1129E+01	0.2000E-01
			0.2125E+01	0.4000E-01
			0.3011E+01	0.6000E-01
			0.3804E+01	0.8000E-01
			0.5164E+01	0.1200E+00
			0.6288E+01	0.1600E+00
			0.1131E+02	0.5000E+00
			0.1759E+02	0.1000E+02
8	10	0.2813E+02		
			0.0000E+00	0.0000E+00
			0.5617E+00	0.1000E-01
			0.1053E+01	0.2000E-01
			0.1872E+01	0.4000E-01
			0.2527E+01	0.6000E-01
			0.3062E+01	0.8000E-01
			0.3886E+01	0.1200E+00

			0.4490E+01	0.1600E+00
			0.6576E+01	0.5000E+00
			0.8298E+01	0.1000E+02
9	10	0.2996E+02		
			0.0000E+00	0.0000E+00
			0.5617E+00	0.1000E-01
			0.1053E+01	0.2000E-01
			0.1872E+01	0.4000E-01
			0.2527E+01	0.6000E-01
			0.3062E+01	0.8000E-01
			0.3886E+01	0.1200E+00
			0.4490E+01	0.1600E+00
			0.6576E+01	0.5000E+00
			0.8298E+01	0.1000E+02
10	10	0.3000E+02		
			0.0000E+00	0.0000E+00
			0.1515E+01	0.1000E-01
			0.3197E+01	0.2000E-01
			0.6647E+01	0.4000E-01
			0.8162E+01	0.6000E-01
			0.8414E+01	0.8000E-01
			0.8162E+01	0.1200E+00
			0.7825E+01	0.1600E+00
			0.7825E+01	0.5000E+00
			0.7825E+01	0.1000E+02
11	10	0.3403E+02		
			0.0000E+00	0.0000E+00
			0.7125E+00	0.1000E-01
			0.1504E+01	0.2000E-01
			0.3127E+01	0.4000E-01
			0.3840E+01	0.6000E-01
			0.3958E+01	0.8000E-01
			0.3840E+01	0.1200E+00
			0.3681E+01	0.1600E+00
			0.3681E+01	0.5000E+00
			0.3681E+01	0.1000E+02
12	10	0.3796E+02		
			0.0000E+00	0.0000E+00
			0.7125E+00	0.1000E-01
			0.1504E+01	0.2000E-01
			0.3127E+01	0.4000E-01
			0.3840E+01	0.6000E-01
			0.3958E+01	0.8000E-01
			0.3840E+01	0.1200E+00
			0.3681E+01	0.1600E+00
			0.3681E+01	0.5000E+00
			0.3681E+01	0.1000E+02
13	10	0.3800E+02		
			0.0000E+00	0.0000E+00
			0.7125E+00	0.1000E-01
			0.1504E+01	0.2000E-01
			0.3127E+01	0.4000E-01
			0.3840E+01	0.6000E-01
			0.3958E+01	0.8000E-01
			0.3840E+01	0.1200E+00
			0.3681E+01	0.1600E+00
			0.3681E+01	0.5000E+00
			0.3681E+01	0.1000E+02

