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GEOTECHNICAL DESIGN REPORT

HALL BRIDGE NO. 3159 OVER BLACK STREAM

MAINE DOT WIN 022226.00

CANAAN, MAINE

Prepared for:

Maine Department of Transportation
Augusta, Maine

March 2021
09.0026000.00

Prepared by:

GZA GeoEnvironmental, Inc.

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VIA EMAIL

March 15, 2021
File No. 09.0026000.00

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

Re: Geotechnical Design Report
Hall Bridge No. 3159
Maine Department of Transportation WIN 22226.00
Canaan, Maine

Dear Laura:

We are pleased to provide this Geotechnical Design Report (GDR) for Maine Department of Transportation (MaineDOT) Bridge No. 3159 over Black Stream in Canaan, Maine. Our work was completed in accordance with GZA GeoEnvironmental, Inc.'s Project Contract dated October 2, 2018, Agreement No. 20150608000000000793 and July 13, 2020 modification, our June 24, 2020 proposal, and the *Limitations* included in **Appendix A** of this report.

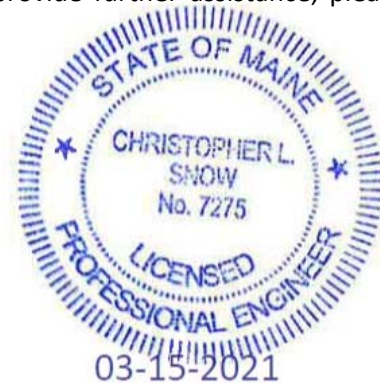
It has been a pleasure serving the MaineDOT on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the Canaan Bridge No. 3159 over Black Stream. Our services are subject to the *Limitations* contained in **Appendix A** of this report. Our services were provided in accordance with GZA's Proposal No. 09.P000017.21, dated June 24, 2020. This report is subject to the *Limitations* included in **Appendix A**.

1.1 BACKGROUND

Canaan Bridge No. 3159 carries Route 23 over Black Stream in Canaan, Maine at the location shown on **Figure 1**. The existing bridge plans (1934 and 1974) indicate that the bridge is a 23-foot-wide, three-span structure with double-bent piers and timber bent pile abutments with timber lagging back faces and steel tiebacks connected to timber pile dead men. The span lengths are approximately 29 feet, 43.5 feet and 29 feet with an overall length of approximately 104 feet. The deck was replaced in 1974 with a steel beam and concrete deck system and the substructures were reused.

We understand that the proposed replacement bridge will consist of a 113-foot-long, single-span structure with pile-supported integral abutments. Plans are to use a temporary bridge to maintain one lane of traffic during construction. Each abutment will be supported by driven H-piles. Based on information provided by VHB, we understand the maximum factored axial pile load is 329 kips for the strength loading condition and the preferred 5-pile configuration. VHB estimated that thermal deformation of the bridge superstructure could result in 0.529 inches of lateral deflection at each abutment.

Significant changes to the existing horizontal and vertical alignments are not anticipated. However, the Abutment 1 approach embankment will be widened to allow for an increase from the current 23-foot width to the proposed 34-foot width. New embankment sliver fill heights due to the widening will vary and are expected to be as high as 3 feet.

Temporary approach embankments and a single-lane temporary bridge are proposed to the north of the existing bridge to maintain traffic during construction.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted site visits to observe surficial conditions, traffic and boring access;
- Coordinated and observed preliminary and final subsurface exploration programs, consisting of a total of four test borings, to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate classification and engineering properties of the site soil and bedrock;



- Completed geotechnical evaluations for soil and bedrock properties; stability and settlement of permanent and temporary approach embankments; frost susceptibility and drainage of approach embankments; AASHTO load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; pile downdrag loads; pile drivability; lateral pile evaluations; and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles, lateral earth pressures and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2018 consisting of three test borings and a final design exploration program in 2020 consisting of one test boring. The preliminary as-drilled boring locations and elevations were surveyed by Maine Department of Transportation (MaineDOT), and the final design as-drilled boring location was measured by taped measurements from existing bridge components, and are shown on **Figure 2**. The elevation at BB-CBS-201 was estimated based on bridge deck elevations.

New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed continuously in the upper 12 feet, and at 5-foot typical intervals thereafter in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler. The sampler was driven with a 140-lb calibrated automatic hammer with a 30-inch drop from a truck or ATV-mounted drill rig. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Elevations referenced in this report are in feet and refer to the North American Vertical Datum of 1988 (NAVD88). Additional details of each program are described below.

2.1 PRELIMINARY BORINGS

GZA completed three (3) test borings (BB-CBS-101 through BB-CBS-103) between October 19 and October 29, 2018. The borings were drilled in the northern (left) lane at the center of the bridge and behind each existing abutment. The abutment borings were backfilled with cuttings, crushed stone, and asphalt cold patch. The concrete deck was filled with quick-set concrete at the location of the borings conducted on the bridge.

The borings were drilled to depths of approximately 41.0 to 63.2 feet below ground surface. All borings were cored approximately 11.0 to 11.3 feet into bedrock. *In situ* field vanes were taken within the silt layer and a thin-walled tube sample was taken in boring BB-CBS-102 for laboratory testing of the cohesive soil. Bedrock cores were obtained using NQ wire-line coring equipment in the preliminary design borings. Photographic logs of the recovered rock core specimens are included in **Appendix C**.



2.2 FINAL BORINGS

GZA completed one (1) supplemental test borings (BB-CBS-201) on July 29, 2020. The boring was drilled in front of the east abutment to a depth of approximately 24 feet bgs, terminating in the Glacial Till in accordance with the planned scope of work.

In situ field vanes were taken within the silt layer and two thin-walled tube samples were successfully retrieved for laboratory testing of the cohesive soil.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil-testing program to assess the gradation and engineering characteristics of the soil; R.W. Gillespie & Associates of Biddeford, Maine and Soil Metrics of Cape Elizabeth, Maine were retained to perform the strength testing of the cohesive soil encountered. The testing program consisted of:

- 9 gradation analysis / AASHTO Classification / Unified Soil Classification System / Frost Classification assessments;
- 1 set of Atterberg Limits;
- 12 moisture content tests;
- 2 organic content tests;
- 1 Shelby tube processed by R.W. Gillespie (lab vanes, unit weight, and moisture content); and
- 1 Shelby tube processed by Soil Metrics (1D consolidation, Atterberg Limits, Grain size analysis)

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on the *Surficial Geologic Map of Maine* (Anderson et al., 1985; on-line version), the site is mapped within a Glacial Till deposit. This unit is described as a heterogeneous mixture of sand, silt, clay, and stones, and may include boulders deposited by glacial ice, with local beds and lenses of variably washed and stratified sediments. An area of Glaciomarine deposit is mapped along the stream alignment. The Glaciomarine unit consists of silt, sand, and minor amounts of gravel, and commonly a clayey silt.

An Esker deposit is mapped along the west side of the general alignment of the Black Stream, and consists of gravel and sand with minor amounts of till. Swamp, marsh, and bog deposits are also mapped to the east of the bridge, which contain peat, muck, clay, silt, and sand formed by accumulation of sediments and organic material in depressions and other poorly drained areas.



Bedrock at the site is mapped as granofels, interlayered with Schist of the Sangerville Formation, based on the Bedrock Geologic Map of Maine (Osberg et al., 1985; on-line version).

4.2 SUBSURFACE PROFILE

Five soil units were encountered beneath surficial pavement in the test borings: Fill, River Bottom Deposit, Peat, Silt, and Glacial Till. Approximately 8 to 12 inches of asphalt pavement was encountered in the borings drilled through the approaches, and 2.5 feet of riprap fill was encountered in front of the east abutment. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile on **Figure 3**.

GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	4.8 to 10	Variable ranging <u>from</u> : Brown, loose to medium dense, fine to coarse SAND, little to some Gravel, trace Silt <u>to</u> : very dense, sandy GRAVEL, trace Silt (USCS: SM, SP, GW-GM, SW-SM). <ul style="list-style-type: none">• MaineDOT Frost Classification = 0-II• Encountered in borings BB-CBS-101, -103, and -201
Peat	1	Brown wet PEAT (USCS: OL) <ul style="list-style-type: none">• Encountered in boring BB-CBS-101 only
River Bottom Deposit	1 to 3	Brown, very loose, Silty SAND with wood and leaves (USCS: SM). <ul style="list-style-type: none">• Encountered in BB-CBS-102 and BB-CBS-201
Silt	13 to 23	Variable: <u>From</u> Grey, soft to stiff Clayey SILT, <u>to</u> Sandy SILT <u>to</u> SILT with little to some fine Sand in interbedded seams (USCS: ML) <ul style="list-style-type: none">• Sand observed in interbedded seams• Encountered in all borings
Glacial Till	14 to 21	Variable: <u>From</u> Brown, medium dense to dense, fine to coarse SAND, trace to some Silt, little to some Gravel <u>to</u> Gravelly fine to coarse SAND, little Silt (USCS: SM, SW). <ul style="list-style-type: none">• Encountered in all borings• Probable cobbles/boulder encountered in BB-CBS-101, -103 and -201 ranging in thickness from 3.2' to 1.2'
Encountered Top of Bedrock Elevation		Abutment 1: Approx. El. 196.3 Abutment 2: Approx. El. 185.0

4.2.1 Bedrock

Bedrock was cored in all test borings and was described as hard, fresh, fine-grained, grey, SCHIST. The primary joints were typically very close to moderately spaced, low angle to moderately dipping, undulating, rough to smooth, fresh to discolored, and partially open to open. The secondary joints were typically close to moderately spaced, high angle, planar to undulating, rough to smooth, fresh to



discolored, open. The Rock Quality Designation (RQD) ranged from 0 to 69 percent, with a weighted average RQD of 45 percent, indicating an average Rock Mass Quality of Poor.

4.2.2 Groundwater

Groundwater was encountered at depths of approximately 3.7 to 9.2 feet below ground surface in the test borings. These depths correspond to approximately El. 225.5 to 227.2, and were within a few feet of stream level at the time of drilling. Water levels were generally measured in completed boreholes shortly after completion of drilling and were likely affected by the water introduced during drilling. The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater levels will occur due to variations in season, precipitation, stream level and other factors. 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 based on the *2017 AASHTO LRFD Bridge Design Specifications, 8th Edition with Interims*, (AASHTO), and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

The replacement bridge is proposed to be constructed on or close to the existing horizontal and vertical alignments. Grading within the limits of the existing roadway will be limited to minor cuts and fills on the order of less than 1 foot. The approach embankments will be widened on both sides to allow for an increase from the current 23-foot width to the proposed 34-foot width, with typical side slope angles of approximately 2H:1V or flatter, resulting in widened embankment sliver fill heights as high as approximately 3 feet. A steeper side slope inclination (1.7H:1V or flatter) is proposed at the location of the Abutment 1 approach slab from approximately STA 102+75 to STA 108+85. The embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over the existing embankment/side slopes after muck removal.

GZA interpreted the generalized soil profile at each abutment and summarized the results in the table below. The strength parameters for each soil unit were developed using the corrected SPT N-values and soil types encountered in the borings. Supporting calculations are presented in **Appendix E**.



GENERALIZED SUBSURFACE CONDITIONS, PROPOSED APPROACH FILLS				
Soil Unit	Total Unit Weight (pcf)	Effective Friction Angle (deg) / Undrained Shear Strength (psf)	Estimated Thickness ¹ (feet)	
			West Approach (Abutment 1)	East Approach (Abutment 2)
Fill (Sand, Gravel)	125	32	9.5	8
Peat	95	200	1.0	-
Silt	95	28	16	23
Glacial Till	130	36	14	21

Note:

1. Estimated thickness is measured from existing approach embankment pavement grades.

5.2.1 Settlement

Approach Embankments

We anticipate approximately under 1 inch of total settlement due to the proposed 3 feet of new fill. We anticipate a portion of the settlement will occur as the load is applied. Considering the variability of the deposits, it's possible that greater than 0.4 inches of settlement may occur at some locations within the new abutments after pile installation. Therefore the piles should be designed for downdrag loading from movement of the fill and silt relative to the piles. Downdrag loads are discussed further in **Section 5.5**.

Peat was encountered in boring BB-CBS-101 on the west approach. It could be present on both approaches. If present in proposed embankment widening areas, it should be removed prior to placing embankment fill to prevent excessive post-construction settlement and/or stability concerns.

Temporary Bridge Embankments

The temporary embankments will be contractor-designed. VHB provided possible grading for a single-lane detour for GZA's design considerations. The plans provided by VHB show approach fills for the temporary bridge that tie into the existing road at approximately El. 236, corresponding to as much as 11 feet of fill above the existing grade (El. 225). Based on the grading provided by VHB, we anticipate typical 2H:1V side slopes on the temporary embankment, and that the embankments will consist of engineered fill placed over the existing ground.

We anticipate as much as 6 inches of settlement from the proposed 11 feet of new fill due to temporary embankment widening will occur rapidly as the load is applied. Due to variability of deposit, we anticipate this settlement could take 2 months to a year to complete.

5.2.2 Global Stability

GZA completed lateral and longitudinal stability analyses to assess the factors of safety against rotational instability of the proposed slope modifications. We used the computer analytical software *Slope/W 2020*, developed by GeoSlope International, based on the Morgenstern-Price method.

Lateral

The proposed typical 2H:1V and the Abutment 1 steepened embankment inclinations were modeled. An entry/exit search technique was used to identify the slip surface with the lowest calculated factor of safety. The search limits, slope geometry and results of analyses are included in **Appendix E**. The results indicated



that the factor of safety was in excess of 1.3 for the typical 2H:1V or flatter approach embankments which is suitable for support of embankments without structures. The results also indicated that the factor of safety was in excess of 1.5 for the side slope at the location of the approach slab which includes a plain riprap detail and is suitable for support of embankments with structures.

Longitudinal

Two longitudinal analyses were conducted to determine the factor of safety against rotational failure through the abutments and wingwalls. The first analysis was conducted through the center of the abutment, and it was assumed the new HP14x89 piles carry the full weight of the abutment and $\frac{1}{2}$ of the traffic surcharge, and the beneficial reinforcing effect of the piles was ignored in Slope/W. The second analysis was conducted normal to the wingwall, outside the limits of the approach slab. It was assumed the full weight of the abutment was carried by the subgrade, and included the traffic surcharge along the proposed roadway.

The results of both analyses indicate a minimum calculated factor of safety of 1.5 for the Abutment 1 profile. Abutment 2 was not analyzed separately because GZA judged the soil profile to be less critical there in the absence of the organic layer and because the stream channel is closer to Abutment 1. Therefore, the global stability is considered acceptable in the longitudinal direction.

Temporary Embankments

A planning-level stability analysis was conducted for the side slopes of a possible west temporary approach embankment. The analysis was based on a maximum fill height of approximately 11 feet and a 2H:1V side slope. The results indicated a calculated factor of safety between 1.3 and 1.5, which is suitable for support of embankments without structures. However, it is less than the 1.5 minimum factor of safety typically required for structures.

For preliminary planning purposes, we anticipate that a sheet pile or other temporary retaining structure could be used to form the abutments, contain fills and prevent global instability in front of and on the sides of the abutments. Alternatively, the detour could be constructed on a temporary trestle with flatter temporary slopes beneath.

Peat was encountered in boring BB-CBS-101 and may be present and could be thicker than 1 foot at both temporary approaches. If present in proposed temporary embankment areas, it should be removed prior to placing embankment fill to prevent excessive post-construction settlement and/or stability concerns.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment Foundations

Based on constructability and cost considerations, VHB selected an integral abutment bridge supported on steel H-piles. Given the depth to rock and the nature of the overburden soils, we anticipate that the piles will be supported by a combination of skin friction in the overburden soil and end bearing on or near bedrock.

5.4 SEISMIC DESIGN CONSIDERATIONS



Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value of the soil materials encountered in the borings. LRFD allows the assumption that rock within the upper 100 feet of the profile has an N-value equal to 100. However, the SPT N-value used to determine the site class was conservatively evaluated by including only the blow counts, undrained shear strength, and thickness of soil above the rock, reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet. The average SPT N-value is less than 15 blows per foot at both abutments. Therefore, the bridge should be assigned to Site Class E.

5.5 EVALUATION OF FOUNDATIONS

The VHB plans show the bridge will be supported on ASTM A572 Grade 50 steel HP14x89 piles in a 5-pile per abutment, integral abutment configuration. Design considerations and evaluations are presented below.

5.5.1 Pile Design Considerations

Based on our experience with similar soils, we anticipate that the proposed HP14x89 piles will be driven to refusal on or near the top of rock to achieve the required axial geotechnical resistance. The axial geotechnical resistance of piles was calculated using the computer analytical software *APile* by Ensoft based on the Nordlund method with a critical depth (10B) in accordance with LRFD Section 10.7. The results indicate that the piles will gain on the order of 54 to 74 kips side resistance when driven to/near the top of rock and that the remainder will be developed in bearing on or near bedrock, indicating approximately 10 to 13 percent of the nominal resistance would be developed as side friction. Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by VHB do not include uplift loading on the piles.

By utilizing steel H-piles for support of the abutments, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

5.5.2 Load and Resistance Factors

Piles should be driven to achieve the required nominal geotechnical resistance of the piles. In GZA's experience for piles gaining a significant portion of their geotechnical resistance on bedrock, the drivability resistance will control the geotechnical static resistance of the pile. The piles will be driven to a nominal resistance calculated by dividing the maximum factored pile load (Strength I load case) by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1. Resistance factors for service and extreme limit state design should be taken as 1.0.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor $\phi_c=0.50$, per AASHTO LRFD Article 10.7.3.2.3 for hard driving conditions. 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 LRFD Article 6.5.4.2, the axial resistance factor $\phi_{cc}=0.75$ and the flexural resistance factor $\phi_f=1.0$ 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 LRFD Table 3.4.1-2 for strength limit state foundation design.



A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck, per MaineDOT BDG Section 5.4.2.11.

5.5.3 Downdrag

As discussed previously, given the potential for greater than 0.4 inch of settlement to occur relative to the abutment piles, the piles should be designed to resist downdrag loading. Based on the subsurface stratification and estimated settlement, we estimate an unfactored downdrag load of approximately 52 kips will occur on piles at both abutments.

Side friction contributing to downdrag load was estimated using the β -method, assuming $\beta = 0.23$ for the Silt. Based on past practice, a load factor of 1.0 was applied to the calculated downdrag resistance, which was added to the maximum factored load provided by VHB.

5.5.4 Pile Type and Loading

Five steel HP14x89 piles are proposed to support each abutment. VHB provided a maximum factored axial load of 329 kips per pile for the strength condition. The estimated downdrag load is 52 kips. Therefore, the maximum factored axial load is 381 kips. The required nominal axial resistance is 586 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted during construction to assess nominal geotechnical pile resistance in accordance with AASHTO requirements.

5.5.5 Preliminary Wave Equation Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an ASTM A572 Grade 50 Steel HP14x89 pile with a nominal geotechnical resistance of 586 kips. Analyses were completed using a Delmag D30 diesel hammer with a ram weight of 6,600 pounds and a maximum rated energy of 59,730 foot-pounds (ft-lbs).

The analyzed pile lengths anticipate that the piles would be driven close to the top of rock elevation at each abutment. The calculated side resistance of approximately 10 and 13 percent was estimated using static analysis. The results are summarized below.

Pile Location and Type	Embedded Pile Length	Driving System	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
Abutment 1 HP 14x89	30 feet	Delmag D30*	586	44	8
Abutment 2 HP 14x89	40 feet	Delmag D30 **	586	44	7

* Hammer was operated on the second highest fuel setting (1273 psi).

** Hammer was operated on the highest fuel setting (1415 psi).

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 is judged acceptable to install the piles to the required nominal resistance. Results of the preliminary wave equation analyses are provided in **Appendix E**.

5.5.6 Lateral Pile Analysis

GZA developed soil profiles for lateral pile evaluations at each abutment. The profiles reflect the soil conditions encountered in the test borings.

L-PILE® INPUT PARAMETERS ABUTMENT 1, PILE LENGTH = 30 FT (BORING BB-CBS-101)						
Stratum	Soil Model	Top of Layer Elevation (ft- NAVD 88)	Layer Thickness (ft)	k (pci) / E50 / UC (psi)	ϕ' (deg) / Su (psf)	γ_e (pcf)
Silt	Reese Sand ¹	226	16	15	28	33
Glacial Till	Reese Sand	210	14	95	36	68

1. The silt had very low plasticity and was modelled as an equivalent granular material.

L-PILE® INPUT PARAMETERS ABUTMENT 2, PILE LENGTH = 40 FT (BORING BB-CBS-103)						
Stratum	Soil Model	Top of Layer Elevation (ft- NAVD 88)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg) / Su (psf)	γ_e (pcf)
Silt	Reese Sand ¹	226	20	15	28	33
Glacial Till	Reese Sand	206	20	95	36	68

1. The silt had very low plasticity and was modelled as an equivalent granular material.

GZA conducted lateral pile analyses based on a maximum thermal deflection of 0.529 inches, as provided by VHB. Lateral pile analysis used the Ensoft, Inc. LPILE version 2016.9.06 software. 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 14x89 piles aligned for weak-axis bending parallel to the bridge centerline. The assumed axial load was 329 kips, representing the maximum factored axial load provided by VHB. The downdrag load is not included in the axial load for LPILE because it occurs at a greater depth and therefore does not affect the total stress at the pile head. Our results are summarized in the table below.

L-PILE® RESULTS				
Pile Type	Axial Load (kips)	Shear Force for Lateral deflection of 0.529 in. (kips)	Moment at Pile Head (ft-kips)	Total Stress at Pile Head (ksi)



Abutment 1 HP 14x89	329	15	-92.1	40
Abutment 2 HP 14x89	329	15	-92.1	40

The results indicate that a plastic hinge will not form in the piles due to the imposed lateral pile deformation.

5.6 ADDITIONAL FOUNDATION CONSIDERATIONS

5.6.1 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1750, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 6.5 feet. Consequently, new foundation levels should be set at least 6.5 feet below ground surfaces exposed to freezing temperatures.

Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-a and A-1-b with MaineDOT Frost Classification from 0 to II, indicating they are considered to exhibit low to moderate frost susceptibility. These materials are judged to be suitable for continued use beneath the approach roadway after reconstruction. In accordance with MaineDOT Standards, new backfill placed behind abutments will consist of non-frost-susceptible materials.

5.6.2 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.

Based on the estimated thermal bridge expansion of approximately ½ inch and the abutment height of approximately 10 feet, the calculated abutment rotation is 0.0042 feet/foot. In accordance with the requirements of the BDG Section 5.4.2.11, integral abutment reinforcement is to be designed for full Coulomb passive pressure if the wall rotation is greater than 0.005 feet/foot. Therefore, we conclude that Rankine passive earth pressure is appropriate for design. We recommend using a Rankine Coefficient of Passive Earth Pressure, $K_p = 3.25$ for design of backwalls and wingwalls.

Design lateral earth pressure recommendations are provided in **Section 6.3** of this report.

5.6.3 Gradation for Scour Analysis

Available laboratory data for the soil at the approximate elevation of the integral abutment footings was evaluated to estimate D_{50} values, as summarized in the table below. Full laboratory testing results are included in **Appendix D**.