14	10	0.5503E+02		
			0.0000E+00	0.0000E+00
			0.5625E+00	0.1000E-01
			0.1187E+01	0.2000E-01
			0.2469E+01	0.4000E-01
			0.3031E+01	0.6000E-01
			0.3125E+01	0.8000E-01
			0.3031E+01	0.1200E+00
			0.2906E+01	0.1600E+00
			0.2906E+01	0.5000E+00
			0.2906E+01	0.1000E+02
15	10	0.7196E+02		
			0.0000E+00	0.0000E+00
			0.5625E+00	0.1000E-01
			0.1187E+01	0.2000E-01
			0.2469E+01	0.4000E-01
			0.3031E+01	0.6000E-01
			0.3125E+01	0.8000E-01
			0.3031E+01	0.1200E+00
			0.2906E+01	0.1600E+00
			0.2906E+01	0.5000E+00
			0.2906E+01	0.1000E+02
16	10	0.7200E+02		
			0.0000E+00	0.0000E+00
			0.5682E+00	0.1000E-01
			0.9615E+00	0.2000E-01
			0.1471E+01	0.4000E-01
			0.1786E+01	0.6000E-01
			0.2000E+01	0.8000E-01
			0.2273E+01	0.1200E+00
			0.2439E+01	0.1600E+00
			0.2867E+01	0.5000E+00
			0.3111E+01	0.1000E+02
17	10	0.8503E+02		
			0.0000E+00	0.0000E+00
			0.6451E+00	0.1000E-01
			0.1204E+01	0.2000E-01
			0.2127E+01	0.4000E-01
			0.2855E+01	0.6000E-01
			0.3446E+01	0.8000E-01
			0.4344E+01	0.1200E+00
			0.4994E+01	0.1600E+00
			0.7193E+01	0.5000E+00
			0.8955E+01	0.1000E+02
18	10	0.9796E+02		
			0.0000E+00	0.0000E+00
			0.6451E+00	0.1000E-01
			0.1204E+01	0.2000E-01
			0.2127E+01	0.4000E-01
			0.2855E+01	0.6000E-01
			0.3446E+01	0.8000E-01
			0.4344E+01	0.1200E+00
			0.4994E+01	0.1600E+00
			0.7193E+01	0.5000E+00
			0.8955E+01	0.1000E+02
19	10	0.9800E+02		
			0.0000E+00	0.0000E+00
			0.1099E+01	0.1000E-01

```
                                                  0.1960E+01      0.2000E-01
                                                  0.3223E+01      0.4000E-01
                                                  0.4106E+01      0.6000E-01
                                                  0.4757E+01      0.8000E-01
                                                  0.5653E+01      0.1200E+00
                                                  0.6241E+01      0.1600E+00
                                                  0.7922E+01      0.5000E+00
                                                  0.9007E+01      0.1000E+02
   20          10      0.1092E+03
                                                  0.0000E+00      0.0000E+00
                                                  0.1180E+01      0.1000E-01
                                                  0.2235E+01      0.2000E-01
                                                  0.4040E+01      0.4000E-01
                                                  0.5529E+01      0.6000E-01
                                                  0.6779E+01      0.8000E-01
                                                  0.8758E+01      0.1200E+00
                                                  0.1025E+02      0.1600E+00
                                                  0.1574E+02      0.5000E+00
                                                  0.2070E+02      0.1000E+02
   21          10      0.1204E+03
                                                  0.0000E+00      0.0000E+00
                                                  0.1180E+01      0.1000E-01
                                                  0.2235E+01      0.2000E-01
                                                  0.4040E+01      0.4000E-01
                                                  0.5529E+01      0.6000E-01
                                                  0.6779E+01      0.8000E-01
                                                  0.8758E+01      0.1200E+00
                                                  0.1025E+02      0.1600E+00
                                                  0.1574E+02      0.5000E+00
                                                  0.2070E+02      0.1000E+02


TIP   LOAD        TIP MOVEMENT
    KIP               IN.

0.0000E+00         0.0000E+00
0.4785E+00         0.1000E-03
0.3384E+01         0.5000E-02
0.4785E+01         0.1000E-01
0.1070E+02         0.5000E-01
0.1513E+02         0.1000E+00
0.2140E+02         0.2000E+00
0.3384E+02         0.5000E+00
0.4785E+02         0.1000E+01
0.6767E+02         0.2000E+01


              LOAD VERSUS SETTLEMENT CURVE
              ****************************


 TOP  LOAD     TOP MOVEMENT       TIP  LOAD      TIP MOVEMENT
    KIP            IN.              KIP              IN.
0.5603E+01      0.3598E-02      0.4785E+00      0.1000E-03
0.2272E+02      0.1466E-01      0.1012E+01      0.1000E-02
```

0.8851E+02	0.6089E-01	0.3384E+01	0.5000E-02
0.1322E+03	0.9997E-01	0.4785E+01	0.1000E-01
0.2601E+03	0.2830E+00	0.1070E+02	0.5000E-01
0.3341E+03	0.4304E+00	0.1513E+02	0.1000E+00
0.5267E+03	0.1110E+01	0.3384E+02	0.5000E+00
0.5507E+03	0.1650E+01	0.4785E+02	0.1000E+01
0.5904E+03	0.2714E+01	0.6767E+02	0.2000E+01