Boring and Sample ID Number	Depth (feet)	W.C. (%)	D_{50} (mm)	Classification	
				USCS	AASHTO
BB-CBS-101, 7D	15.0-17.0	26.8	0.03	ML	A-4
BB-CBS-103, 3D	5.0-7.0	4.2	0.76	SW-SM	A-1-b



BB-CBS-103, 5D	9.0-11.0	19.8	0.02	ML	A-4
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6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Typical embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Slopes should be provided with loam and seed for permanent erosion protection. If steeper slopes are planned at the abutment approaches, riprap should be employed to limit surface erosion. Riprap should also be provided where the embankment side slopes will be near or below typical water levels in the Black Stream to protect from scour.

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 D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	2.5
F _a	2.5
F _v	3.5
A _s (Period = 0.0 sec)	0.188 g
SD _s (Period = 0.2 sec)	0.398 g
SD ₁ (Period = 1.0 sec)	0.161 g

Per AASHTO Article 4.7.4.2, single span bridges need not be analyzed for seismic loads, but the minimum requirements for superstructure connections and support lengths as specified in AASHTO Articles 4.7.4.4 and 3.10.9 apply.

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill behind new abutments and wingwalls should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Rankine Coefficient of Passive Earth Pressure, K_{pr} = 3.25 (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 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.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Pile Design

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles 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.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile 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.
- The piles should be driven to a nominal resistance of 586 kips, calculated by dividing the maximum factored pile load of 381 kips (including downdrag) by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the ASTM A572 Grade 50 HP14x89 piles can be driven to a nominal resistance of 586 kips using a diesel hammer with a rated energy of 59,730 ft-lbs without exceeding the allowable driving stress of 45 ksi (0.9F_y). The final penetration resistance was 7 to 8 blows per inch, which is within the MaineDOT preferred range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the top of rock elevations encountered in the borings and shown on the interpretive cross-section profile (approximately El 196 at Abutment 1 and approximately El. 185 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.
- We recommend that one pile in each substructure 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 at 24 hours after initial drive.
- Piles shall be spliced in accordance with MaineDOT Standard Specification Section 501.047.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction.



7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavations for abutment foundations and removal of existing foundations are anticipated to extend approximately 12 feet below existing pavement grades. It is our understanding that a temporary bridge will be used during construction of the new bridge, therefore, we anticipate sufficient space is available and if water conditions permit, the excavation slopes may consist of sloped, open cuts. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

Considering the proximity of the required abutment excavations to the river water level, management of water will be related to stream water levels at the time of construction. With the bottom of existing abutment elevations at approximately El. 223 to El. 225 and Q1.1 at approximately El. 226 for the proposed structure, water levels may be at or above the bottom of excavation level during construction. It may be desirable to over-excavate and place an 8- to 12-inch-thick crushed stone working mat to improve accessibility and allow dewatering.

We anticipate that the inflow of groundwater or surface water to excavations could be handled by open pumping from sumps installed at the bottoms of excavations if cofferdams are installed. Stacked sand bags or a porta-dam type system may be sufficient to limit inflow of surface water in lieu of a sheet pile cofferdam, given the relatively small anticipated head. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade and permit concrete placement in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.2 PILE INSTALLATION CONTROL

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

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, three of the four fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill typically meets MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill.

If the contractor wishes to reuse excavated material on site, 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 TEMPORARY BRIDGE AND APPROACH EMBANKMENTS

We recommend that the Contractor be required to submit design computations and plans that show that temporary embankments achieve acceptable minimum factors of safety for slope stability. AASHTO specifies that a minimum factor of safety of 1.3 shall be provided for approach embankments, and 1.5 for slopes that contain or support structures.



As noted in **Section 5.2.1**, up to approximately 6 inches of settlement was estimated for a typical temporary approach embankment. The Contractor's design should provide means to accommodate possible settlement of the approaches to maintain serviceability of the temporary bridge.



3/15/2021

**HALL BRIDGE NO. 3159 OVER BLACK STREAM
GEOTECHNICAL DESIGN REPORT**

09.0026000.00

FIGURES



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SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCGIS 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 :



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

SCALE IN FEET



PROJ. MGR.: NVW
DESIGNED BY: NVW
REVIEWED BY: CLS
OPERATOR: ADM

DATE: 04-02-2019

LOCUS PLAN


**HALL BRIDGE
CANAAN, MAINE**

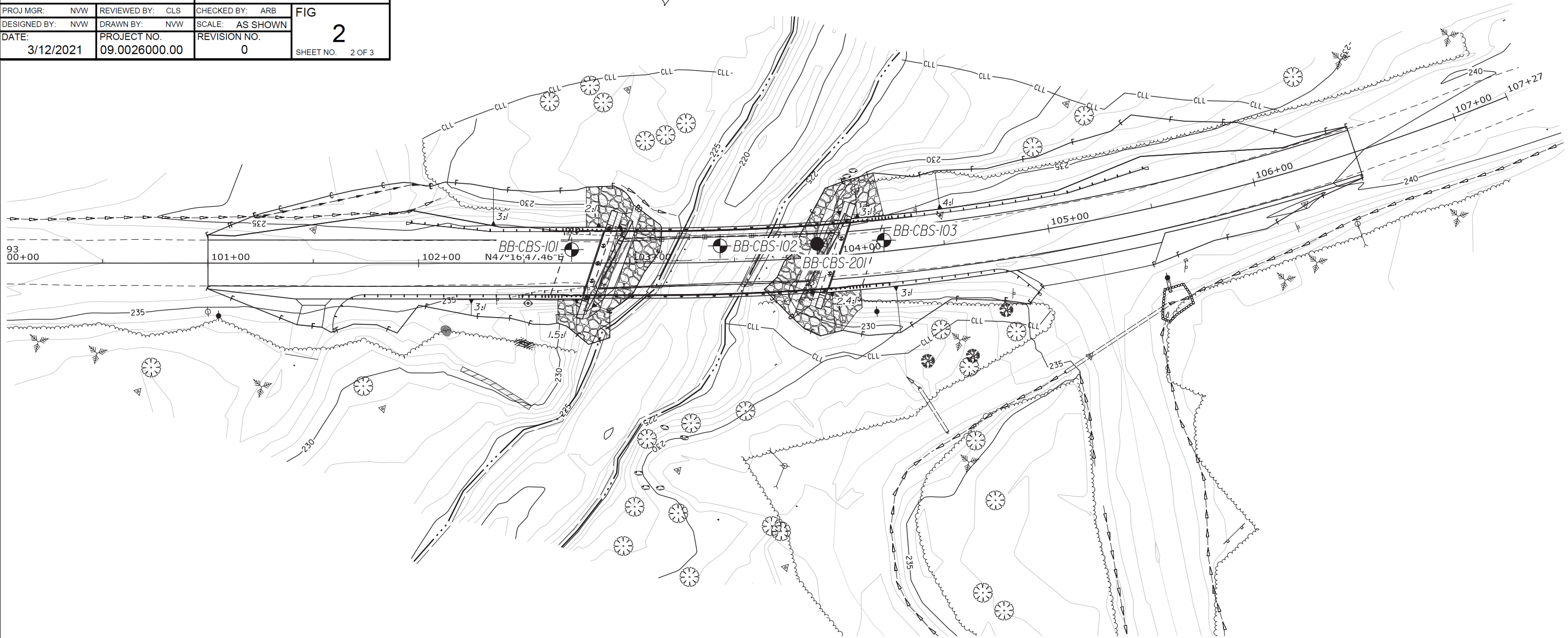
JOB NO.
09.0026000.00

FIGURE NO.
1

HALL BRIDGE NO. 3159
MAINEDOT WIN 22226.00
CANAAAN, ME

BORING LOCATION PLAN



PREPARED BY:  GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: NVW	REVIEWED BY: CLS	CHECKED BY: ARB	FIG 2 SHEET NO. 2 OF 3
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	
DATE: 3/12/2021	PROJECT NO. 09.0026000.00	REVISION NO. 0	

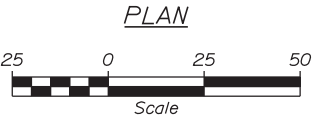


NOTES

- 1) Base map developed from electronic files (Alignments.dgn, Bridge.dgn, Contours.dgn, RWPlan.dgn, Text.dgn, Topo.dgn, highway.dgn, and Highway_TempBridge.dgn) provided by VHB on February 9, 2021.
- 2) The as-drilled locations of the test borings were surveyed by MaineDOT and developed from an electronic file (2226 Canaan Clearance Report.xlsx) provided by MaineDOT on January 17, 2019.
- 3) BB-CBS-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between October 19 and November 1, 2019. BB-CBS-200 series bridge boring was performed by New England Boring Contractors and observed by GZA personnel on July 29, 2020.


BORING LOCATION PLAN LEGEND

-  BB-CBS-103 Location and designation of 100-Series cased wash boring
-  BB-CBS-201 Location and designation of 200-Series cased wash boring



STATE OF MAINE DEPARTMENT OF TRANSPORTATION	2222600	WIN 22226.00	BRIDGE NO. 3159 BRIDGE PLANS



	SIGNATURE 7275	P.E. NUMBER 7275	DATE MARCH 14, 2021

PROJ. MANAGER	M. KERSBERGEN	BY	DATE
DESIGN-DETAILED	NVW	NVW	3/12/2021
CHECKED-REVIEWED	CLS	ARB	3/12/2021
DESIGN-DETAILED	CLS		
DESIGN-DETAILED	CLS		
REVISIONS	1		
REVISIONS	2		
REVISIONS	3		
REVISIONS	4		
FIELD CHANGES			

HALL BRIDGE ROUTE 23 (HARTLAND ROAD) OVER BLACK STREAM	
CANAAAN SOMERSET COUNTY	
BORING LOCATION PLAN	

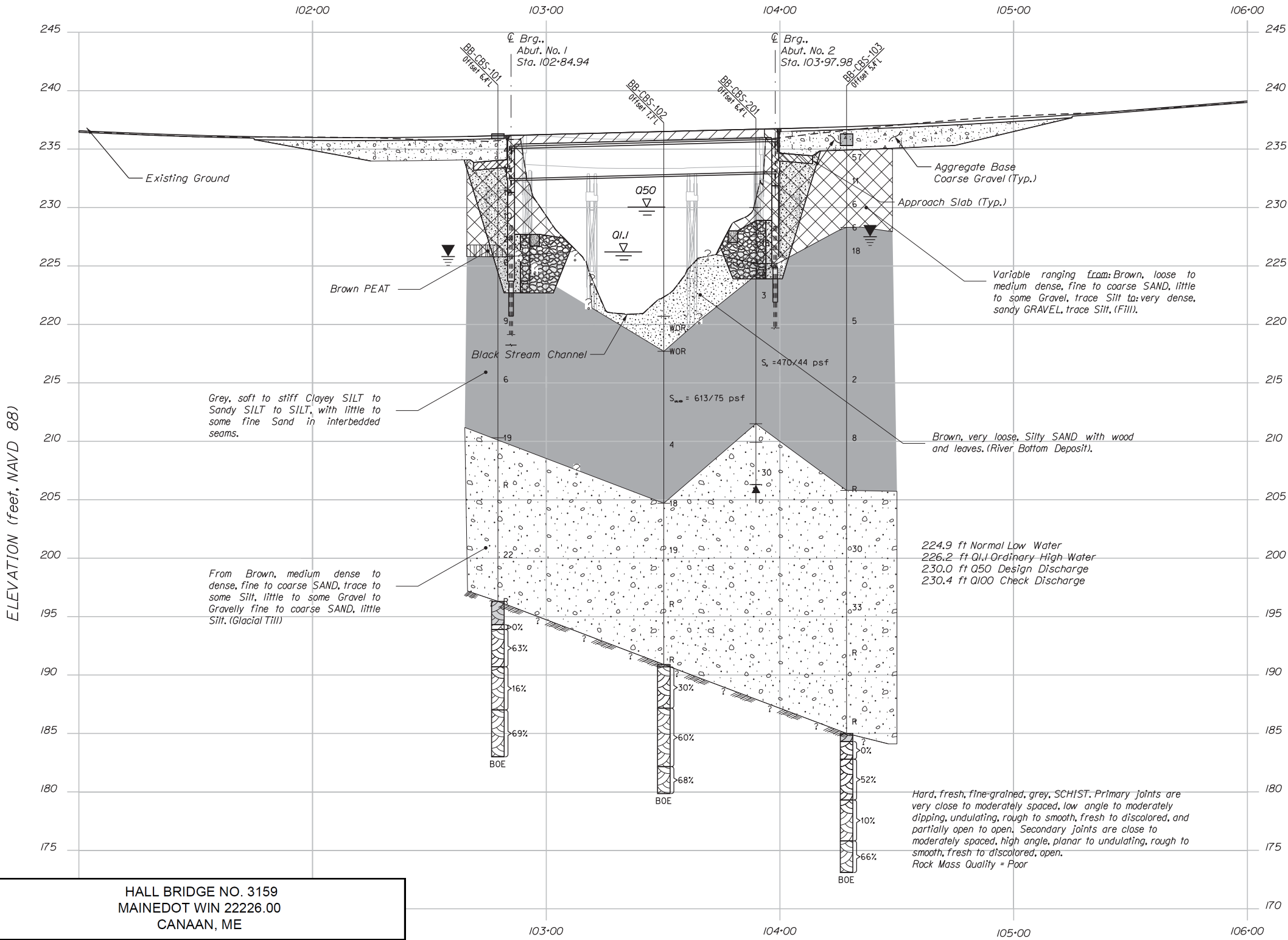
SHEET NUMBER

6

OF 30

PREPARED BY:





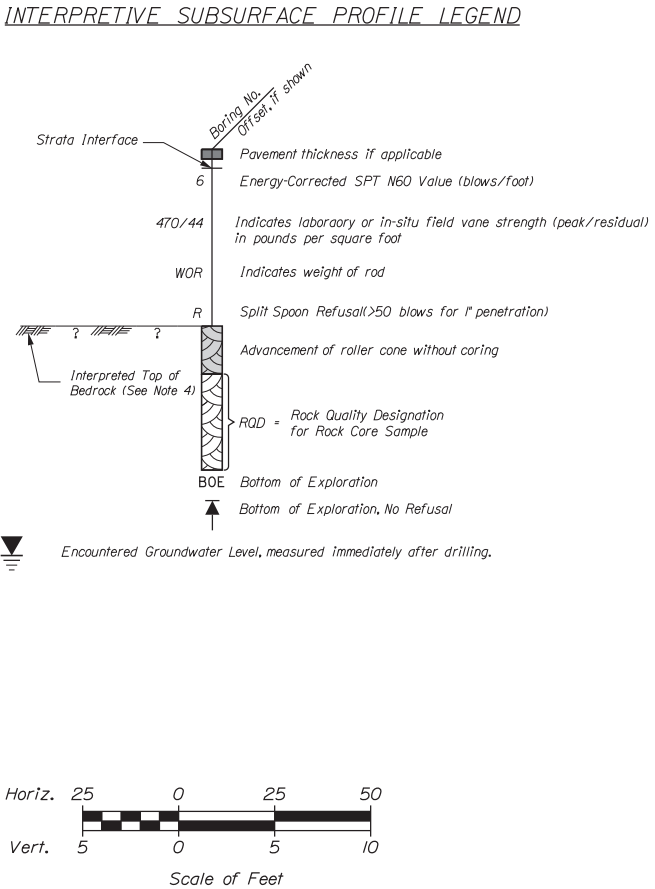
NOTES

1) Base map developed from electronic files provided by VHB dated February 9, 2021. Files included Profile_HWY.dgn, z_Profile.dgn, and z_Ex-Profile.dgn)

2) The as drilled locations and elevations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA.

3) BB-CBS-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between October 19 and November 1, 2018. BB-CBS-200 series bridge boring was performed by New England Boring Contractors and observed by GZA personnel on July 29, 2020.

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



HALL BRIDGE NO. 3159 MAINEDOT WIN 22226.00 CANAAN, ME			
INTERPRETIVE SUBSURFACE PROFILE			
PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: MAINE DEPARTMENT OF TRANSPORTATION	
PROJ MGR: NVW	REVIEWED BY: CLS	CHECKED BY: ARB	FIG
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	3
DATE: 3/12/2021	PROJECT NO. 09.0026000.00	REVISION NO. 0	SHEET NO. 3 OF 3



STATE OF MAINE DEPARTMENT OF TRANSPORTATION	2222600	WIN 22226.00	BRIDGE NO. 3159	BRIDGE PLANS
SIGNATURE 7275		P.E. NUMBER 7275		
DATE MARCH 14, 2021		DATE		
HALL BRIDGE ROUTE 23 (HARTLAND ROAD) OVER BLACK STREAM CANAAN SOMERSET COUNTY				
INTERPRETIVE SUBSURFACE PROFILE				
SHEET NUMBER 7 OF 30				



3/15/2021

**HALL BRIDGE NO. 3159 OVER BLACK STREAM
GEOTECHNICAL DESIGN REPORT**

09.0026000.00

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.



3/15/2021

HALL BRIDGE NO. 3159 OVER BLACK STREAM

GEOTECHNICAL DESIGN REPORT

09.0026000.00

APPENDIX B – GZA BORING LOGS

<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>					<div>Project: Hall Bridge Replacement #3159</div> <div>Route 23 Over Black Stream</div>			<div>Boring No.: BB-CBS-101</div>																																																																																																																																																																																																																																																																																																																																																																					
					<div>Location: Canaan, Maine</div>			<div>PIN: 22226.00</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Driller: New England Boring Contractors</div>				<div>Elevation (ft.) 236.3</div>				<div>Auger ID/OD: 4.5" SSA</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Operator: Brad Enos</div>				<div>Datum: NAVD 88</div>				<div>Sampler: Standard Splitspoon</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Logged By: B. Woodman</div>				<div>Rig Type: Mobile Drill B-53 (Truck)</div>				<div>Hammer Wt./Fall: 140#/30"</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Date Start/Finish: 10-19-18/10-19-18</div>				<div>Drilling Method: Drive & Wash</div>				<div>Core Barrel: NQ</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Boring Location: N473296.0, E1531236.0</div>				<div>Casing ID/OD: 4/4.5", 3/3.5"</div>				<div>Water Level*: 10.5' bgs</div>																																																																																																																																																																																																																																																																																																																																																																					
<div>Hammer Efficiency Factor: 0.895</div>				<div>Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/></div>																																																																																																																																																																																																																																																																																																																																																																									
<div>Definitions:</div> <div>D = Split Spoon Sample</div> <div>MD = Unsuccessful Split Spoon Sample attempt</div> <div>U = Thin Wall Tube Sample</div> <div>MU = Unsuccessful Thin Wall Tube Sample attempt</div> <div>V = Insitu Vane Shear Test</div> <div>MV = Unsuccessful Insitu Vane Shear Test attempt</div> <div>R = Rock Core Sample</div> <div>SSA = Solid Stem Auger</div> <div>HSA = Hollow Stem Auger</div> <div>RC = Roller Cone</div> <div>WOH = weight of 140lb. hammer</div> <div>WOR = weight of rods</div> <div>WO1P = Weight of one person</div> <div>S_u = Insitu Field Vane Shear Strength (psf)</div> <div>T_v = Pocket Torvane Shear Strength (psf)</div> <div>q_p = Unconfined Compressive Strength (ksf)</div> <div>N_{uncorrected} = Raw field SPT N-value</div> <div>Hammer Efficiency Factor = Annual Calibration Value</div> <div>N₆₀ = SPT N-uncorrected corrected for hammer efficiency</div> <div>N₆₀ = (Hammer Efficiency Factor/60%)*N_{uncorrected}</div> <div>S_{u(lab)} = Lab Vane Shear Strength (psf)</div> <div>WC = water content, percent</div> <div>LL = Liquid Limit</div> <div>PL = Plastic Limit</div> <div>PI = Plasticity Index</div> <div>G = Grain Size Analysis</div> <div>C = Consolidation Test</div>																																																																																																																																																																																																																																																																																																																																																																													
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="7">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td rowspan="6">0</td><td>1D</td><td>24/24</td><td>0.4 - 2.4</td><td>15-17-13-11</td><td>30</td><td>45</td><td>SSA</td><td>235.9</td><td rowspan="6"></td><td rowspan="6">-ASPHALT- Brown, dry, dense, fine to coarse SAND, some gravel, little silt, (Fill). Brown, dry, medium dense, Gravelly fine to coarse SAND, trace silt, (Fill). Brown, dry, medium dense, fine to coarse SAND, little gravel, (Fill). Brown, dry, loose, fine to coarse SAND, little gravel, (Fill). Brown, moist, medium dense, fine to medium SAND, little silt. Organic matter present - apparent Peat, (Fill). After sampling 5D, switch from solid stem auger to 4" casing. Top 6": Brown, wet, PEAT. Bottom 7": Grey, wet, Sandy SILT, trace organics. After sampling 6D, sampled at 5' intervals. Brown, wet, stiff, Clayey SILT, some fine sand, trace gravel. Brown, wet, medium stiff, Clayey SILT, some fine sand. 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25	9D	24/9	25.0 - 27.0	2-8-5-7	13	19		210.3		Top 8": Brown, wet, very stiff, Clayey SILT, some fine sand.																		
										Bottom 1": Fine to coarse SAND and gravel, (Glacial Till).																		
										Increased resistance during roller cone advancement 29.8'-31.0'. Grey, wet, fine to coarse SAND, little gravel, (Glacial Till).																		
30	10D	1/1	30.0 - 30.1	108/1"	-					110																		
										163																		
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	11D	24/15	35.0 - 37.0	9-9-13-20	22	33							Grey, wet, dense, fine to coarse SAND, little silt, (Glacial Till).															
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								179																				
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45	12D	0/0	40.0 - 40.0	50/0"	-			196.3		Spoon refusal at 40.0'; apparent rock fragments in wash return during roller cone advancement from 40.0'-42.0' (possible weathered rock). Drove 3" casing to 42.0', withdraw 4" to 35.0'. Seat 3" casing and set up to core.																		
50	R1	2/1	42.0 - 42.2	RQD = 0%								R1: Fractured pieces of gravel (SCHIST) Recovery = 25% Rock Mass Quality = Very Poor Rock Core Times (min:sec): 42.0-42.2' (3:00)																
	R2	38/35	42.2 - 45.4	RQD = 63%								R2: Hard, fresh, fine grained, grey, SCHIST. Primary joints are close to moderately spaced, low angle, undulating, rough, fresh to discolored, partially open to open. Secondary joints are closely spaced, high angle, undulating, rough, discolored, open. Recovery = 92% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 42.2-43.0' (1:35), 43.0-44.0' (2:55), 44.0-45.0' (2:40), 45.0-45.4' (2:04)																
												R3: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to moderately spaced, moderately dipping to high angle, undulating, rough, fresh to discolored,																
50	R3	44/42	45.4 - 49.1	RQD = 16%																								
50																												
	R4	48/48	49.1 - 53.1	RQD = 69%																								