Abutment 2

```
==========================================================================

                 APILE for Windows, Version 2018.8.5

                  Serial Number :  653550831

                  A Program for Analyzing the Axial Capacity
                  and Short-term Settlement of Driven Piles
                            under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2015
                      All Rights Reserved

==========================================================================


This program is licensed to :

GZA GeoEnvironmental, Inc.
Portland, OR

Path to file locations      : P:\09 Jobs\0026000s\09.0026024.00 - MEDOT - Johnson Rd -
Falmouth\Work\Calcs\APile\
Name of input data file     : 26024 Johnson Road Apile_AB2.ap8d
Name of output file         : 26024 Johnson Road Apile_AB2.ap8o
Name of plot output file    : 26024 Johnson Road Apile_AB2.ap8p

--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  November 19, 2021     Time:  11:10:47



1
     ********************
     * INPUT INFORMATION *
     ********************



     Johnson Road Bridge 14x89 AB2

     DESIGNER   : E. Tome

     JOB NUMBER : 09.0026024.00


     METHOD FOR UNIT LOAD TRANSFERS :

     - USACE (U.S. Army Corps of Engineers)
       Unfactored Unit Side Friction and Unit Side Resistance are used.



     COMPUTATION METHOD(S) FOR PILE CAPACITY :

     - USACE (U.S. Army Corps of Engineers)
```

```
  # Critical Depth Method for Sand:
    10 to 20 Pile Diameter based on the Density
  Use Long Pile Option
- API RP 2A (American Petroleum Institute)


TYPE OF LOADING :
- COMPRESSION


PILE TYPE :

H-Pile/Steel Pile


DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY     = 0.290E+08 PSI
- CROSS SECTION AREA        =     26.10 IN2


NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL     =     35.00 FT.
- BATTER ANGLE              =      0.00 DEG
- PILE STICKUP LENGTH, PSL  =      5.00 FT.
- ZERO FRICTION LENGTH, ZFL =      0.00 FT.
- PERIMETER OF PILE         =     57.00 IN.
- TIP AREA OF PILE          =     26.10 IN2
- INCREMENT OF PILE LENGTH
  USED IN COMPUTATION       =      1.00 FT.



SOIL INFORMATIONS :



                LATERAL    EFFECTIVE    FRICTION   BEARING
         SOIL   EARTH      UNIT         ANGLE      CAPACITY
 DEPTH   TYPE   PRESSURE   WEIGHT       DEGREES    FACTOR
  FT.                      LB/CF
   0.00  SAND     0.80       130.00       40.00     40.00
  23.50  SAND     0.80       130.00       40.00     40.00
  23.50  SAND     0.80        58.00       30.00     20.00
  28.00  SAND     0.80        58.00       30.00     20.00
  28.00  CLAY     0.00        53.00        0.00      0.00
  39.30  CLAY     0.00        53.00        0.00      0.00



MAXIMUM   MAXIMUM   UNDISTURB  REMOLDED
UNIT      UNIT      SHEAR      SHEAR      BLOW   UNIT SKIN  UNIT END
FRICTION  BEARING   STRENGTH   STRENGTH   COUNT  FRICTION   BEARING
  KSF       KSF        KSF       KSF                KSF       KSF
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
0.10E+08* 0.10E+08*      0.00      0.00   0.00       0.00      0.00
```

0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	1.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING
  WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT
  PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
23.50	1.000	1.000
23.50	1.000	1.000
28.00	1.000	1.000
28.00	1.000	1.000
39.30	1.000	1.000