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<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>					<div>Project: Hall Bridge Replacement #3159</div> <div>Route 23 Over Black Stream</div>			<div>Boring No.: BB-CBS-102</div>																																																																																																																																																																																							
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<div>Remarks:</div> <div>1. Automatic Hammer NEBC #D19 Energy Transfer Ratio = 0.713.</div> <div>2. The as-drilled locations were surveyed by MaineDOT. Cored through 7" concrete deck. Depth measured from mudline elevation, 15.0' below top of bridge.</div> <div>3. OC = Organic Content results from laboratory testing.</div> <div>4. 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.</div>																																																																																																																																																																																															
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Maine Department of Transportation						Project: Hall Bridge Replacement #3159 Route 23 Over Black Stream			Boring No.: BB-CBS-102			
Soil/Rock Exploration Log US CUSTOMARY UNITS						Location: Canaan, Maine			PIN: 22226.00			
Driller: New England Boring Contractors				Elevation (ft.): 220.7			Auger ID/OD: 4.5" SSA					
Operator: Tom Schaefer				Datum: NAVD 88			Sampler: Standard Splitspoon					
Logged By: N. Williams				Rig Type: ATV Mobile Drill			Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 10-30-18/11-01-18				Drilling Method: Drive & Wash			Core Barrel: NQ					
Boring Location: N473345.1, E1531286.6				Casing ID/OD: 4/4.5", 3/3.5"			Water Level*: River +5.6' bgs					
Hammer Efficiency Factor: 0.713				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>								
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25										Till).		
										Drove 4" casing to 27.3' and 3" casing to 28.2' to practical refusal. Washed ahead to 29.0' prior to casing advancement. Olive, wet, very dense, fine to coarse SAND, some silt, little gravel, (Glacial Till).		
30	7D	10/10	29.0 - 29.8	29-50/4"	-			190.9				
	R1	44/39	30.0 - 33.7	RQD = 30%			NQ					
										Split spoon refusal at 29.8' bgs; advance roller cone to 30.0' and set up to core. R1: Hard, fresh, fine grained, grey, SCHIST. Primary joints are very close to close, low angle, planar, rough to smooth, fresh to discolored, partially open to open. Secondary joints are close to moderately spaced, high angle, planar, smooth, fresh, open. Quartz seam throughout. Recovery = 89% Rock Mass Quality = Poor -SANGERVILLE FORMATION- Rock Core Times (min:sec): 30.0-31.0' (3:20), 31.0-32.0' (3:15), 32.0-33.0' (2:40), 33.0-33.7' (2:00) R2: Hard, fresh, fine grained to aphanitic, grey to dark grey, SCHIST with calcite stringers. Joints are close to moderately spaced, moderately dipping, planar, rough to smooth, fresh, open. Two high angle joints. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 33.7-34.7' (2:40), 34.7-35.7' (2:15), 35.7-36.7' (2:05), 36.7-37.7' (2:10), 37.7-38.7' (1:55) R3: Hard, fresh, fine grained to aphanitic, dark grey, SCHIST. Joints are moderately spaced, undulating to planar, rough, fresh, partially open to open. One high angle joint. Recovery = 96% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 38.7-39.7' (2:00), 39.7-40.7' (2:00), 40.7-41.0' (0:40)		
	R2	60/60	33.7 - 38.7	RQD = 60%								
35												
	R3	28/27	38.7 - 41.0	RQD = 68%								
40								179.7				
45												
50												
Remarks:												
1. Automatic Hammer NEBC #D19 Energy Transfer Ratio = 0.713. 2. The as-drilled locations were surveyed by MaineDOT. Cored through 7" concrete deck. Depth measured from mudline elevation, 15.0' below top of bridge. 3. OC = Organic Content results from laboratory testing. 4. 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.												
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.										Page 2 of 2 Boring No.: BB-CBS-102		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Hall Bridge Replacement #3159 Route 23 Over Black Stream				Boring No.: BB-CBS-103																																																																																																																																																																																																																																																															
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Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>235.3</td><td rowspan="20"></td><td>-ASPHALT-</td><td></td></tr><tr><td></td><td>1D</td><td>24/12</td><td>1.0 - 3.0</td><td>34-26-22-19</td><td>48</td><td>57</td><td></td><td></td><td>Brown, dry, very dense, Sandy GRAVEL, trace silt, (Fill).</td><td>G#7 A-1-a, GW-GM WC=2.5 WC=3.0</td></tr><tr><td></td><td>2D</td><td>24/12</td><td>3.0 - 5.0</td><td>9-5-4-3</td><td>9</td><td>11</td><td></td><td></td><td>Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).</td><td>G#9 A-1-b, SW-SM WC=4.2</td></tr><tr><td>5</td><td>3D</td><td>24/11</td><td>5.0 - 7.0</td><td>3-2-3-3</td><td>5</td><td>6</td><td></td><td></td><td>Tan, dry, loose, fine to medium SAND, some gravel, trace silt, (Fill).</td><td>WC=4.5</td></tr><tr><td></td><td>4D</td><td>24/10</td><td>7.0 - 9.0</td><td>4-2-3-4</td><td>5</td><td>6</td><td></td><td>228.3</td><td>Top 7": Tan, dry, loose, fine to medium SAND, some gravel, trace silt, (Fill). After sampling 4D, switch from solid stem auger to 4" casing.</td><td>G#11 A-4, ML WC=19.8</td></tr><tr><td>10</td><td>5D</td><td>24/13</td><td>9.0 - 11.0</td><td>8-7-8-10</td><td>15</td><td>18</td><td></td><td></td><td>Bottom 3": Grey/blue, dry, Clayey SILT, trace fine sand, organic odor. Blue/grey, moist, very stiff, Clayey SILT, little fine sand. Fine to medium sand in split spoon tip. After sampling 5D, sampled at 5' intervals.</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>21</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>26</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>28</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>29</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>31</td><td></td><td></td><td></td></tr><tr><td>15</td><td>6D</td><td>24/11</td><td>15.0 - 17.0</td><td>2-2-2-2</td><td>4</td><td>5</td><td>23</td><td></td><td>Grey, wet, medium stiff, SILT, some fine Sand, trace gravel.</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>20</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>26</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>31</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>33</td><td></td><td></td><td></td></tr><tr><td>20</td><td>7D</td><td>24/16</td><td>20.0 - 22.0</td><td>WOH-1-1-2</td><td>2</td><td>2</td><td>27</td><td></td><td>Grey, wet, soft, SILT, some fine sand, trace gravel, interbedded silt lenses.</td><td>G#12 A-4, ML WC=28</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>26</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>29</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>27</td><td></td><td></td><td></td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td>25</td><td></td><td></td><td></td></tr></table>												Depth (ft.)	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<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>				<div>Project: Hall Bridge Replacement #3159</div> <div>Route 23 Over Black Stream</div>		<div>Boring No.: BB-CBS-103</div>																																																																																																																																																																																																																													
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Recovery = 83% -SANGERVILLE FORMATION- Rock Mass Quality = Very Poor Rock Core Times (min:sec): 52.0-53.0' (4:30), 53.0-53.5' (2:05) R2: Hard, fresh, fine grained, grey, SCHIST. Joints are close to very close, moderately dipping to high angle, planar, rough, tight to partially open, fresh. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 53.5-54.0' (1:15), 54.0-55.0' (3:00), 55.0-56.0' (2:15), 56.0-57.0' (2:00) R3: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to close, low angle to moderately dipping, planar, rough to smooth, open. Extremely fractured resulting in angular gravel pieces from 57.8'-59.9'. Recovery = 100% Rock Mass Quality = Very Poor -SANGERVILLE FORMATION- Rock Core Times (min:sec): 57.0-58.0' (2:35), 58.0-59.0' (2:30), 59.0-60.0' (2:50), 60.0-60.5' (1:35) R4: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to moderately spaced, moderately dipping to high angle, planar, rough to smooth, tight to open, fresh. One high angle joint. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 60.5-61.5' (2:20), 61.5-62.5' (2:30), 62.5-63.5' (2:00)</td><td rowspan="3">Bottom of Exploration at 63.20 feet below ground surface.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>R3</td><td>42/42</td><td>57.0 - 60.5</td><td>RQD = 10%</td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">60</td><td>R4</td><td>32/32</td><td>60.5 - 63.2</td><td>RQD = 66%</td><td></td><td></td><td></td><td></td><td rowspan="3">173.1</td><td rowspan="3"></td><td rowspan="3"></td><td rowspan="3"></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">65</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">70</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="3">75</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>											Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	50	13D	7/7	50.0 - 50.6	77-50/1"	-				185.0		Brown/grey, wet, very dense, fine to coarse SAND, some silt, little gravel, (Glacial Till).										R1	18/15	52.0 - 53.5	RQD = 0%				NQ	55	R2	42/42	53.5 - 57.0	RQD = 52%							Increased resistance during roller cone advancement from 51.3' to 52.0', and set up to core. R1: Hard, fresh, fine grained, grey, SCHIST. Joints are very close, low angle, planar, smooth to rough, fresh, open. One high angle to vertical joint. Recovery = 83% -SANGERVILLE FORMATION- Rock Mass Quality = Very Poor Rock Core Times (min:sec): 52.0-53.0' (4:30), 53.0-53.5' (2:05) R2: Hard, fresh, fine grained, grey, SCHIST. Joints are close to very close, moderately dipping to high angle, planar, rough, tight to partially open, fresh. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 53.5-54.0' (1:15), 54.0-55.0' (3:00), 55.0-56.0' (2:15), 56.0-57.0' (2:00) R3: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to close, low angle to moderately dipping, planar, rough to smooth, open. Extremely fractured resulting in angular gravel pieces from 57.8'-59.9'. Recovery = 100% Rock Mass Quality = Very Poor -SANGERVILLE FORMATION- Rock Core Times (min:sec): 57.0-58.0' (2:35), 58.0-59.0' (2:30), 59.0-60.0' (2:50), 60.0-60.5' (1:35) R4: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to moderately spaced, moderately dipping to high angle, planar, rough to smooth, tight to open, fresh. One high angle joint. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 60.5-61.5' (2:20), 61.5-62.5' (2:30), 62.5-63.5' (2:00)	Bottom of Exploration at 63.20 feet below ground surface.									R3	42/42	57.0 - 60.5	RQD = 10%					60	R4	32/32	60.5 - 63.2	RQD = 66%					173.1																				65																																					70																																					75																																				
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.																																																																																																																																																																																																																							
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	R1	18/15	52.0 - 53.5	RQD = 0%				NQ																																																																																																																																																																																																																											
55	R2	42/42	53.5 - 57.0	RQD = 52%							Increased resistance during roller cone advancement from 51.3' to 52.0', and set up to core. R1: Hard, fresh, fine grained, grey, SCHIST. Joints are very close, low angle, planar, smooth to rough, fresh, open. One high angle to vertical joint. Recovery = 83% -SANGERVILLE FORMATION- Rock Mass Quality = Very Poor Rock Core Times (min:sec): 52.0-53.0' (4:30), 53.0-53.5' (2:05) R2: Hard, fresh, fine grained, grey, SCHIST. Joints are close to very close, moderately dipping to high angle, planar, rough, tight to partially open, fresh. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 53.5-54.0' (1:15), 54.0-55.0' (3:00), 55.0-56.0' (2:15), 56.0-57.0' (2:00) R3: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to close, low angle to moderately dipping, planar, rough to smooth, open. Extremely fractured resulting in angular gravel pieces from 57.8'-59.9'. Recovery = 100% Rock Mass Quality = Very Poor -SANGERVILLE FORMATION- Rock Core Times (min:sec): 57.0-58.0' (2:35), 58.0-59.0' (2:30), 59.0-60.0' (2:50), 60.0-60.5' (1:35) R4: Hard, fresh, fine grained, grey, SCHIST. Joints are very close to moderately spaced, moderately dipping to high angle, planar, rough to smooth, tight to open, fresh. One high angle joint. Recovery = 100% Rock Mass Quality = Fair -SANGERVILLE FORMATION- Rock Core Times (min:sec): 60.5-61.5' (2:20), 61.5-62.5' (2:30), 62.5-63.5' (2:00)	Bottom of Exploration at 63.20 feet below ground surface.																																																																																																																																																																																																																							
	R3	42/42	57.0 - 60.5	RQD = 10%																																																																																																																																																																																																																															
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<div>Remarks:</div> <div>1. Automatic Hammer NEBC #D19 Energy Transfer Ratio = 0.713.</div> <div>2. Water levels were measured on 10/30/18 prior to commencing drilling activities with casing still in the exploration.</div> <div>3. The as-drilled locations were surveyed by MaineDOT.</div> <div>4. 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.</div>																																																																																																																																																																																																																																			
<div>Stratification lines represent approximate boundaries between soil types; transitions may be gradual.</div> <div>* 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.</div>											<div>Page 3 of 3</div> <div>Boring No.: BB-CBS-103</div>																																																																																																																																																																																																																								



APPENDIX C – ROCK CORE PHOTOGRAPHS



Hall Bridge No. 2848 over Black Stream
MaineDOT WIN 22226.00, Canaan, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CBS-101	R1	42.0 - 42.2	1	50	0	0	SCHIST	1
BB-CBS-101	R2	42.2 - 45.4	35	92	24	63	SCHIST	1
BB-CBS-101	R3	45.4 - 49.1	42	95	7	16	SCHIST	1,2
BB-CBS-101	R4	49.1 - 53.1	48	100	33	69	SCHIST	2,3



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
2. Top photo is dry, bottom photo is wet.



Hall Bridge No. 2848 over Black Stream
MaineDOT WIN 22226.00, Canaan, ME
 Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CBS-102	R1	30.0 - 33.7	39	89	13	30	SCHIST	1
BB-CBS-102	R2	33.7 - 38.7	60	100	36	60	SCHIST	1,2
BB-CBS-102	R3	38.7 - 41.0	29	96	19	68	SCHIST	3



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
 2. Top photo is dry, bottom photo is wet.



Hall Bridge No. 2848 over Black Stream
MaineDOT WIN 22226.00, Canaan, ME
 Rock Core Photographs


Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-CBS-103	R1	52.0 - 53.5	18	83	0	0	SCHIST	1
BB-CBS-103	R2	53.5 - 57.0	42	100	22	52	SCHIST	1
BB-CBS-103	R3	57.0 - 60.5	42	100	4	10	SCHIST	2
BB-CBS-103	R4	60.5 - 63.2	32	100	21	66	SCHIST	3



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



APPENDIX D – LABORATORY TEST RESULTS


	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com <i>Let's Build a Solid Foundation</i>	Client Information: GZA GeoEnvironmental Portland, ME PM: Nick Williams Assigned By: NVW Collected By: NVW	Project Information: Hall Bridge Replacement #3159, WIN 22226.00 Canaan, ME GZA Project Number: 09.0026000.00 Summary Page: 1 of 2 Report Date: 11.26.18
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LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests						Corrosivity Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	Sulfate (mg/kg)	Chloride (mg/kg)	Sulfide (mg/kg)	Redox Potential (mv)	Electrical Resist. As Received Ohm-cm	Electrial Resist. Saturated Ohm-cm	pH	
				D2216	D4318		D6913			D2874	EPA				G57		G51	
101	1D	0.4-2.4	S-1	3.8			24.8	61.7	13.5									Brown f-c SAND, some f-c Gravel, little Silt
101	2D	2-4	S-2	2.0			45.9	49.5	4.6									Brown Gravelly fine to coarse SAND, trace Silt
101	4D	6-8	S-3	7.3														Water Content Only
101	6D	10-12 (Upper 6")	S-4	83.5						12.2								Organic Content Only
101	7D	15-17	S-5	26.8			0.7	21.4	77.9									Brown CLAYEY SILT, some fine Sand, trace fine Gravel
102	3D	10-12 (Upper 12")	S-6	25.2	25	19				1.3								Grey Clayey SILT
103	1D	1-3	S-7	2.5			46.2	45.0	8.8									Brown Sandy fine to coarse GRAVEL, trace Silt
103	2D	3-5	S-8	3.0														Water Content Only
103	3D	5-7	S-9	4.2			28.4	64.8	6.8									Brown f-m SAND, some f-c Gravel, trace Silt
103	4D	7-9 (Upper 7)	S-10	4.5														Water Content Only
103	5D	9-11	S-11	19.8			0.0	13.8	86.2									Grey Clayey SILT, little fine Sand

Reviewed By SKW

11.26.2018

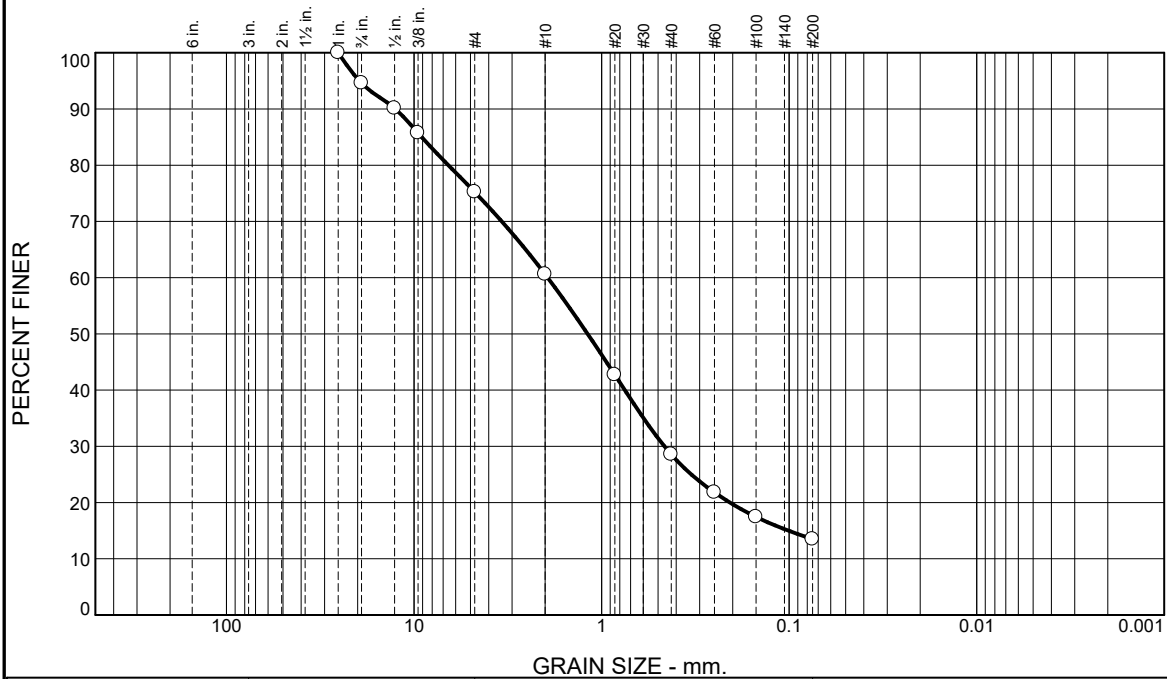
	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com <i>Let's Build a Solid Foundation</i>	Client Information: GZA GeoEnvironmental Portland, ME PM: Nick Williams Assigned By: NVW Collected By: NVW	Project Information: Hall Bridge Replacement #3159, WIN 22226.00 Canaan, ME GZA Project Number: 09.0026000.00 Summary Page: 2 of 2 Report Date: 11.21.18

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests						Corrosivity Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	Sulfate (mg/kg)	Chloride (mg/kg)	Sulfide (mg/kg)	Redox Potential (mv)	Electrical Resist. As Received Ohm-cm	Electrial Resist. Saturated Ohm-cm	pH	
				D2216	D4318		D6913			D2874	EPA				G57		G51	
103	7D	20-22	S-12	28.0			2.7	22.9	74.4									Grey SILT, some fine Sand, trace fine Gravel
103	8D	25-27	S-13				0.0	39.5	60.5									Grey Sandy SILT
103	10D	34.5-36.5	S-14				0.6	55.5	43.9									Brown Silty fine to medium SAND, trace fine Gravel

Reviewed By  _____

11.26.2018 _____

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.4	19.4	14.6	32.0	15.1	13.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	94.6		
0.5"	90.1		
0.375"	85.7		
#4	75.2		
#10	60.6		
#20	42.7		
#40	28.6		
#60	21.8		
#100	17.4		
#200	13.5		

* (no specification provided)

Material Description

Brown 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

D₉₀= 12.5774 D₈₅= 9.1041 D₆₀= 1.9386
D₅₀= 1.1876 D₃₀= 0.4623 D₁₅= 0.1007
D₁₀= C_u= C_c=

Remarks

Date Received: 11.19.18 Date Tested: 11.21.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 101 / 1D

Depth: 0.4-2.4'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

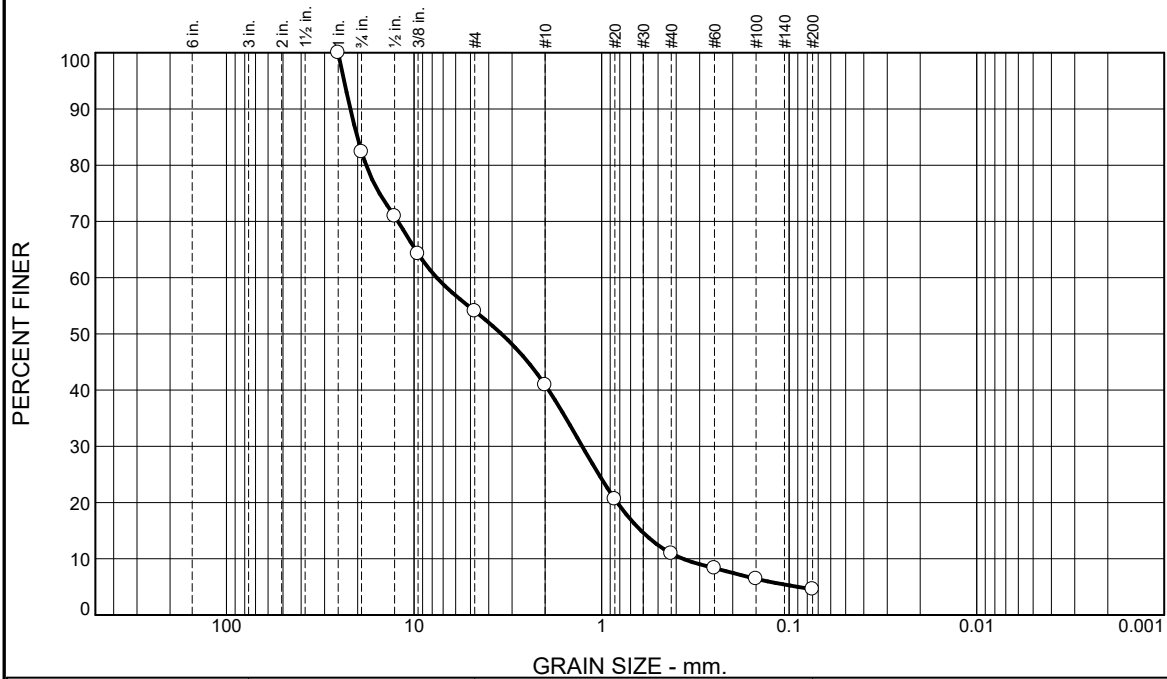
Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.6	28.3	13.2	29.9	6.4	4.6	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	82.4		
0.5"	70.9		
0.375"	64.3		
#4	54.1		
#10	40.9		
#20	20.6		
#40	11.0		
#60	8.3		
#100	6.4		
#200	4.6		

* (no specification provided)

Material Description

Brown Gravelly fine to coarse SAND, trace Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SP AASHTO (M 145)= A-1-a

Coefficients

D₉₀= 21.8544 D₈₅= 20.0648 D₆₀= 7.5605
D₅₀= 3.4218 D₃₀= 1.2680 D₁₅= 0.6159
D₁₀= 0.3682 C_u= 20.53 C_c= 0.58

Remarks

Date Received: 11.19.18 Date Tested: 11.21.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 101 / 2D

Depth: 2-4'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

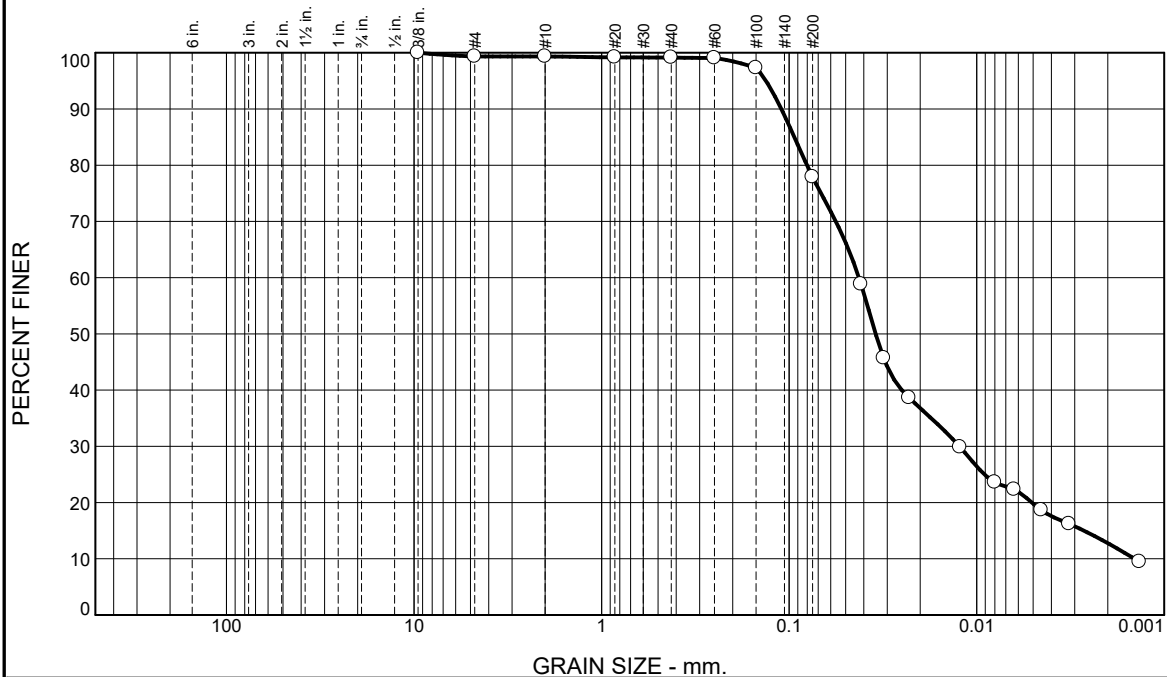
Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.7	0.0	0.2	21.2	65.1	12.8

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	99.3		
#10	99.3		
#20	99.2		
#40	99.1		
#60	99.1		
#100	97.3		
#200	77.9		
0.0414 mm.	58.9		
0.0313 mm.	45.7		
0.0229 mm.	38.6		
0.0123 mm.	29.9		
0.0080 mm.	23.6		
0.0063 mm.	22.3		
0.0045 mm.	18.7		
0.0032 mm.	16.2		
0.0014 mm.	9.5		

* (no specification provided)

Material Description

Brown CLAYEY SILT, some fine Sand, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1101 D₈₅= 0.0939 D₆₀= 0.0425
D₅₀= 0.0346 D₃₀= 0.0124 D₁₅= 0.0027
D₁₀= 0.0014 C_u= 29.52 C_c= 2.50

Remarks

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

Date Received: 11.19.18 Date Tested: 11.26.18

Tested By: RR / JD

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 101 / 7D

Depth: 15-17'

Date Sampled:

Thielsch Engineering Inc.

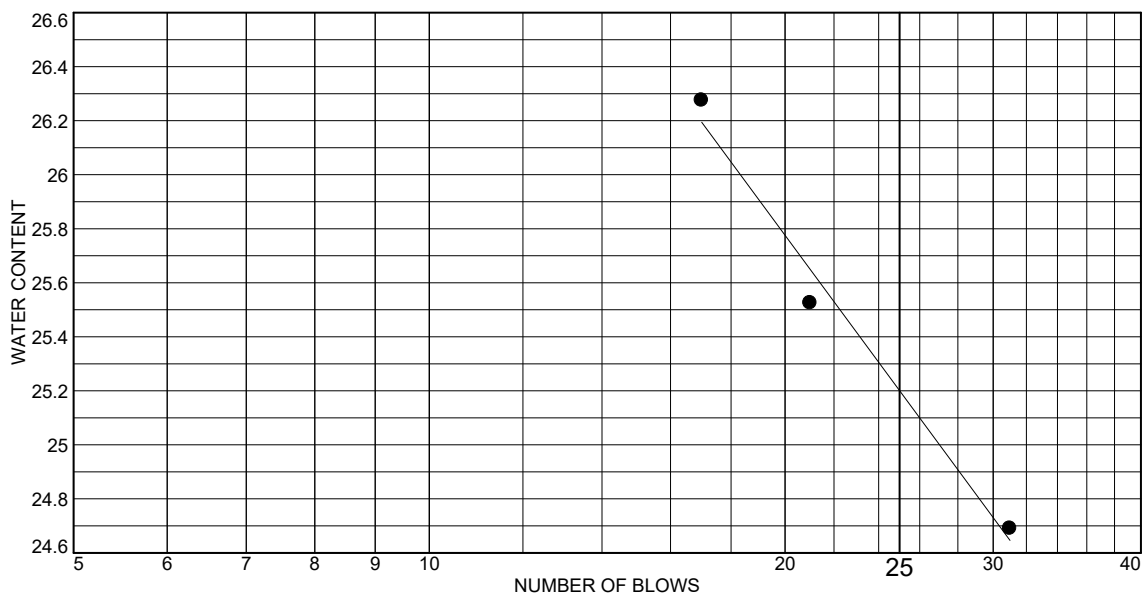
Cranston, RI

Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-5



Tested By: RR **Checked By:** sa

The graph illustrates the grain size distribution of a soil sample. The y-axis represents the percentage of soil finer than a given grain size, ranging from 0 to 100. The x-axis represents the grain size in millimeters on a logarithmic scale, ranging from 100 mm down to 0.001 mm. The curve shows that approximately 100% of the soil is finer than 6 inches, and about 9% of the soil is finer than 0.075 mm (No. 200 sieve).

Grain Size (mm)	Grain Size (inches)	Sieve Size	Percent Finer (%)
150	6 in.	-	100
75	3 in.	-	100
47.5	2 in.	-	100
25	1 in.	-	100
19	3/4 in.	-	90
14.9	1/2 in.	-	82
11.8	3/8 in.	-	72
4.75	#4	-	54
2.0	#10	-	39
0.85	#20	-	26
0.425	#40	-	19
0.25	#60	-	15
0.15	#100	-	12
0.075	#200	-	9

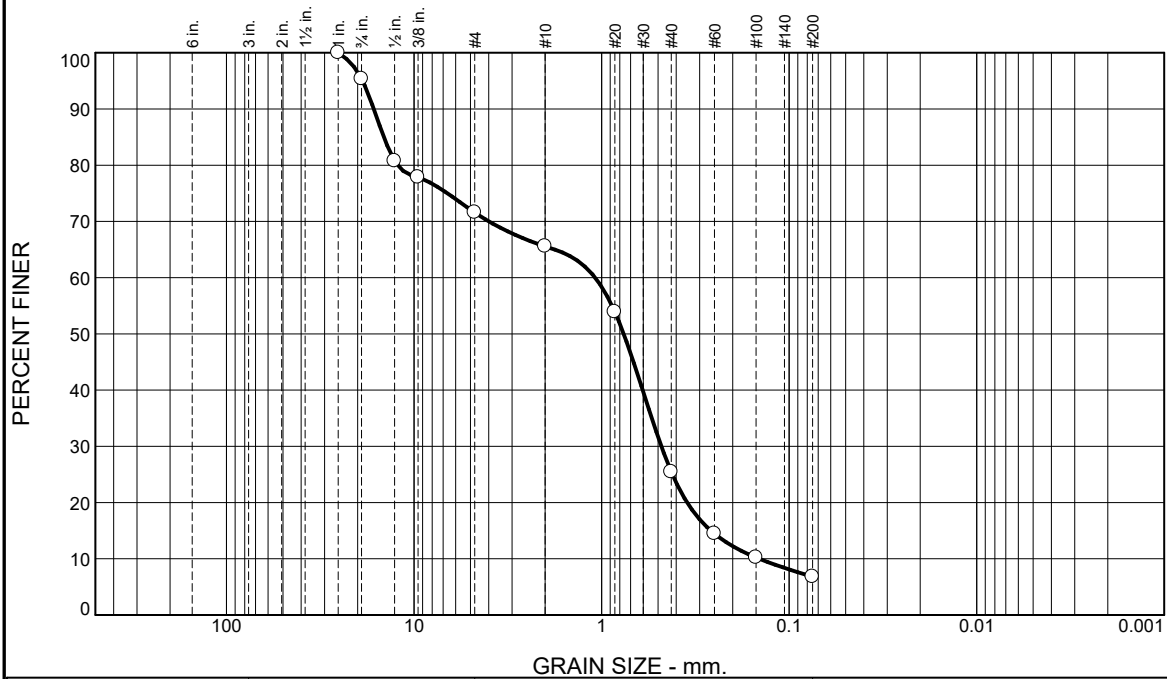
% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.8	35.4	14.9	20.2	9.9	8.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	89.2		
0.5"	81.0		
0.375"	70.8		
#4	53.8		
#10	38.9		
#20	25.6		
#40	18.7		
#60	15.0		
#100	11.9		
#200	8.8		

Title: Laboratory Coordinator

Cranston, RI

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.6	23.8	6.0	40.1	18.7	6.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	95.4		
0.5"	80.8		
0.375"	77.8		
#4	71.6		
#10	65.6		
#20	53.9		
#40	25.5		
#60	14.5		
#100	10.2		
#200	6.8		

* (no specification provided)

Material Description

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

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 16.4072 D₈₅= 14.4907 D₆₀= 1.0825
D₅₀= 0.7628 D₃₀= 0.4799 D₁₅= 0.2611
D₁₀= 0.1442 C_u= 7.50 C_c= 1.47

Remarks

Date Received: 11.19.18 Date Tested: 11.21.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 103 / 3D

Depth: 5-7'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-9

The graph illustrates the relationship between sieve size and the percentage of material finer than that sieve. The x-axis represents sieve size in inches (top) and sieve number (bottom). The y-axis represents the percent finer (0 to 100). A smooth curve is drawn through the data points, showing that as the sieve size decreases, the percent finer also decreases.

Sieve Size (inches)	Sieve Number	Percent Finer (%)
100	-	100
30	-	100
25	-	100
20	-	100
15	-	100
12.5	-	100
10	-	100
7.5	-	100
6	-	100
4.75	#40	100
4.25	#45	100
3.75	#50	100
3.0	#60	100
2.5	#75	98
2.0	#100	95
1.5	#120	85
1.18	#150	72
0.85	#20	62
0.75	#25	51
0.6	#30	35
0.5	#35	28
0.425	#40	24
0.375	#45	20
0.3	#60	15
0.25	#75	10
0.2	#90	8

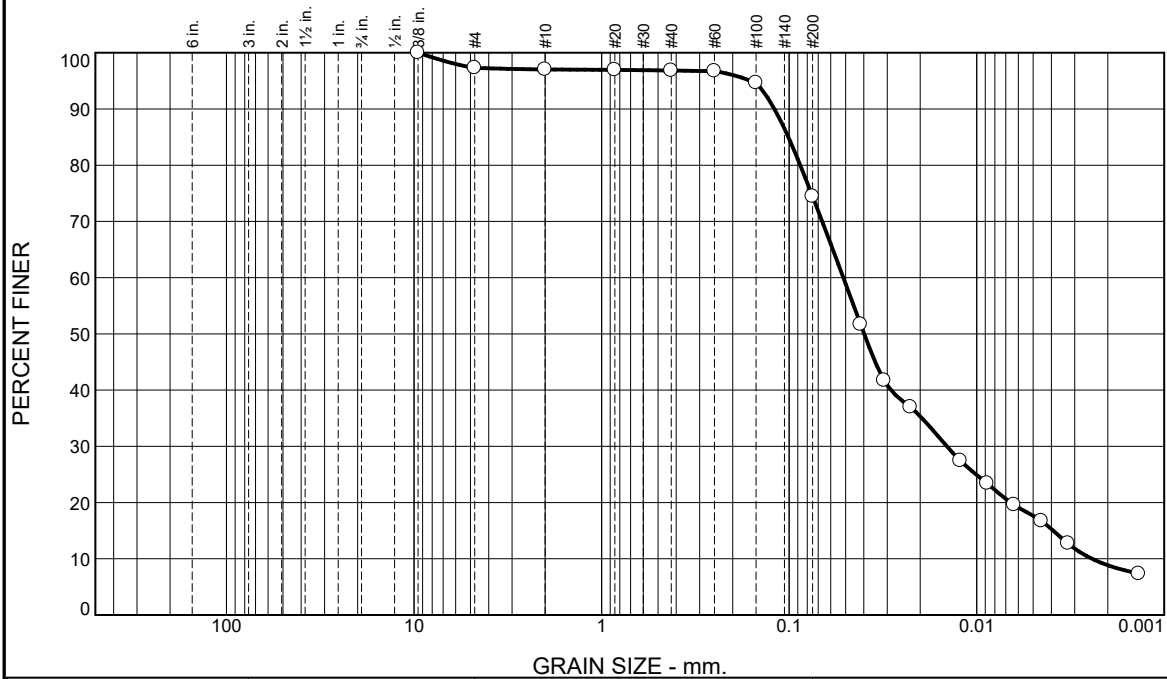
GRAIN SIZE - mm.							
% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	13.2	66.1	20.1

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#10	100.0		
#20	99.9		
#40	99.4		
#60	98.7		
#100	96.1		
#200	86.2		
0.0377 mm.	70.9		
0.0284 mm.	61.2		
0.0214 mm.	50.3		
0.0119 mm.	34.6		
0.0081 mm.	27.4		
0.0063 mm.	23.3		
0.0045 mm.	18.6		
0.0032 mm.	13.8		
0.0014 mm.	8.3		

Title: Laboratory Coordinator

Cranston, RI

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.7	0.3	0.2	22.4	65.6	8.8

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	97.3		
#10	97.0		
#20	97.0		
#40	96.8		
#60	96.7		
#100	94.6		
#200	74.4		
0.0416 mm.	51.7		
0.0311 mm.	41.7		
0.0226 mm.	37.0		
0.0122 mm.	27.5		
0.0088 mm.	23.4		
0.0063 mm.	19.6		
0.0045 mm.	16.7		
0.0033 mm.	12.7		
0.0014 mm.	7.3		

* (no specification provided)

Material Description

Grey SILT, some fine Sand, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1197 D₈₅= 0.1010 D₆₀= 0.0514
D₅₀= 0.0398 D₃₀= 0.0144 D₁₅= 0.0039
D₁₀= 0.0024 C_u= 21.18 C_c= 1.67

Remarks

Sample visually classified as non-plastic.

Date Received: 11.19.18 Date Tested: 11.26.18

Tested By: RR / JD

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 103 / 7D

Depth: 20-22'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

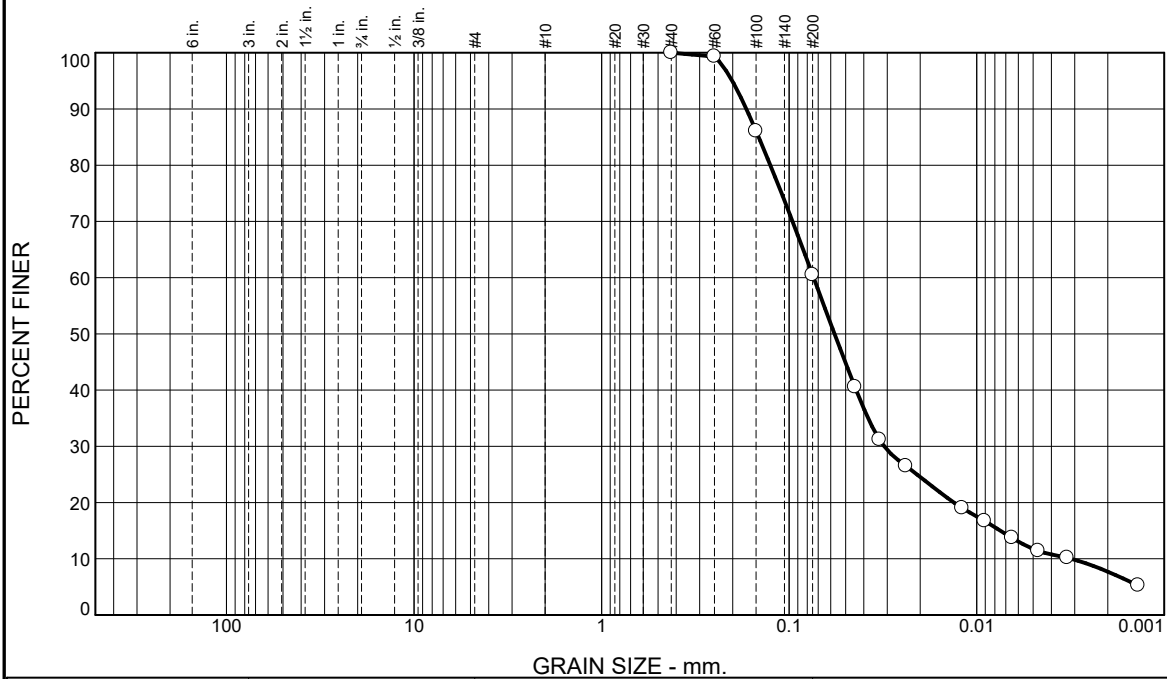
Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-12

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.0	39.5	52.8	7.7

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#40	100.0		
#60	99.3		
#100	86.1		
#200	60.5		
0.0446 mm.	40.6		
0.0330 mm.	31.2		
0.0238 mm.	26.5		
0.0119 mm.	19.0		
0.0091 mm.	16.7		
0.0065 mm.	13.7		
0.0047 mm.	11.4		
0.0033 mm.	10.2		
0.0014 mm.	5.3		

* (no specification provided)

Material Description

Grey Sandy SILT

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.1690 D₈₅= 0.1453 D₆₀= 0.0740
D₅₀= 0.0573 D₃₀= 0.0311 D₁₅= 0.0075
D₁₀= 0.0031 C_u= 23.57 C_c= 4.17

Remarks

Sample visually classified as non-plastic.

Date Received: 11.19.18 Date Tested: 11.26.18

Tested By: RR / JD

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 103 / 8D

Depth: 25-27'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

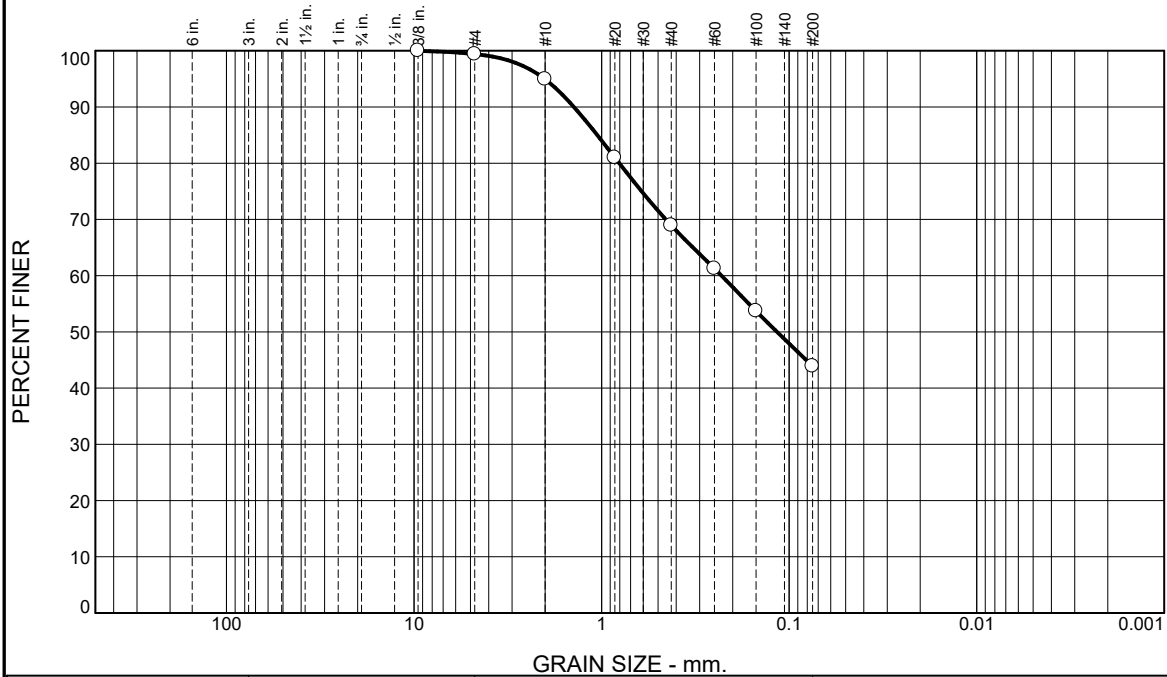
Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-13

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	4.5	26.0	25.0	43.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	99.4		
#10	94.9		
#20	81.0		
#40	68.9		
#60	61.3		
#100	53.7		
#200	43.9		

* (no specification provided)

Material Description

Brown Silty fine to medium SAND, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

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

Coefficients

D₉₀= 1.4033 D₈₅= 1.0545 D₆₀= 0.2294
D₅₀= 0.1160 D₃₀= C_u=
D₁₀= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/8".

Date Received: 11.19.18 Date Tested: 11.21.18

Tested By: RR

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: BB-CBS
Sample Number: 103 / 10D

Depth: 34.5-36.5'

Date Sampled:

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental

Project: Hall Bridge Replacement #3159, WIN 22226.00
Canaan, ME

Project No: 09.0026000.00

Figure S-14

**R. W. Gillespie & Associates, Inc.**

20 Pomerleau St., Suite 100, Biddeford, ME 04005 207-286-8008
177 Shattuck Way, Suite 1 West, Newington NH 03801 603-427-0244
44 Wood Avenue, Suite I, Mansfield, MA 508-623-0101

LETTER OF TRANSMITTAL

Date: June 26, 2019	Project No.: 0876-017
Attention: Nicholas Williams PE (nicholas.williams@gza.com)	
Re: Laboratory Testing Halls Bridge Shelby Tube Testing Canaan, ME	

GZA Geoenvironmental

477 Congress Street

Portland, ME 04101

We are sending you attached Laboratory Test Results.