1

****
* COMPUTATION RESULT *
****

****
* ARMY CORPS METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.2	1.2
1.00	0.3	2.2	2.5
2.00	1.1	3.9	5.0
3.00	2.4	6.2	8.7
4.00	4.3	8.3	12.6
5.00	6.8	10.4	17.1
6.00	9.7	12.4	22.2
7.00	13.2	14.5	27.8
8.00	17.3	16.6	33.9
9.00	21.9	18.7	40.6
10.00	27.0	20.7	47.8
11.00	32.7	22.8	55.5
12.00	38.9	24.9	63.8
13.00	45.7	27.0	72.6
14.00	53.0	29.0	82.0
15.00	60.8	31.1	91.9
16.00	69.2	33.2	102.4
17.00	78.1	35.2	113.3
18.00	87.6	37.3	124.9
19.00	97.6	39.4	137.0
20.00	108.1	41.5	149.6

21.00	119.2	43.5	162.7
22.00	130.8	43.1	173.9
23.00	143.0	35.7	178.7
24.00	155.7	27.8	183.5
25.00	165.0	19.4	184.5
26.00	170.8	10.2	181.0
27.00	176.5	6.4	182.9
28.00	182.3	4.9	187.2
29.00	186.9	3.5	190.4
30.00	190.5	2.0	192.5

****
* API RP-2A (2010) *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.5	0.5
1.00	0.2	1.0	1.2
2.00	0.7	1.8	2.5
3.00	1.6	2.8	4.4
4.00	2.8	3.8	6.5
5.00	4.3	4.7	9.0
6.00	6.2	5.7	11.9
7.00	8.5	6.6	15.1
8.00	11.1	7.5	18.6
9.00	14.0	8.5	22.5
10.00	17.3	9.4	26.7
11.00	20.9	10.4	31.3
12.00	24.9	11.3	36.2
13.00	29.2	12.3	41.5
14.00	33.9	13.2	47.1
15.00	38.9	14.1	53.1
16.00	44.3	15.1	59.4
17.00	50.0	16.0	66.0
18.00	56.0	17.0	73.0
19.00	62.4	17.9	80.3
20.00	69.2	18.9	88.0
21.00	76.3	19.8	96.1
22.00	83.7	20.1	103.8
23.00	91.5	18.4	109.9
24.00	99.6	16.5	116.2
25.00	106.6	14.6	121.1
26.00	112.3	11.8	124.0
27.00	118.1	9.0	127.0
28.00	123.9	6.8	130.7
29.00	129.1	4.5	133.6
30.00	133.5	2.3	135.8