Laboratory No. (s)

Test (s) Performed

15327: BB-CBS-102 Tube Sample, U-1, 4'-6', Canaan, ME

Atterburg Limits, Shear Vanes, Moisture Contents, Unit Weight

Remarks:

Copy to:

Laboratory Vane Shear and Moisture Content Test Results

ASTM D4648 Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil

Project: Halls Bridge Shelby Tube Testing
 Client: GZA GeoEnvironmental, Inc.
 Project No.: 0876-014

Location: Canaan, ME
 Date: 11/21/2018
 Test Depth: 4.02 to 5.19

Atterberg Test Results (ASTM D4318)

Sample was taken at a depth of appoximatly 5' from ground suface. Tests indicated that the sample was non-plastic and non viscous. Tube sample contained frequent silt/clay seams interbedded in a free draining sand

Boring/Sample No.		BB-CBS-102			Lab No.	15327	
Test No.	Test Depth (ft)	Vane Size	Max. Torque (Undisturbed) (kg-cm)	Max. Torque (Remolded) (kg-cm)	Undrained Shear Strength (psf)	Undrained Shear Strength (psf)	Moisture Content
1	4.02	S	3	0	125	0	29%
2	4.19	M	10	0	209	0	33%
3	4.35	L	52	8	543	84	34%
4	4.52	L	98	12	1023	125	31%
5	Bad Test	L					30%
6	4.85	L	140	15	1462	157	30%
7	5.02	M	42	8	438	84	28%
8	5.19	M	47	7	491	73	28%
9	Remaining	MC Only					26%
						Average MC	30%

Unit Wiegth Determination	
Length of Sample	1.76'
Diameter of Tube	2.87"
Weight of Wet Soil	9.69lbs.
Unit Weight (30% MC)	94.35 lbs/ft^3

Vane Size	
	(mm)
S	16 x 32
M	20 x 40
L	24.5 x 50.8

Tested By: JRF/AGS

Checked By: JRF



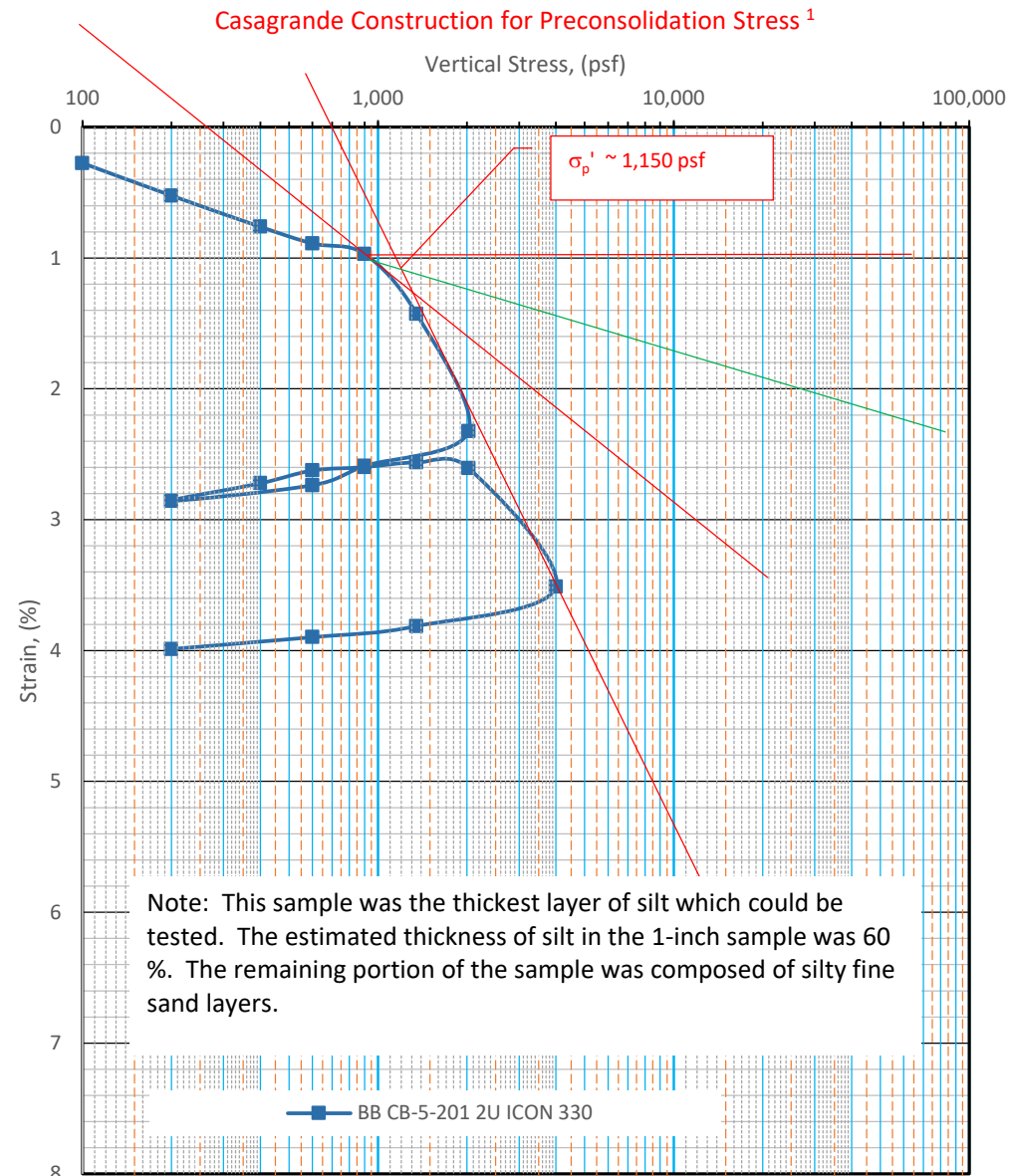
R.W. Gillespie & Associates

JRF

Consolidation Test Data Summary Report

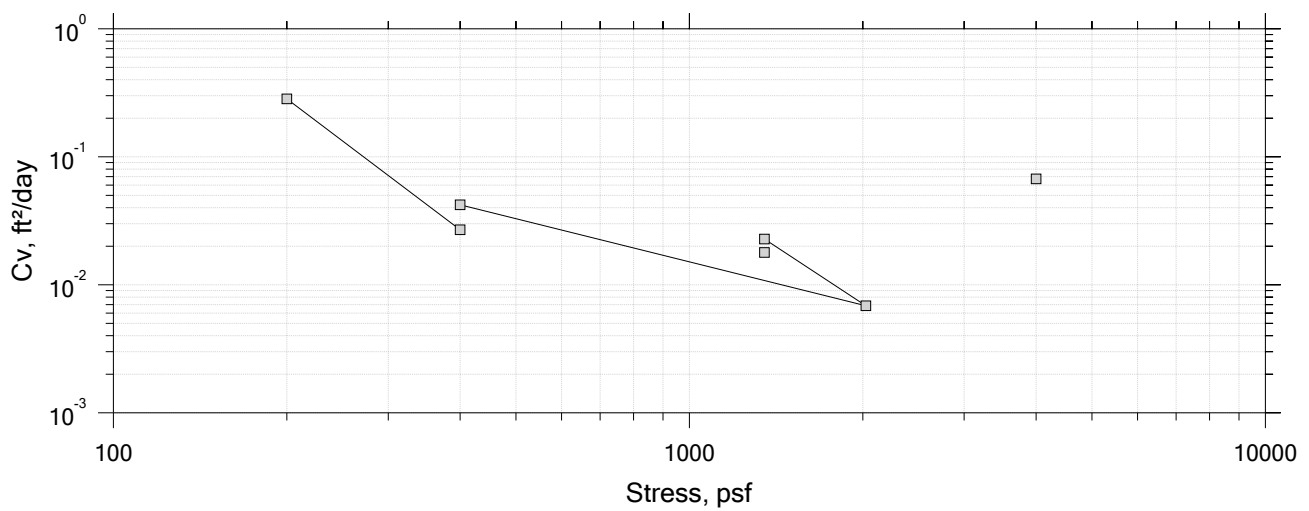
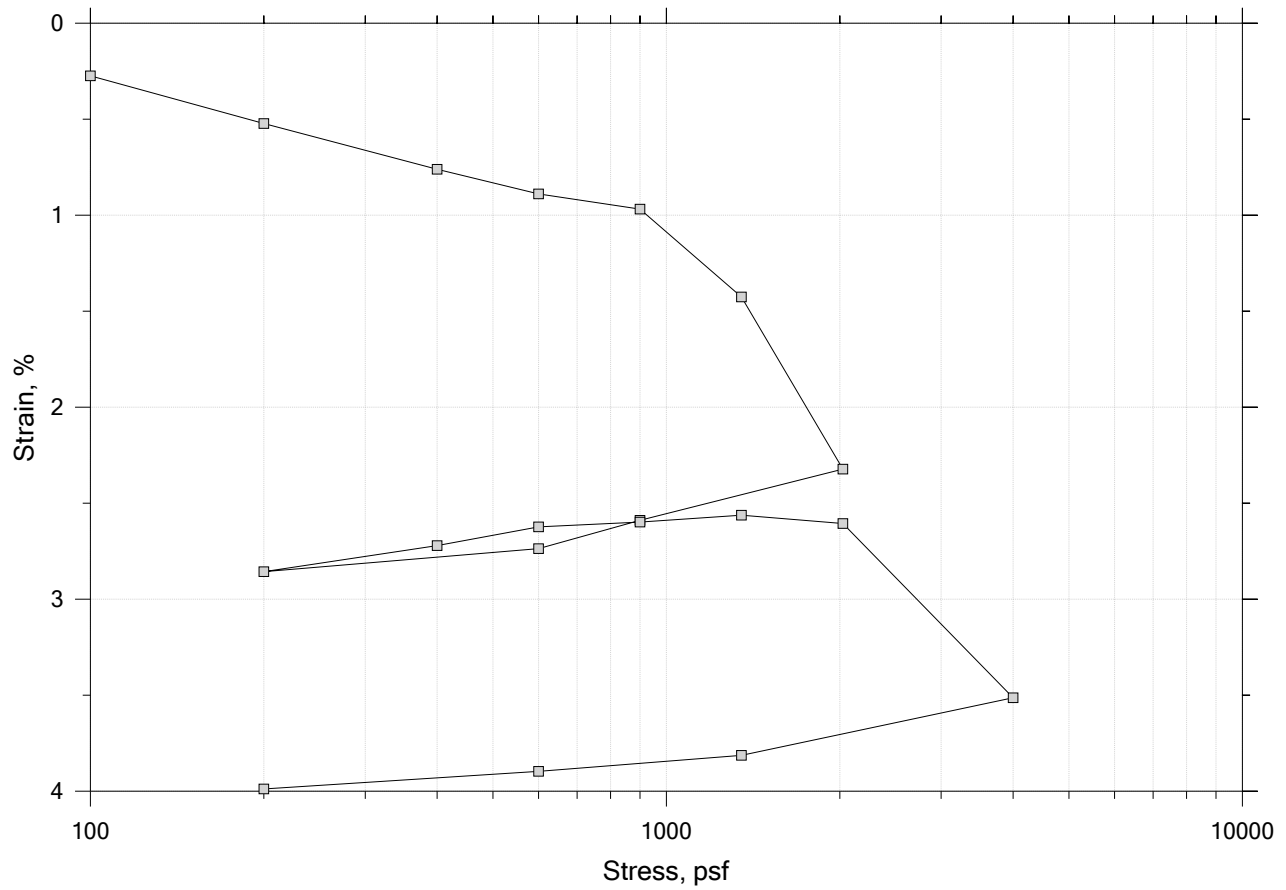
Project Name:		Caanan Bridge 5		
Project Number:		166-14		
Project Location:		Caanan ME		
Client:		GZA		
Sample Description:		Gray Silt with fine sand layers (ML)		
Preparation:		Trimmed Shelby Tube		
Lab Test No:	ICON 330			
Boring No.	BB-CB-5-201			
Sample No:	U2			
Boring Elevation (ft).				
Sample Depth (ft):	11-13			
Test Specimen Depth (Ft):	11.95			
Test Specimen Elevation:				
Water Content (%):	30.48			
Dry Unit Weight (pcf):	92.34			
Wet Unit Weight (pcf):	120.49			
Saturation Before (%):	98.07			
Saturation After (%):	100			
Void Ratio Before:	0.85			
Void Ratio After:	0.78			
Overburden Pressure (psf):				
Max Previous stress (psf):	1,150			
Max Prev. stress (Work) (psf):	1,050			
OCR:				
Compression Index (C_{CE}):	0.042			
Recompression Index (C_{RE}):	0.008			
Liquid Limit:	23.8			
Plastic Limit:	19.1			
Plasticity Index:	4.7			
Liquidity Index:	2.4			
Specific Gravity (implied)	2.74			
Lab Vane Su at 12.25 ft. (psf)	220	The VST was done in a zone that contained many sand layers and may not be representative of the shear strength of the silt layers.		
Tested By:	sjr			
Date Tested:	8/30/2020			
Checked By:	sjr			


Note 1: The calculations for the Max Previous Stress, the Compression Index and the Recompression Index are provided for the convenience of the Specifier. The Specifier should make their own independent assessment of Maximum Previous stress, Cce and Cre for use in any engineering analyses.



One-Dimensional Consolidation by ASTM D2435 - Method B

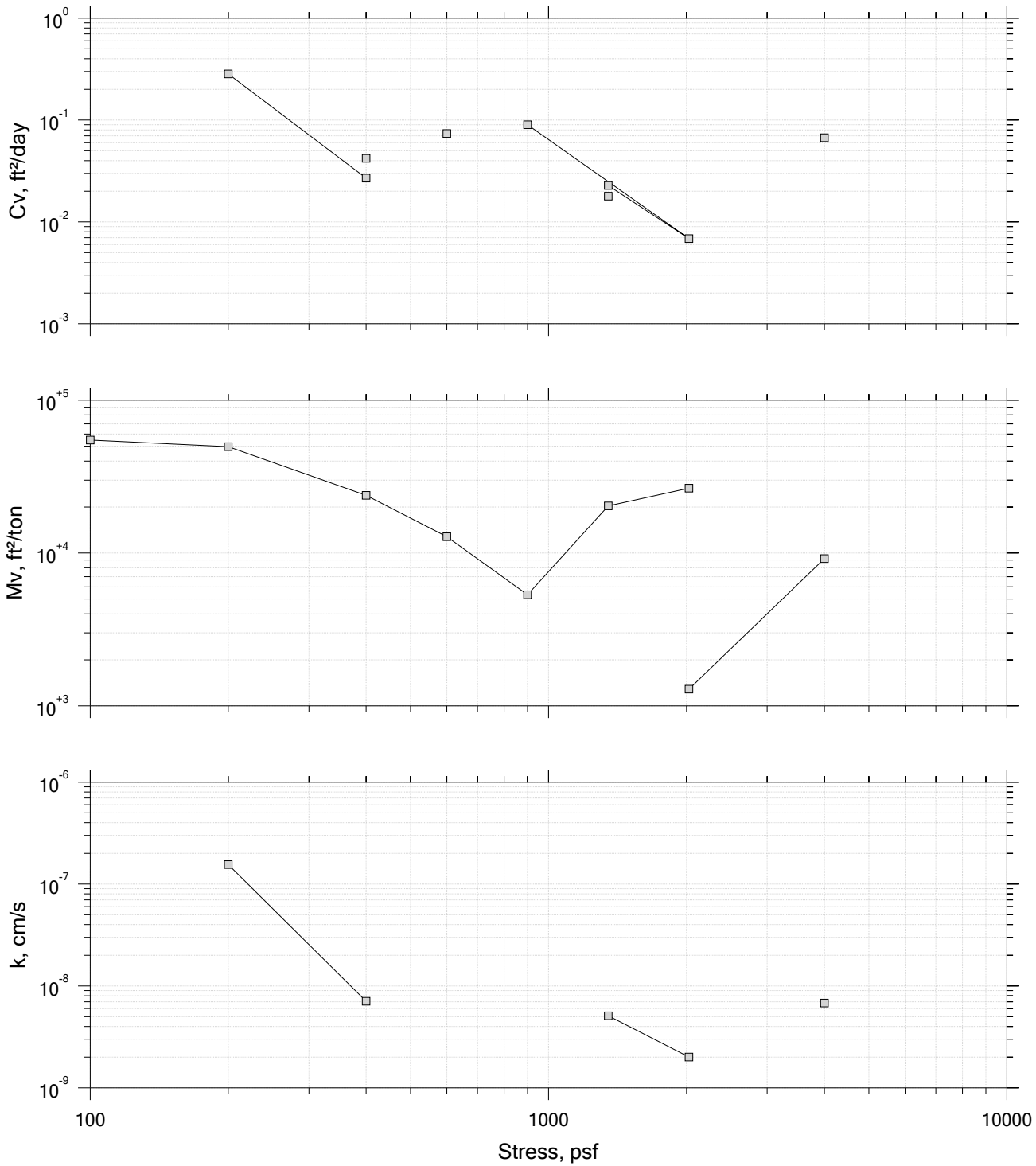
Summary Report




	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		
	Displacement at End of Primary		

One-Dimensional Consolidation by ASTM D2435 - Method B

Sqrt of Time Coefficients



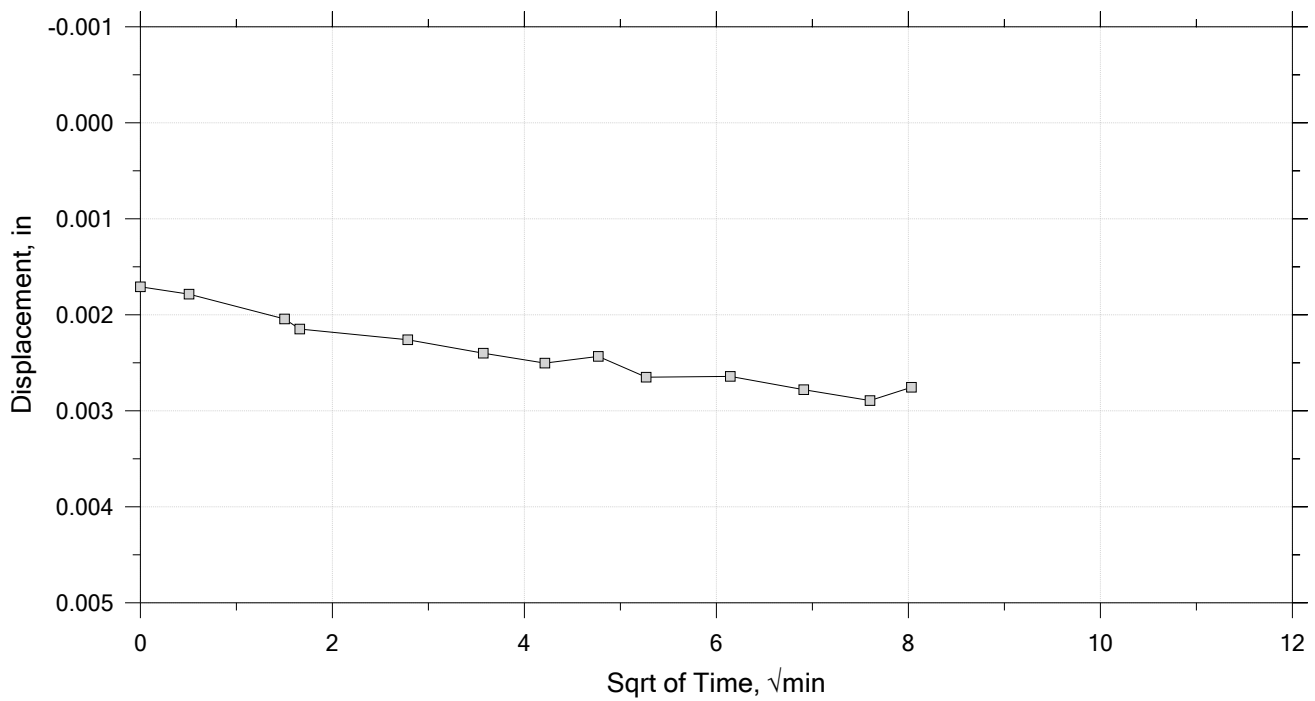
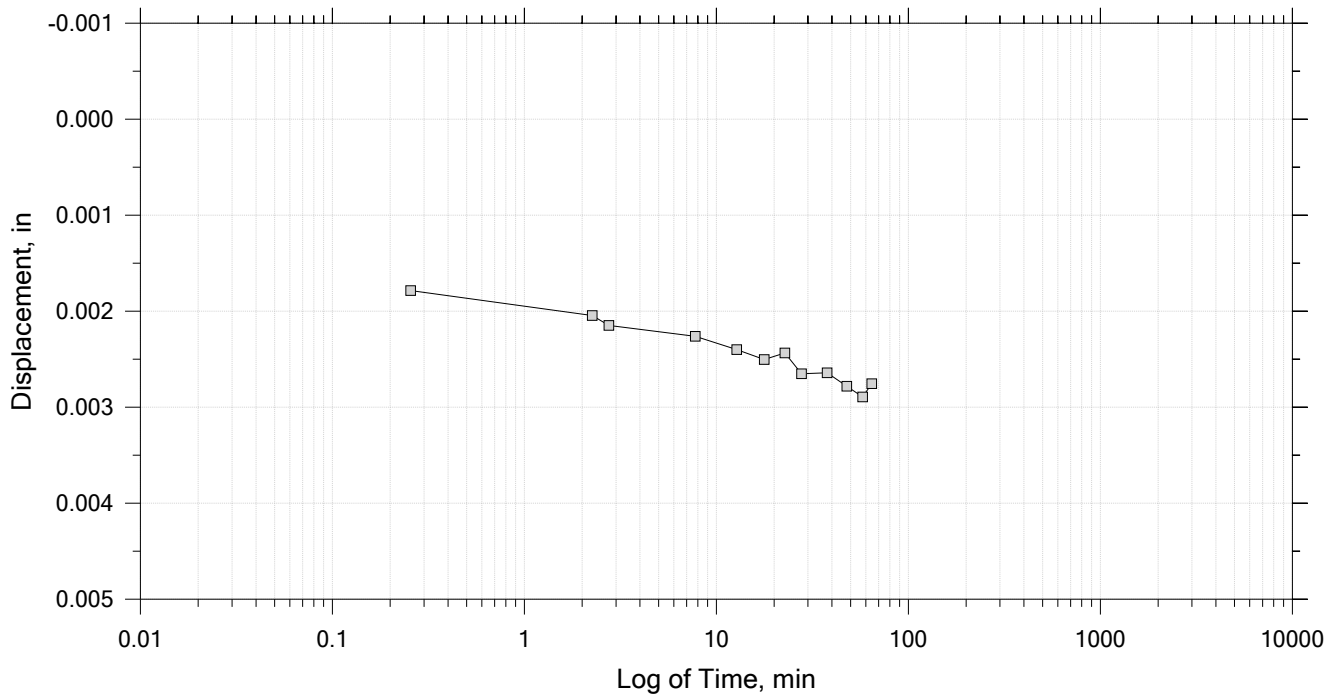
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 19