NOTES:
- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

```
**************************************************
* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
**************************************************


T-Z CURVE   NO. OF   DEPTH TO CURVE   LOAD TRANSFER   PILE MOVEMENT
   NO.      POINTS        FT.              PSI              IN.

    1         10     0.0000E+00
                                     0.0000E+00      0.0000E+00
                                     0.0000E+00      0.1000E-01
                                     0.0000E+00      0.2000E-01
                                     0.0000E+00      0.4000E-01
                                     0.0000E+00      0.6000E-01
                                     0.0000E+00      0.8000E-01
                                     0.0000E+00      0.1200E+00
                                     0.0000E+00      0.1600E+00
                                     0.0000E+00      0.5000E+00
                                     0.0000E+00      0.1000E+02
    2         10     0.1178E+02
                                     0.0000E+00      0.0000E+00
                                     0.1102E+01      0.1000E-01
                                     0.1971E+01      0.2000E-01
                                     0.3252E+01      0.4000E-01
                                     0.4153E+01      0.6000E-01
                                     0.4820E+01      0.8000E-01
                                     0.5742E+01      0.1200E+00
                                     0.6350E+01      0.1600E+00
                                     0.8099E+01      0.5000E+00
                                     0.9235E+01      0.1000E+02
    3         10     0.2346E+02
                                     0.0000E+00      0.0000E+00
                                     0.1171E+01      0.1000E-01
                                     0.2203E+01      0.2000E-01
                                     0.3938E+01      0.4000E-01
                                     0.5340E+01      0.6000E-01
                                     0.6496E+01      0.8000E-01
                                     0.8291E+01      0.1200E+00
                                     0.9620E+01      0.1600E+00
                                     0.1430E+02      0.5000E+00
                                     0.1827E+02      0.1000E+02
    4         10     0.2350E+02
                                     0.0000E+00      0.0000E+00
                                     0.5830E+00      0.1000E-01
                                     0.1130E+01      0.2000E-01
                                     0.2131E+01      0.4000E-01
                                     0.3023E+01      0.6000E-01
                                     0.3823E+01      0.8000E-01
                                     0.5200E+01      0.1200E+00
                                     0.6341E+01      0.1600E+00
                                     0.1148E+02      0.5000E+00
                                     0.1801E+02      0.1000E+02
    5         10     0.2578E+02
                                     0.0000E+00      0.0000E+00
                                     0.5617E+00      0.1000E-01
```

```
                                               0.1053E+01      0.2000E-01
                                               0.1872E+01      0.4000E-01
                                               0.2527E+01      0.6000E-01
                                               0.3062E+01      0.8000E-01
                                               0.3886E+01      0.1200E+00
                                               0.4490E+01      0.1600E+00
                                               0.6576E+01      0.5000E+00
                                               0.8298E+01      0.1000E+02
    6          10      0.2796E+02
                                               0.0000E+00      0.0000E+00
                                               0.5617E+00      0.1000E-01
                                               0.1053E+01      0.2000E-01
                                               0.1872E+01      0.4000E-01
                                               0.2527E+01      0.6000E-01
                                               0.3062E+01      0.8000E-01
                                               0.3886E+01      0.1200E+00
                                               0.4490E+01      0.1600E+00
                                               0.6576E+01      0.5000E+00
                                               0.8298E+01      0.1000E+02
    7          10      0.2800E+02
                                               0.0000E+00      0.0000E+00
                                               0.1515E+01      0.1000E-01
                                               0.3197E+01      0.2000E-01
                                               0.6647E+01      0.4000E-01
                                               0.8162E+01      0.6000E-01
                                               0.8414E+01      0.8000E-01
                                               0.8162E+01      0.1200E+00
                                               0.7825E+01      0.1600E+00
                                               0.7825E+01      0.5000E+00
                                               0.7825E+01      0.1000E+02
    8          10      0.3368E+02
                                               0.0000E+00      0.0000E+00
                                               0.9375E+00      0.1000E-01
                                               0.1979E+01      0.2000E-01
                                               0.4115E+01      0.4000E-01
                                               0.5052E+01      0.6000E-01
                                               0.5208E+01      0.8000E-01
                                               0.5052E+01      0.1200E+00
                                               0.4844E+01      0.1600E+00
                                               0.4844E+01      0.5000E+00
                                               0.4844E+01      0.1000E+02
    9          10      0.3926E+02
                                               0.0000E+00      0.0000E+00
                                               0.9375E+00      0.1000E-01
                                               0.1979E+01      0.2000E-01
                                               0.4115E+01      0.4000E-01
                                               0.5052E+01      0.6000E-01
                                               0.5208E+01      0.8000E-01
                                               0.5052E+01      0.1200E+00
                                               0.4844E+01      0.1600E+00
                                               0.4844E+01      0.5000E+00
                                               0.4844E+01      0.1000E+02


TIP   LOAD        TIP MOVEMENT
    KIP               IN.
```

0.0000E+00	0.0000E+00
0.2370E-01	0.1000E-03
0.1676E+00	0.5000E-02
0.2370E+00	0.1000E-01
0.5299E+00	0.5000E-01
0.7494E+00	0.1000E+00
0.1060E+01	0.2000E+00
0.1676E+01	0.5000E+00
0.2019E+01	0.1000E+01
0.2019E+01	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE
****

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.1404E+00	0.1464E-03	0.2370E-01	0.1000E-03
0.1168E+01	0.1349E-02	0.5013E-01	0.1000E-02
0.5733E+01	0.6693E-02	0.1676E+00	0.5000E-02
0.1120E+02	0.1331E-01	0.2370E+00	0.1000E-01
0.4446E+02	0.6380E-01	0.5299E+00	0.5000E-01
0.6653E+02	0.1221E+00	0.7494E+00	0.1000E+00
0.1418E+03	0.5526E+00	0.1676E+01	0.5000E+00
0.1462E+03	0.1055E+01	0.2019E+01	0.1000E+01
0.1544E+03	0.2058E+01	0.2019E+01	0.2000E+01

Lateral Earth Pressure

GZA
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207-879-9190
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Engineers and Scientists

JOB: 09.0026024.00 Johnson Road Bridge
SUBJECT: Lateral Earth Pressures
SHEET: 1 OF 2
CALCULATED BY B. Cardali 12/2/2021
CHECKED BY A. Blaisdell 2/22/2022

Subject:

Eval uate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifica9ons, 9th Edi9on (2020)
3. Massachuse<s Department of Transporta9on Highway Division LRFD Bridge Manual Part I, sec9on 3.10.8

Input Parameters:

$\beta := 0\text{deg}$ — Angle of backfill to the horizontal

$\theta := 90\text{deg}$ — Angle of backface of wa ll to the horizontal

$\phi := 32\text{deg}$ — Effec9ve angle of internal fric9on *(Granular borrow, Soil Type 4, BDG Table 3-3)*

$\delta_f := 19.5\text{deg}$ — Average value of fric9on angle between, precast concrete and clean sand/silty sand-gravel mixture *(AASHTO LRFD Table 3.11.5.3-1)*

Passive Earth Pressure on Integral Backwall:

Per BDG Sec9on 5.4.2.11, developi ng full passi ve pressure requires that ra9o of latera l abutment movement (y) to abutment height (Hb) exceeds 0.005. If the calculated rota9on is significantly less, Rankine earth pressure may be considered.

$y := 0.78\text{in}$ — Expansion deflec9on From structural engineer

$H_b := 11.7\text{ft}$ — Abutment Height

$\frac{y}{H_b} = 0.0056$ — Ra9o of lateral movement to abutment height is greater than 0.005, use coulomb passive earth Pres sure

Earth Pressure Coefficients:

Since the ra9o of l ateral movement is greater than .005 but not significantly more, GZA evaluated the typical Coloumb values and compared values to the Massachuse< s DOT methodology presented in sec9on 3.1.8 for design of the Johnson Road bridge .

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JOB: 09.0026024.00 Johnson Road Bridge
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SHEET: 2 OF 2
CALCULATED BY B. Cardali 12/2/2021
CHECKED BY A. Blaisdell 2/22/2022

MassDOT Sec9on 3.10.8 presents the plot and calcula9on shown below for a gravel borrow material.

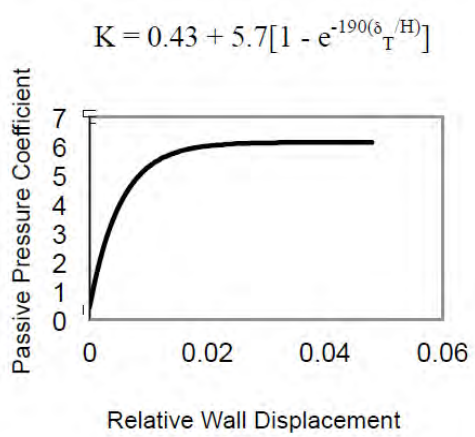

Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

$$\omega := \frac{y}{H_b} = 0.0056$$

$$K_{p.mass} := 0.43 + 5.7\left(1 - \exp\left(-190 \cdot \frac{y}{H_b}\right)\right) = 4.15$$

$$K_{p.mass} = 4.15$$

The passive earth pressure coefficient (Kp) for structural fill is 4.15 using the MassDOT methodology. We an9cipate it would be equal or higher for the lightweight fill since the fricton angle is higher. Therefore a range of 4.15 to 5.0 is appropriate for Kp for lightweight fill considering the MassDOT Methodology. We recommend that the structural engineeer evaluate both limits of Kp for lightweight and u9lize the value of Kp that creates the highest load scenario on the abutment.