Constant Load Step

Stress: 100 psf



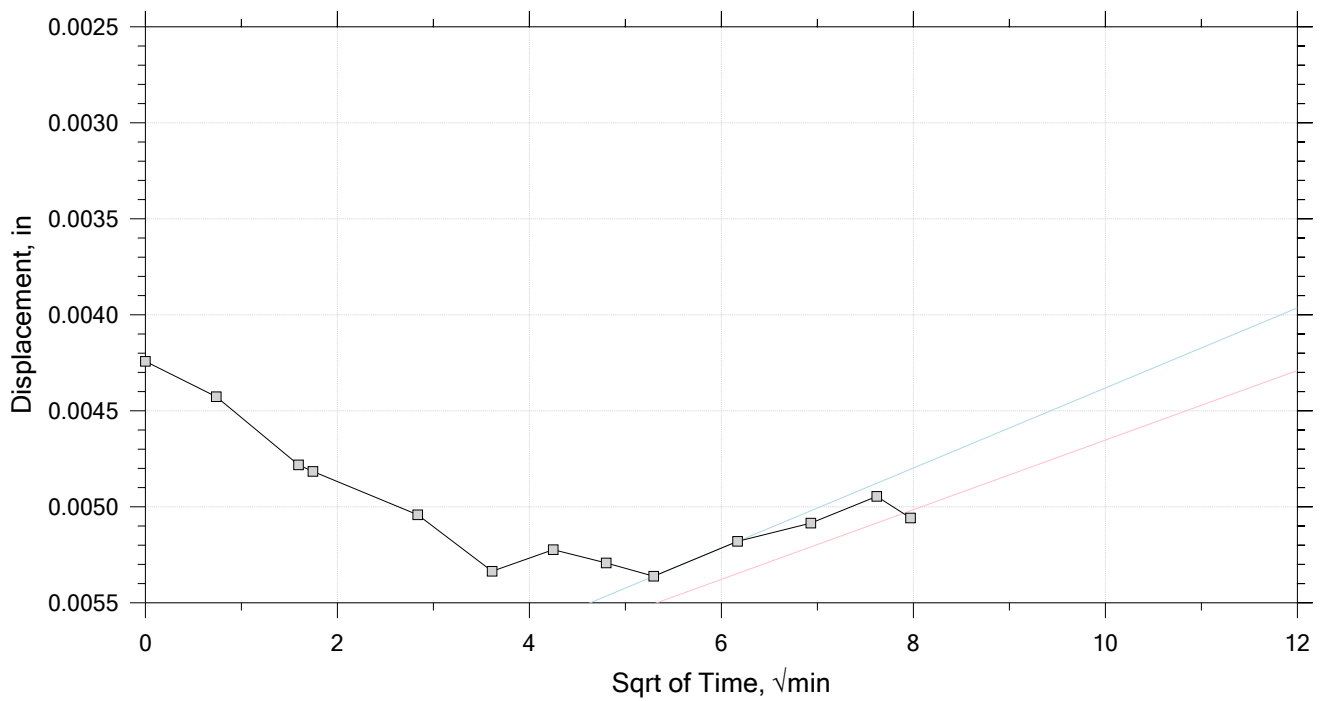
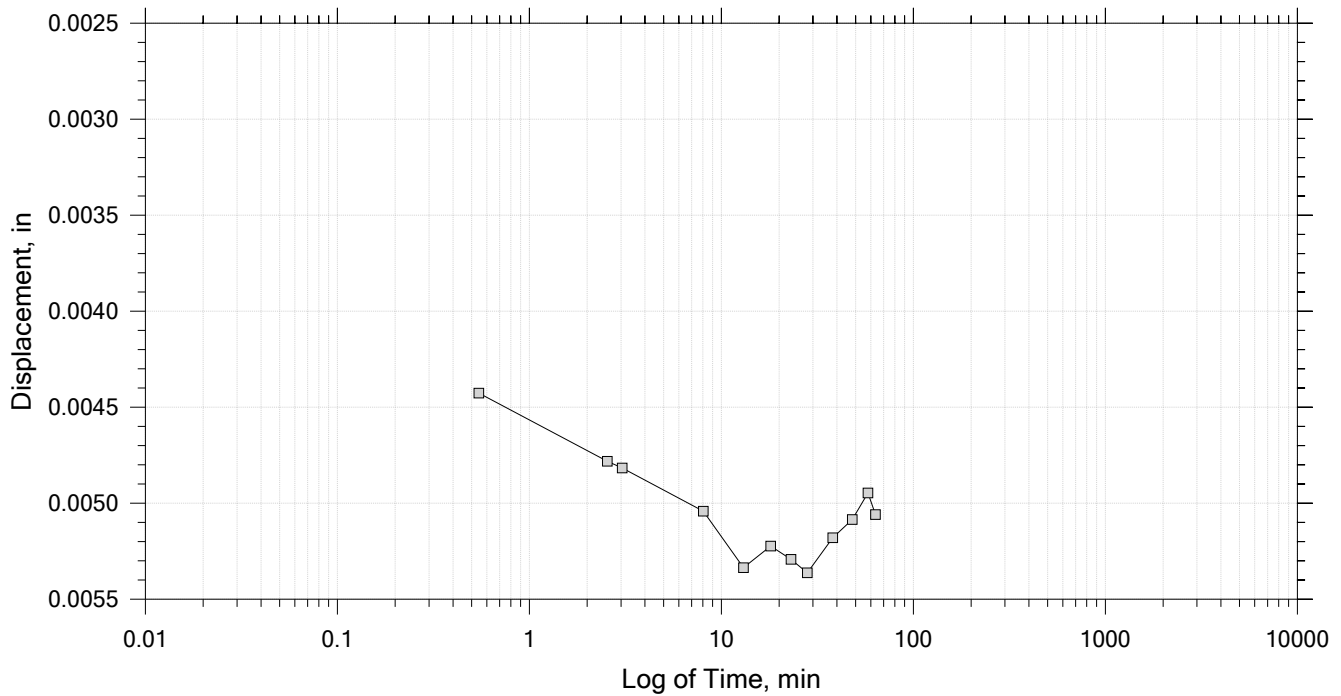
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 19

Constant Load Step

Stress: 200 psf



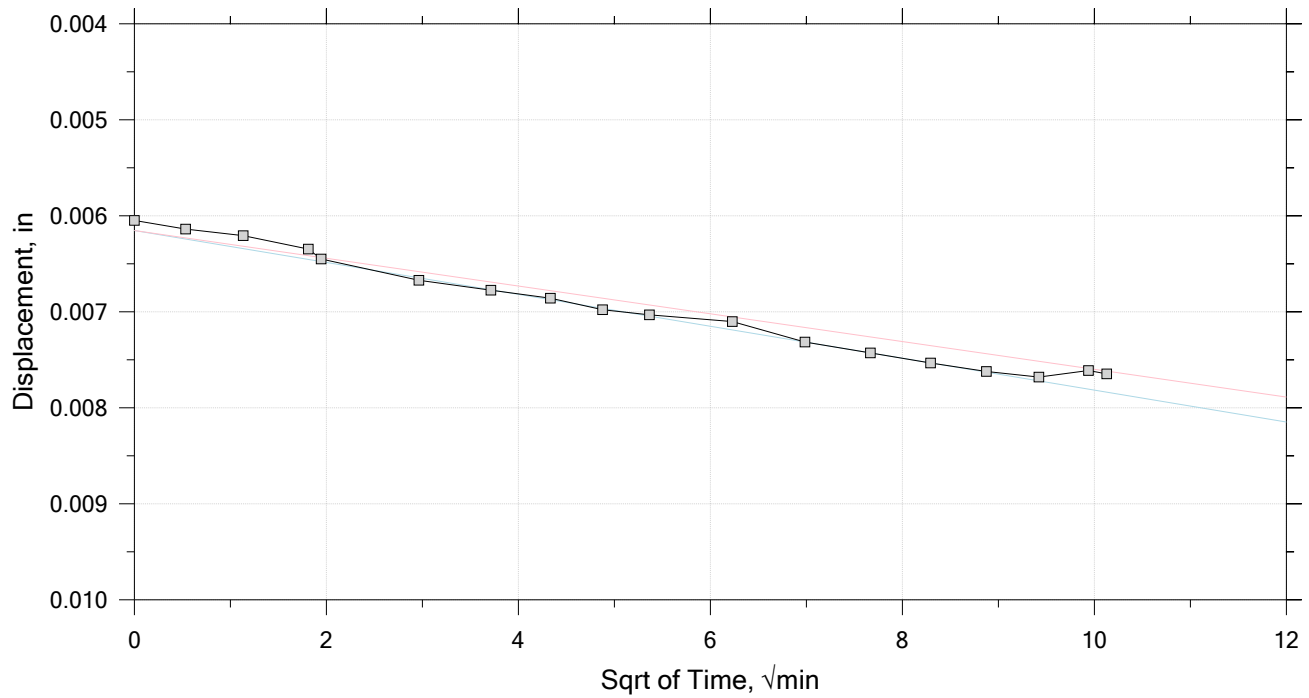
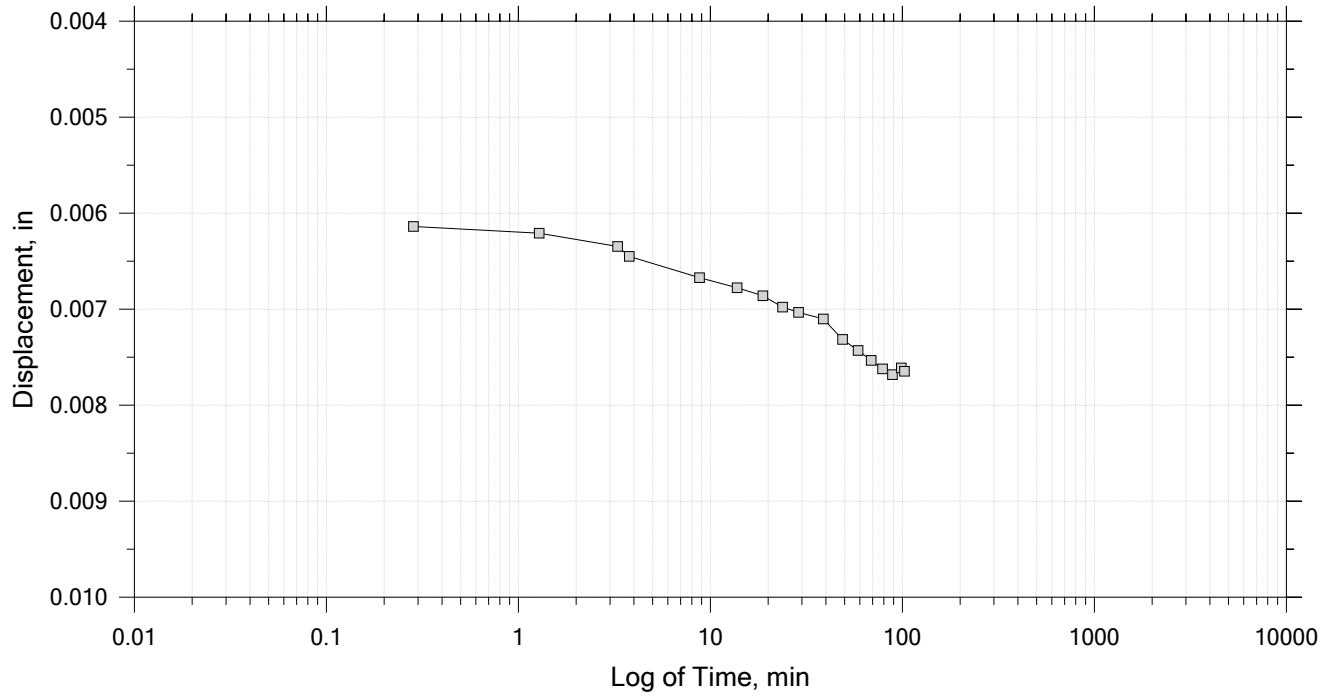
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 19

Constant Load Step

Stress: 400 psf



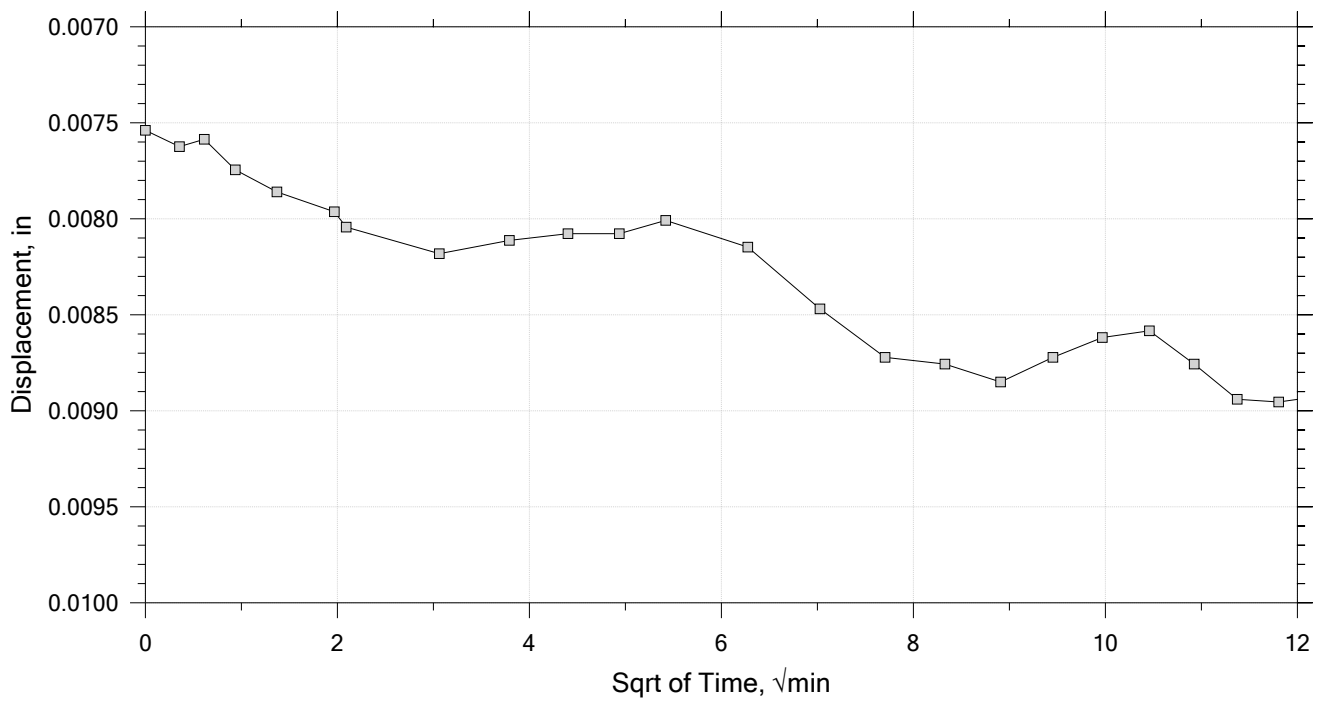
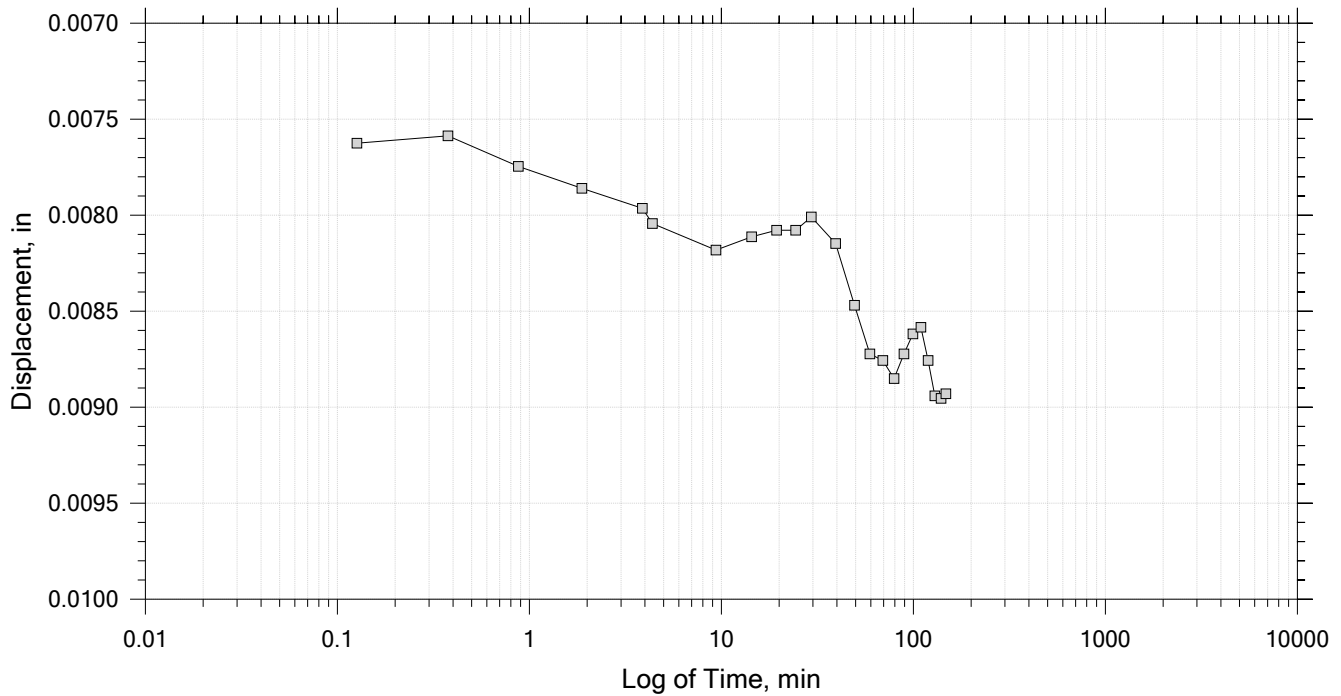
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 19

Constant Load Step

Stress: 600 psf



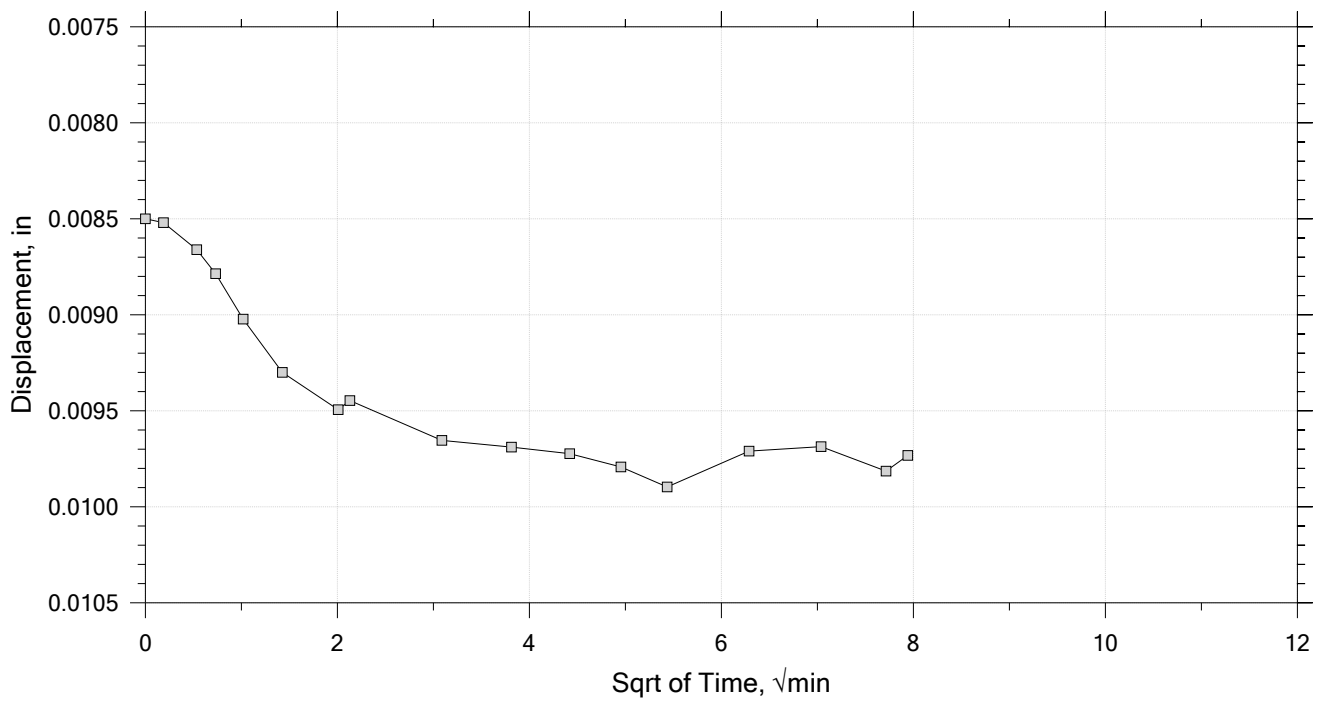
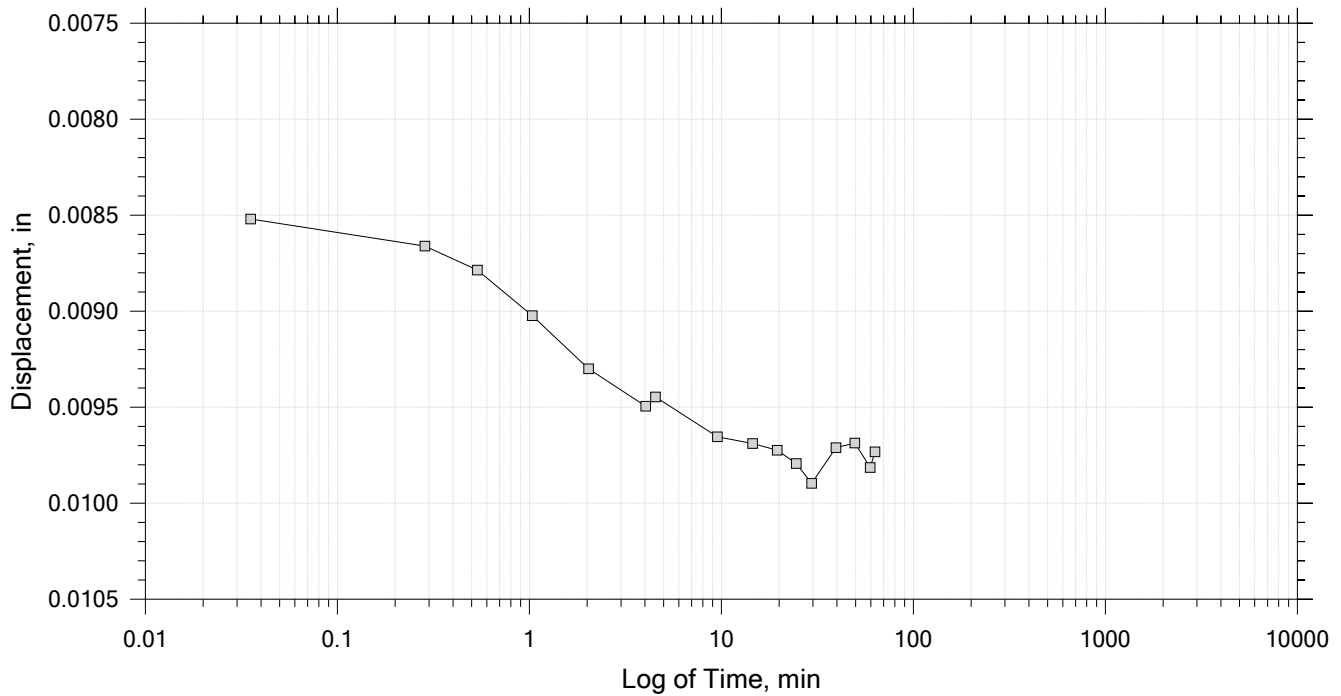
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 19

Constant Load Step

Stress: 900 psf



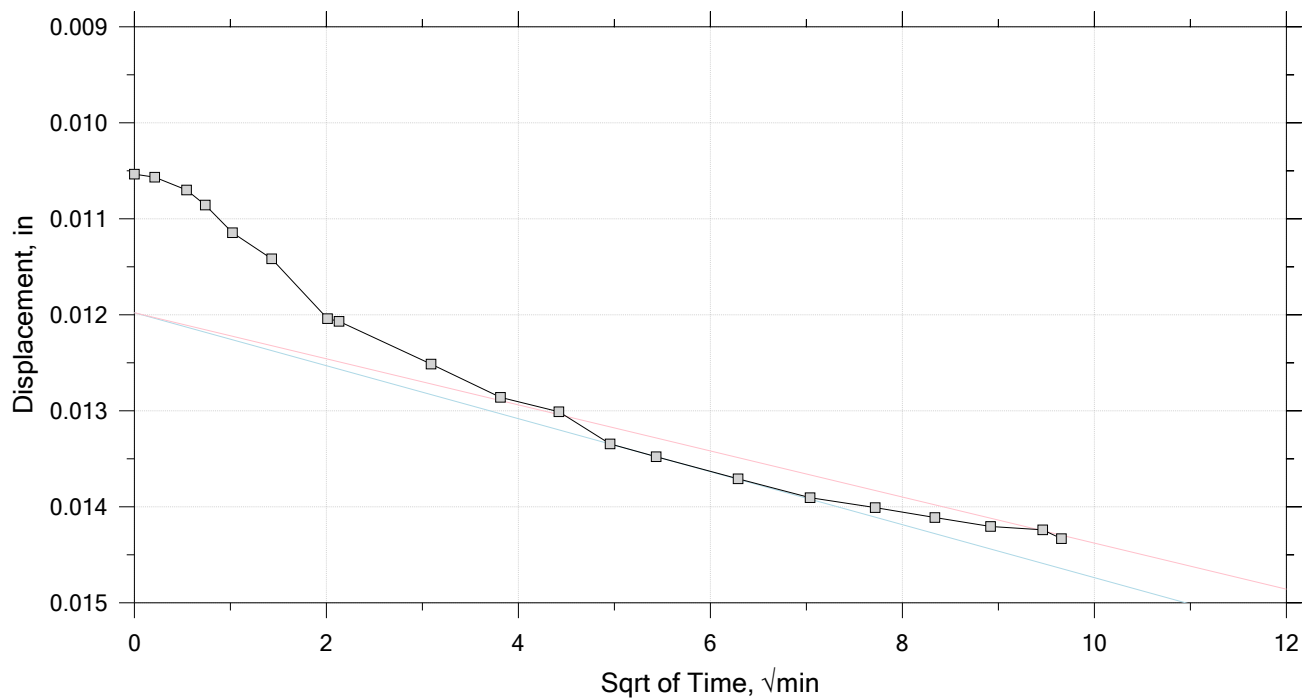
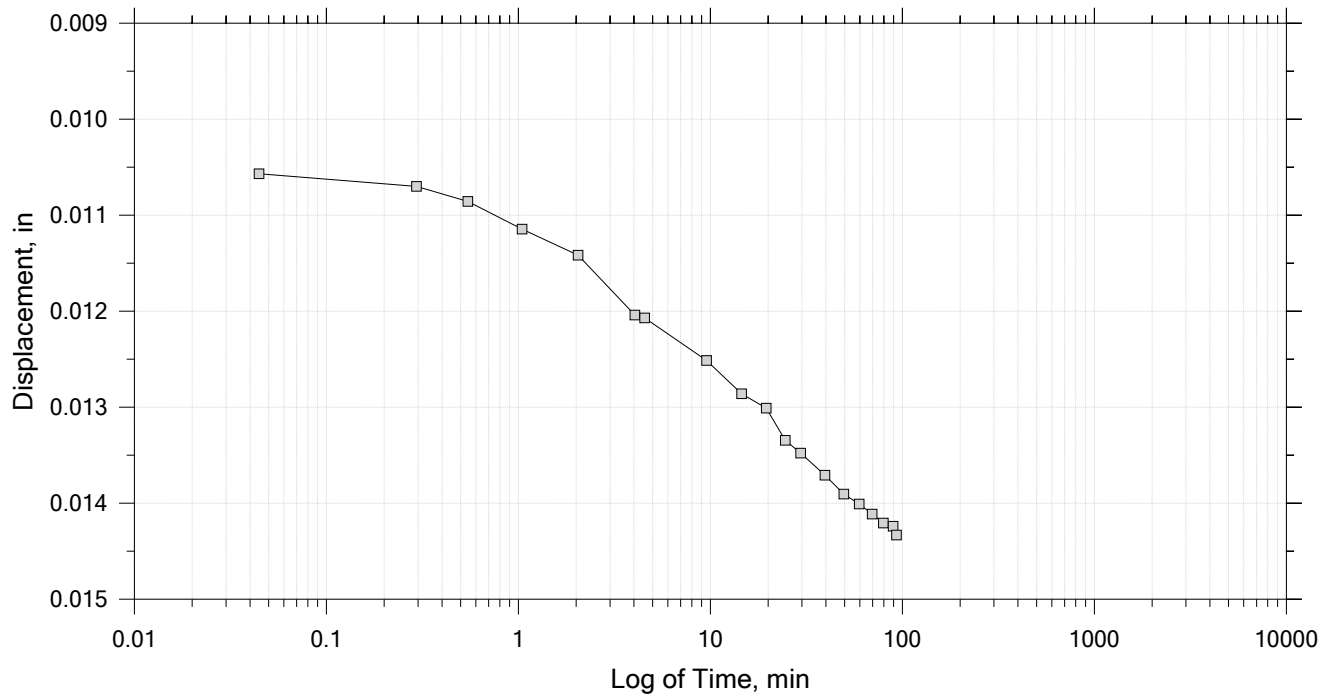
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 19

Constant Load Step

Stress: 1.35e+03 psf



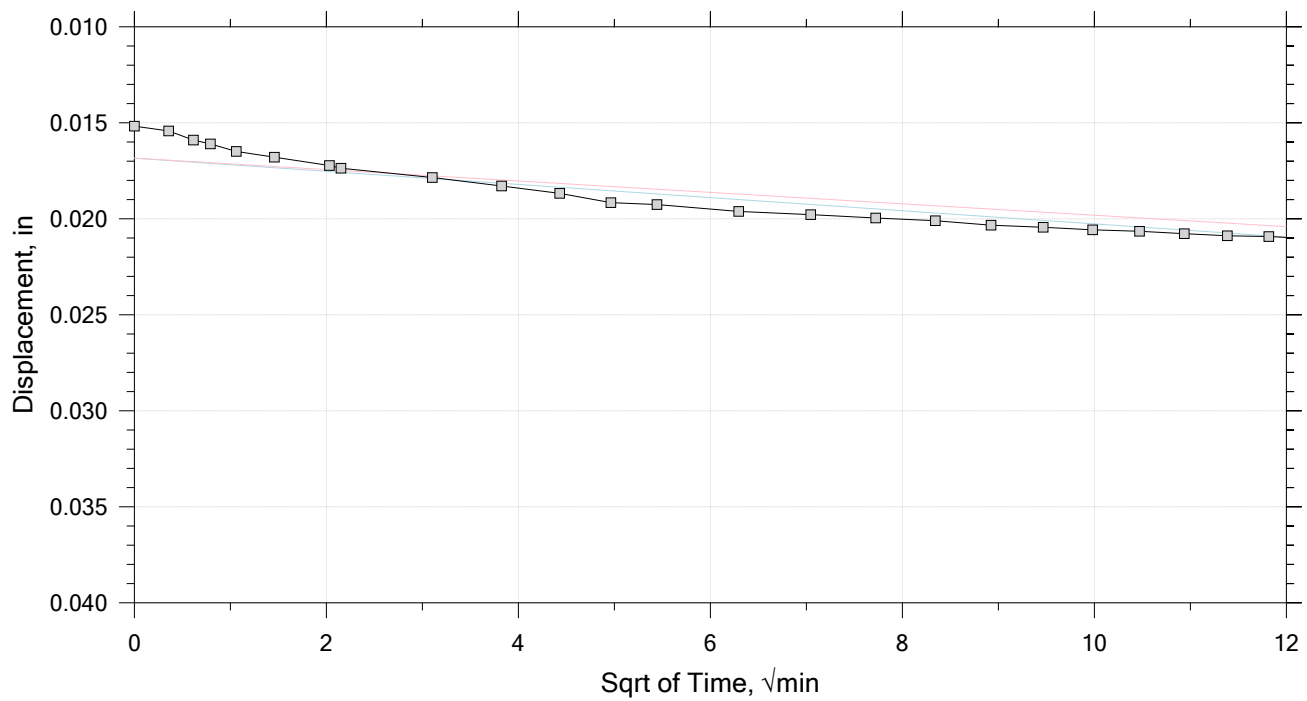
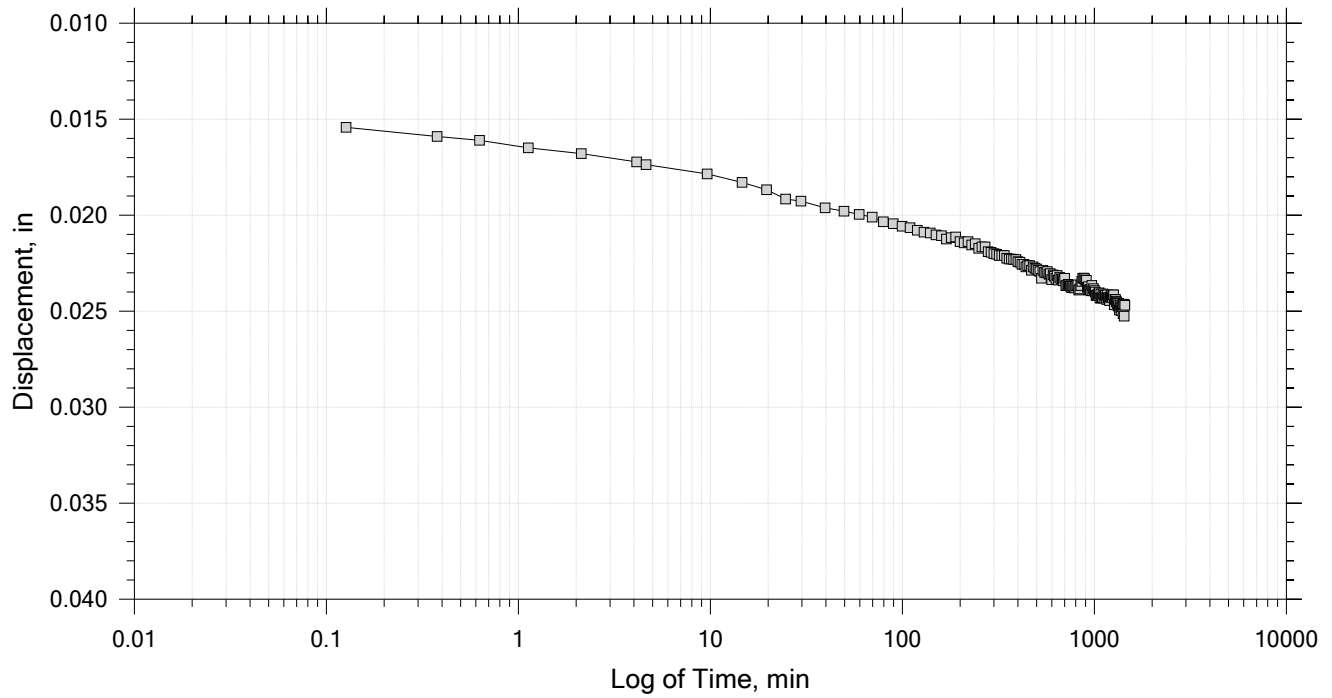
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 19

Constant Load Step

Stress: 2.02e+03 psf



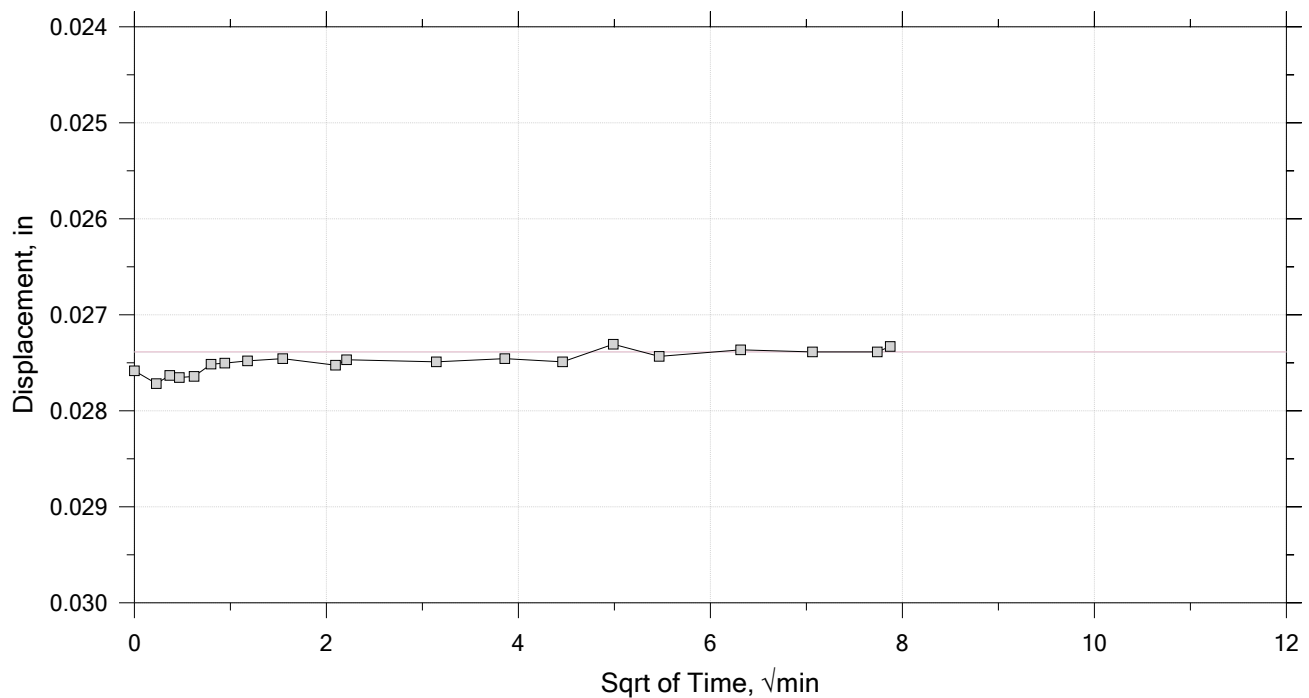
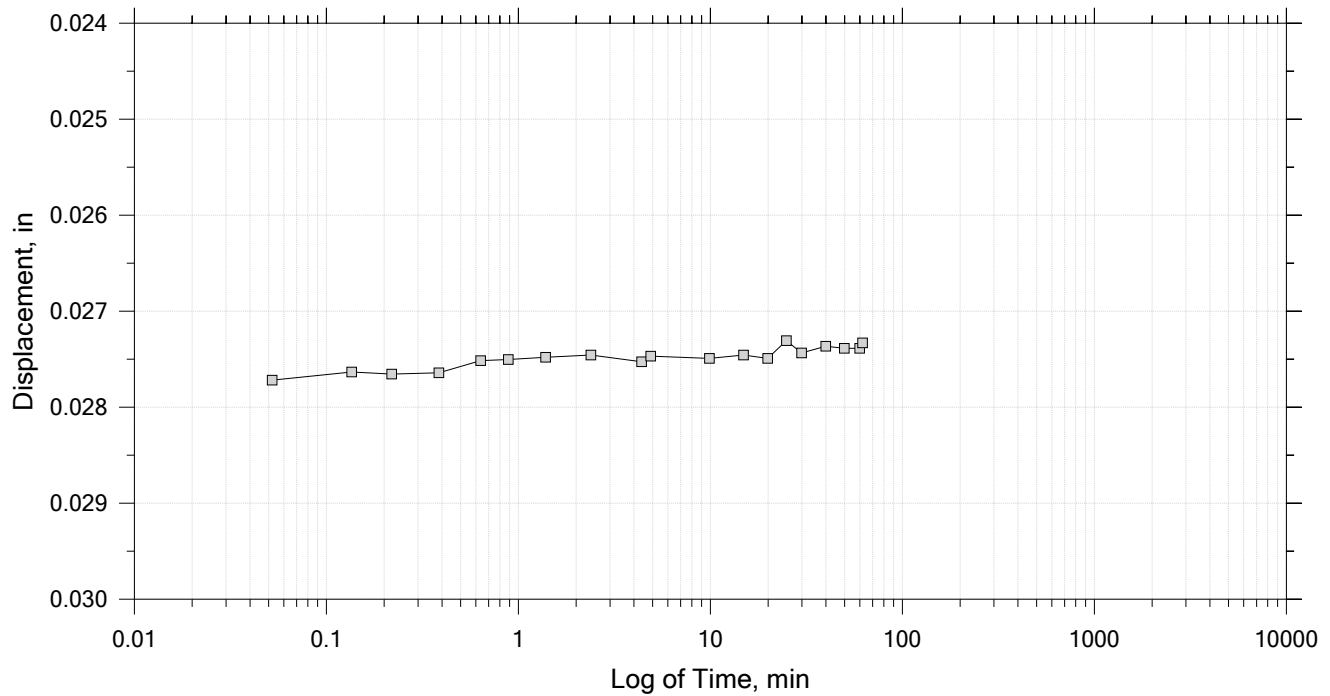
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 19

Constant Load Step

Stress: 900 psf



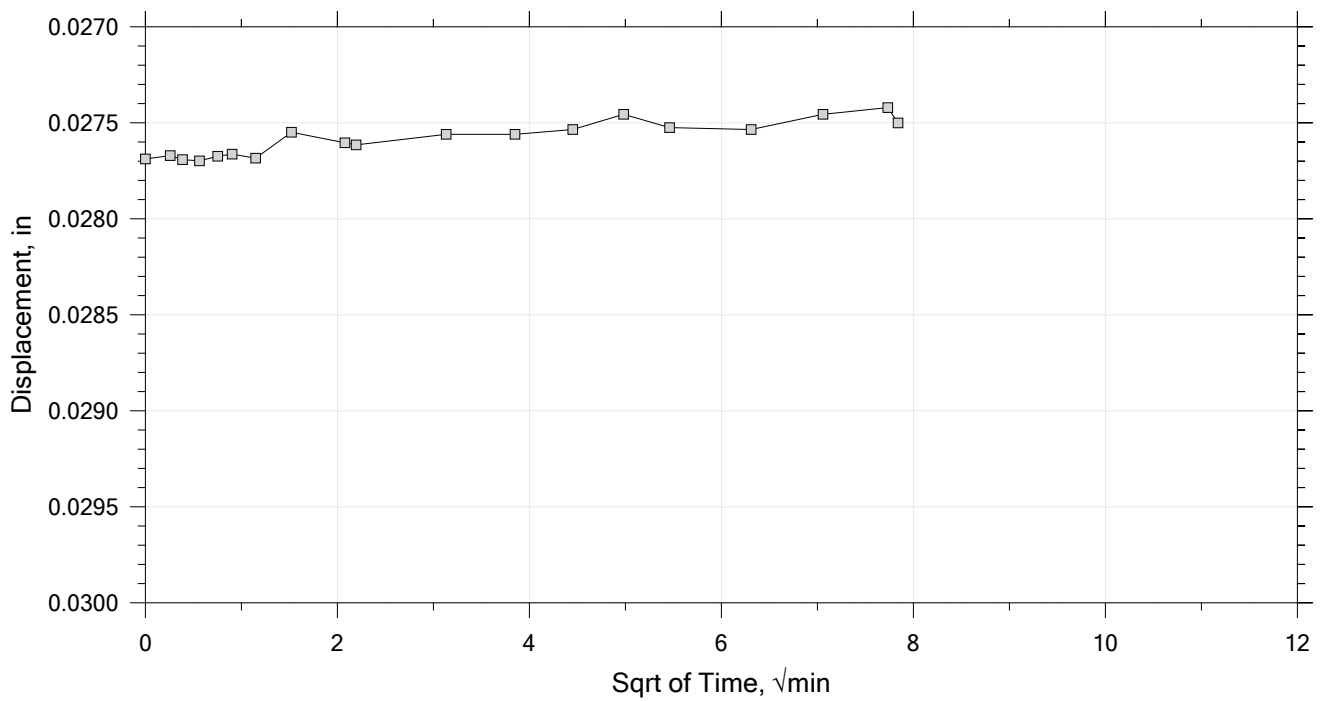
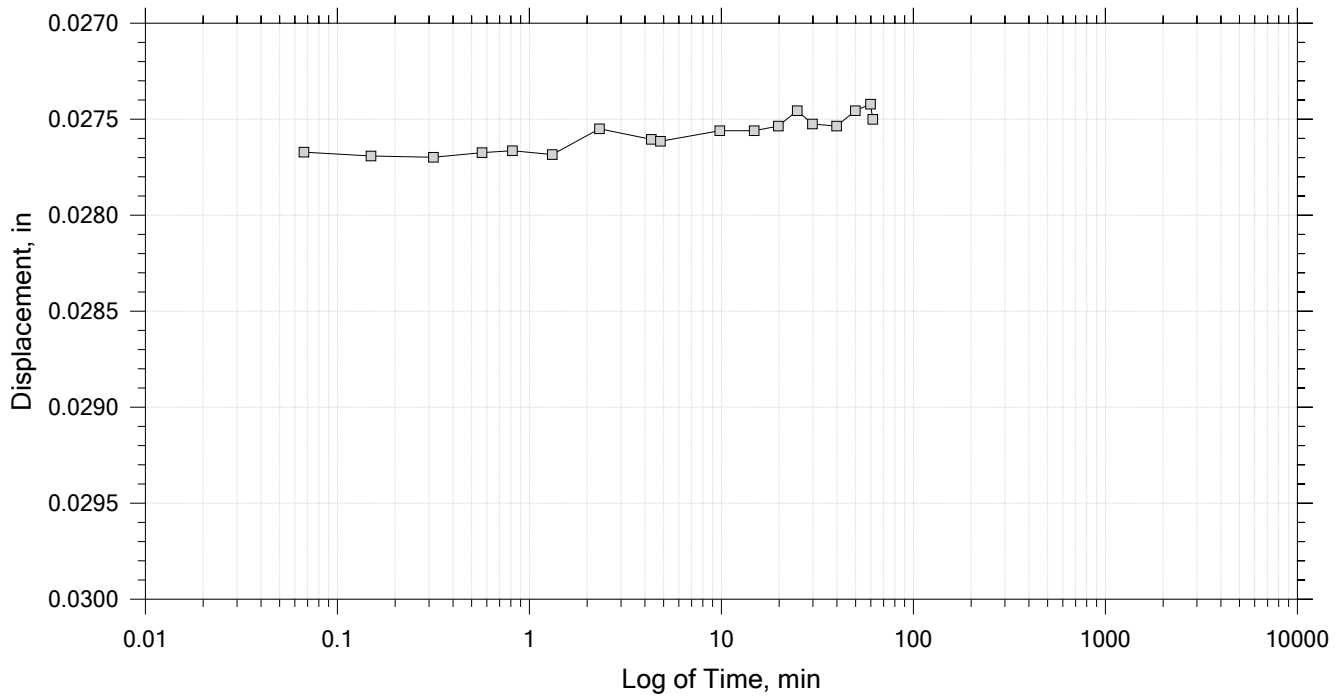
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 19