Frost Calculation

March 2014

Figure 5-1 Maine Design Freezing Index Map

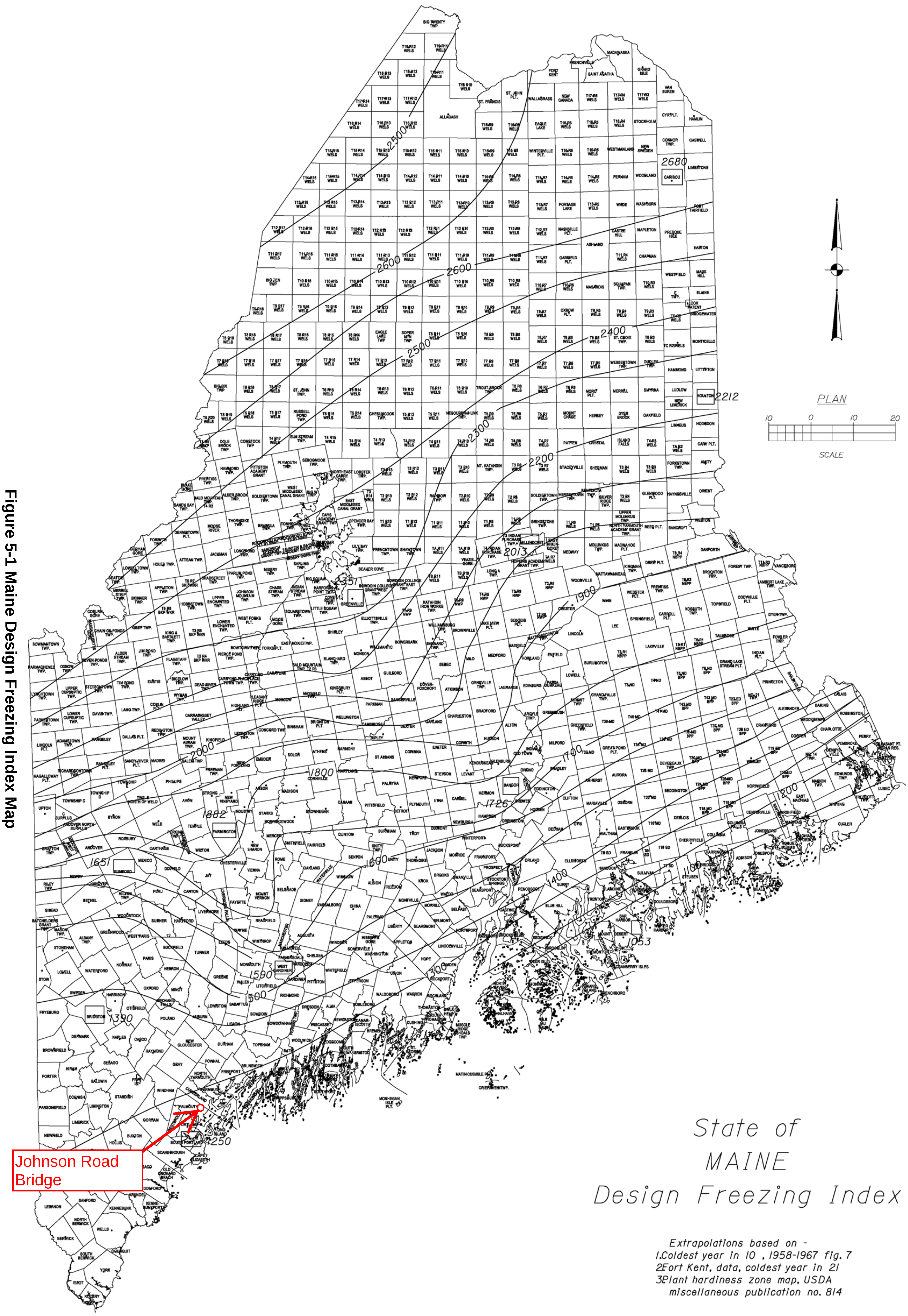

Table 5-1 Depth of Frost Penetration

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

1275

wc 10 to 20%
average = 5.75'

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

Fills soils are anticipated to be present abutment bearing elevation and near the elevation of the pier pile caps and anticipated granular fill adjacent to the pile cap. The granular material is coarse-grained with water contents of approximately 10-15%. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing Index of 1275 the estimated depth of frost penetration is 5.75 feet.