Constant Load Step

Stress: 600 psf



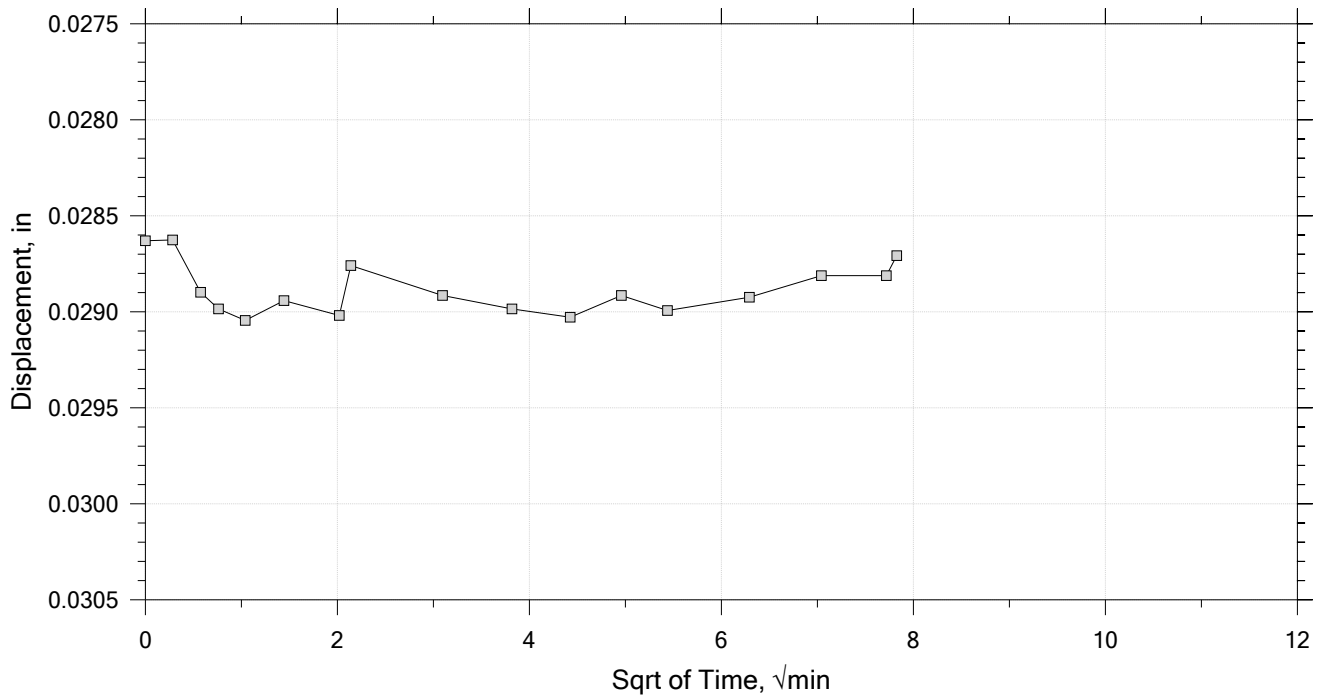
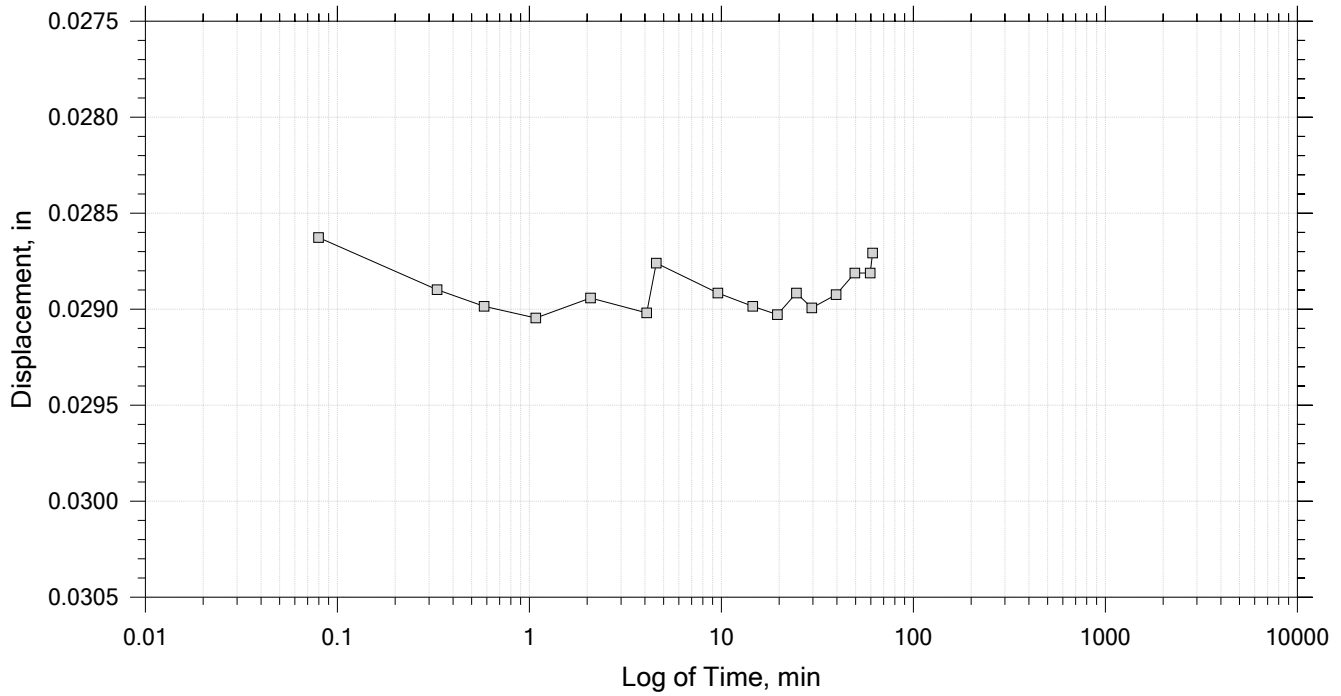
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 19

Constant Load Step

Stress: 200 psf



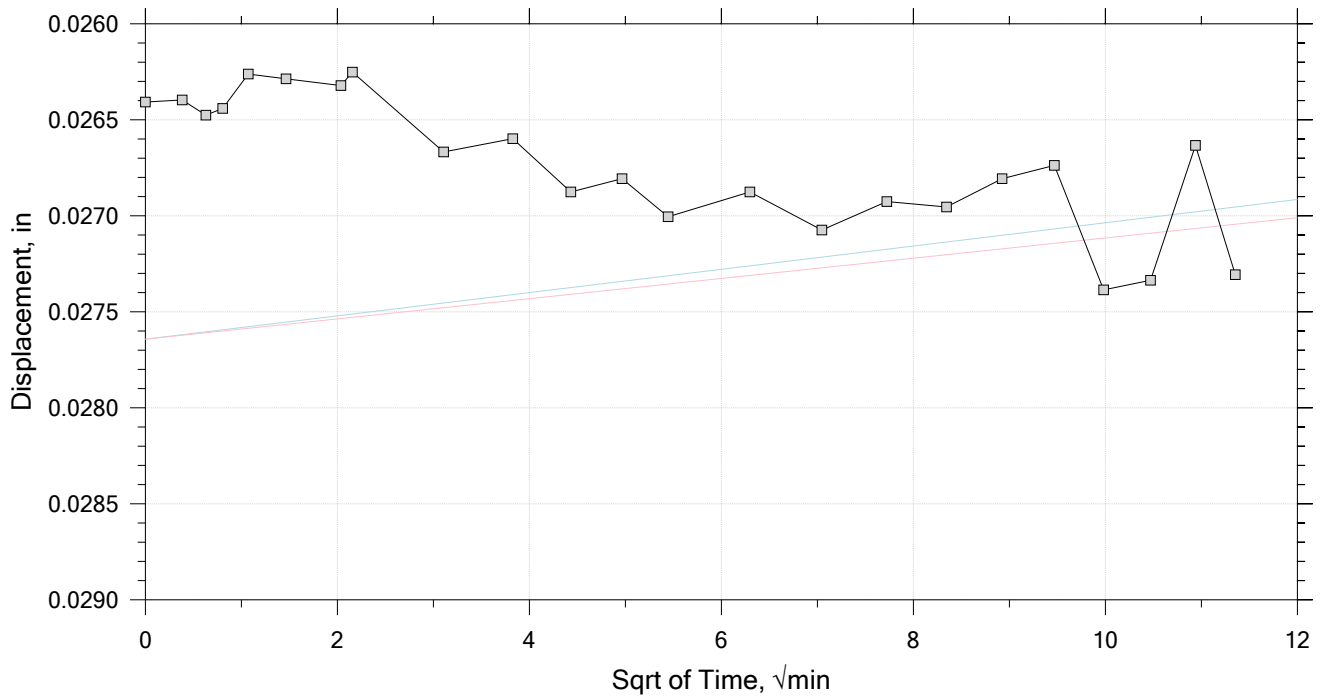
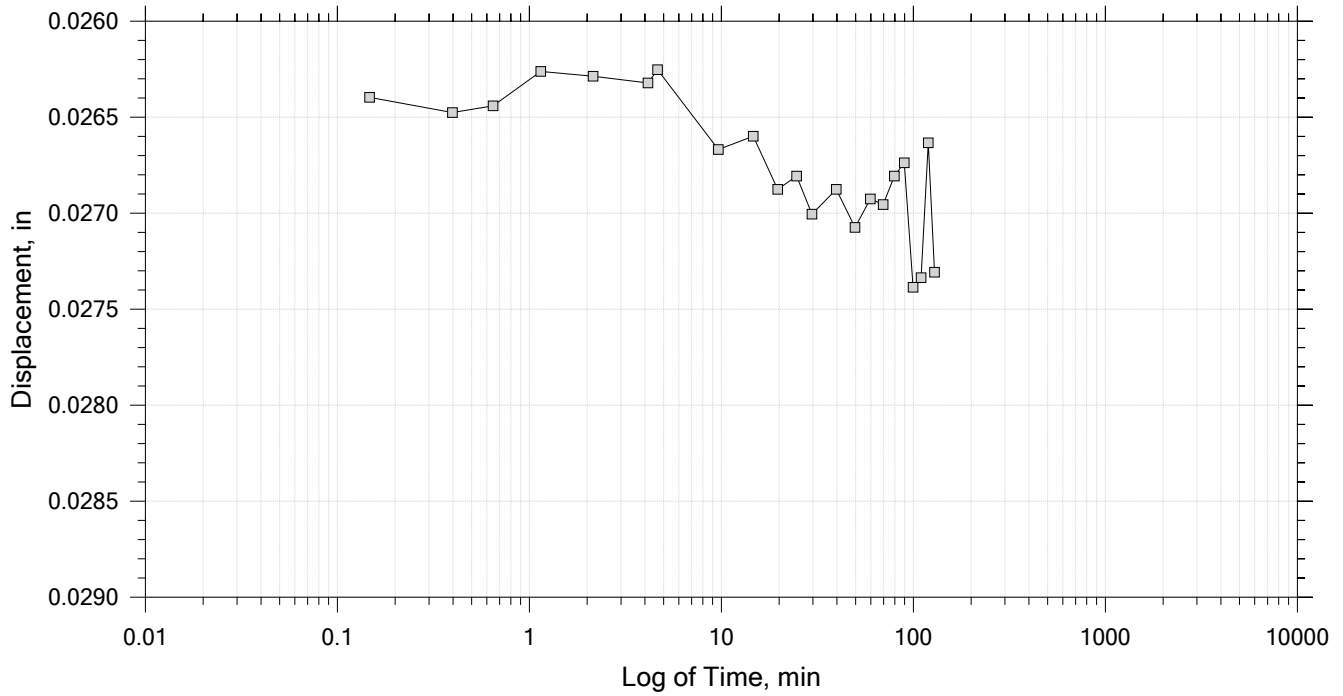
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 19

Constant Load Step

Stress: 400 psf



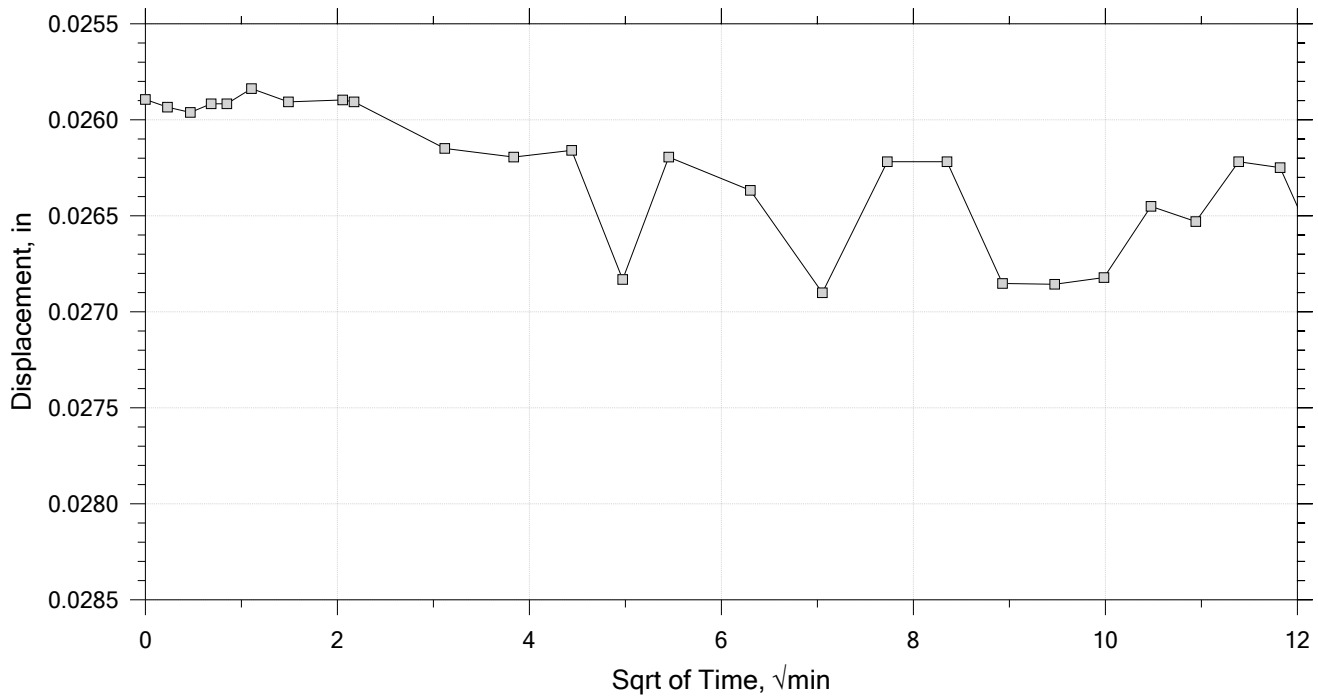
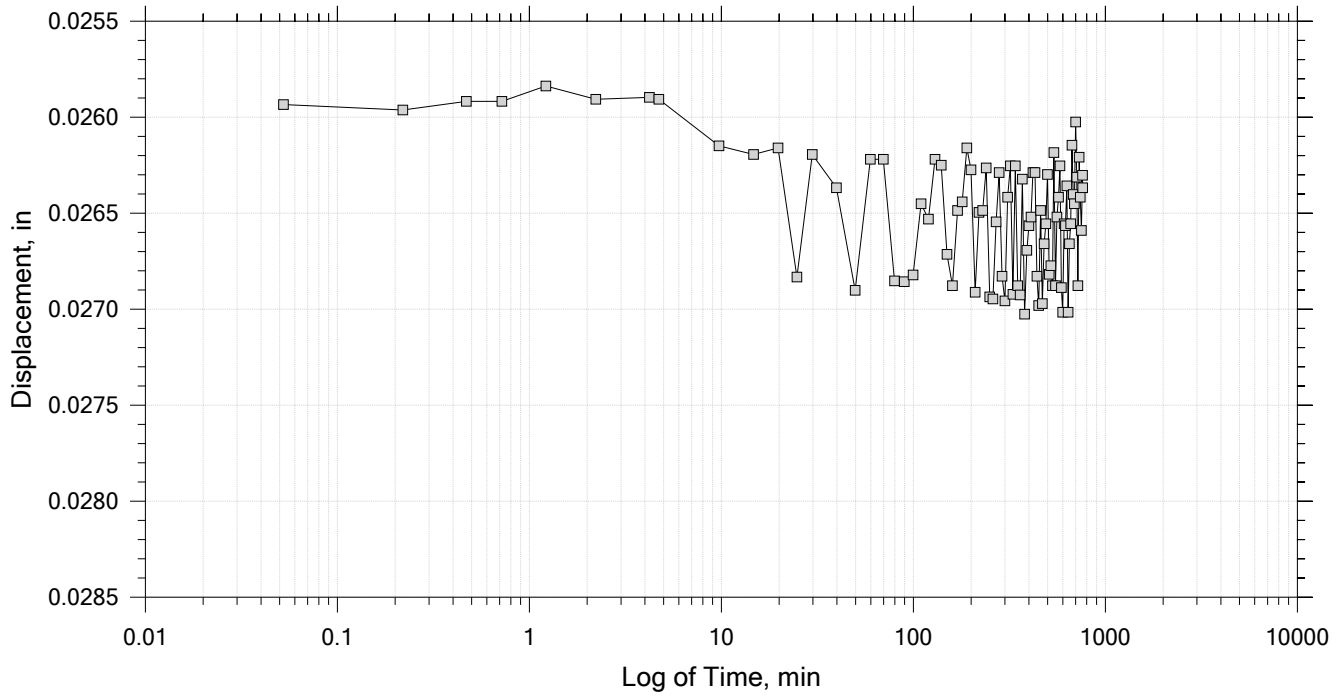
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 19

Constant Load Step

Stress: 600 psf



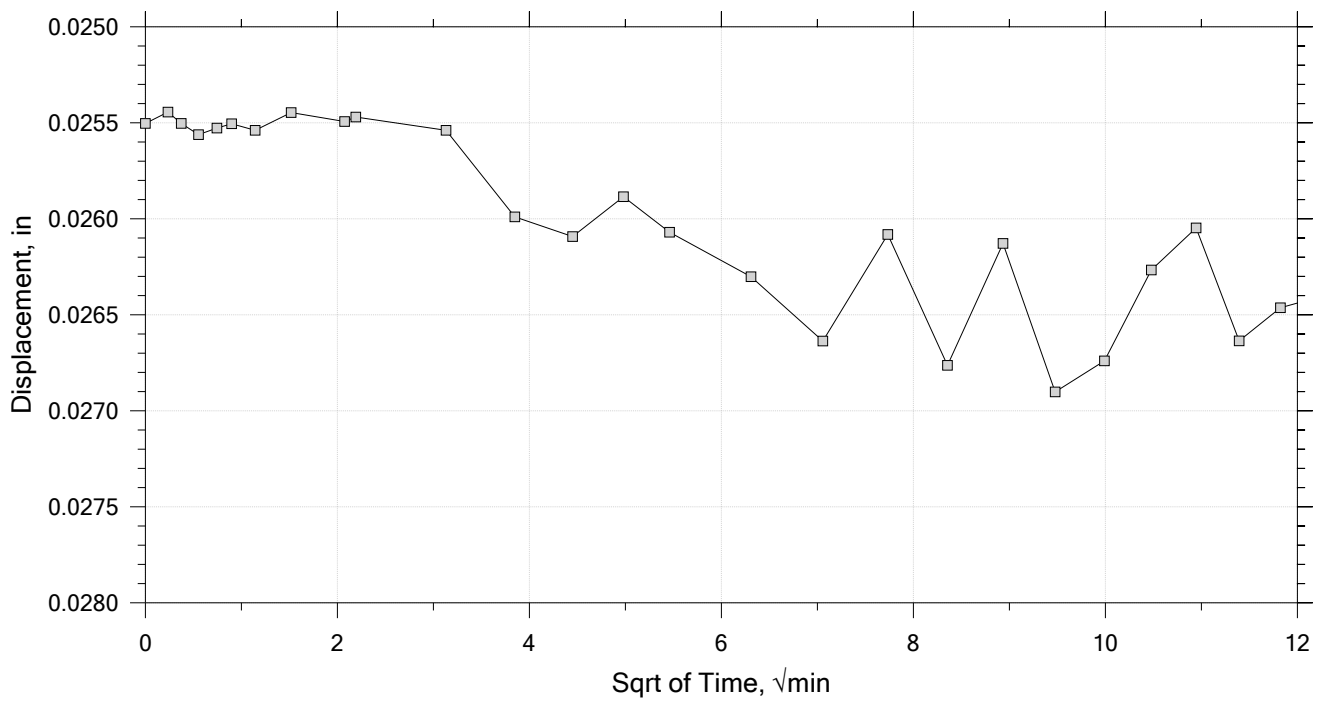
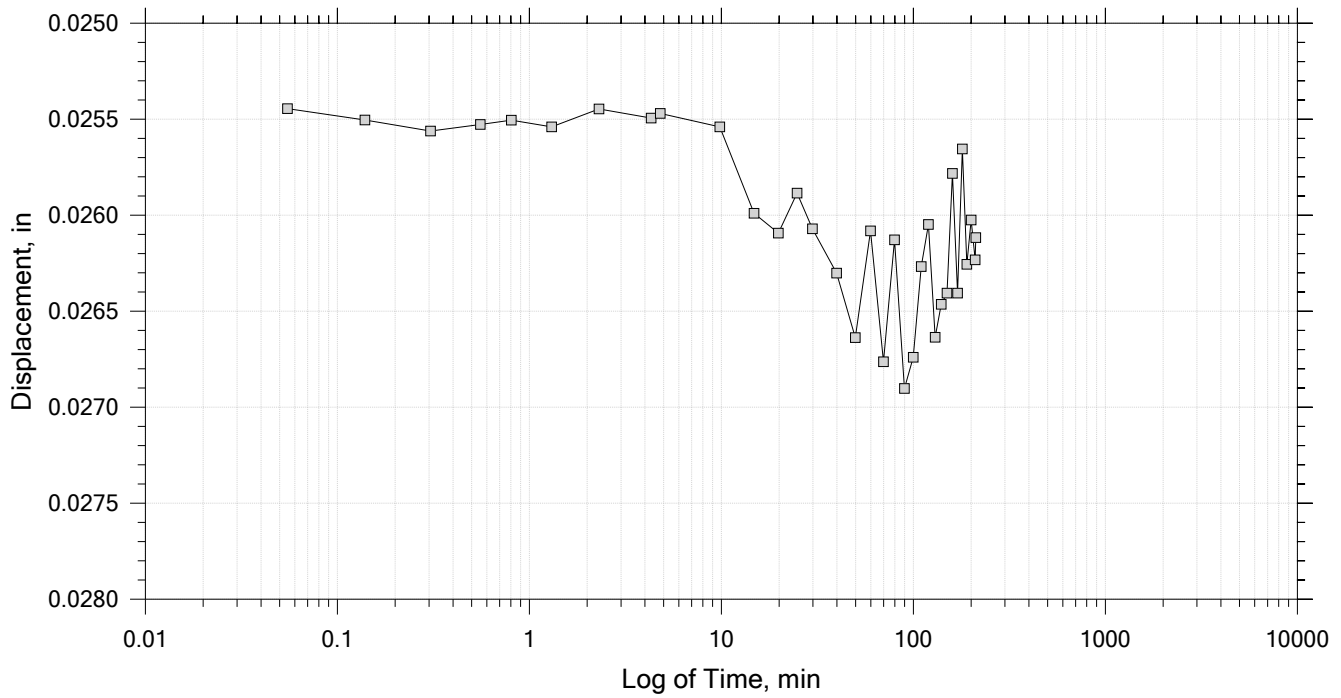
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 19

Constant Load Step

Stress: 900 psf



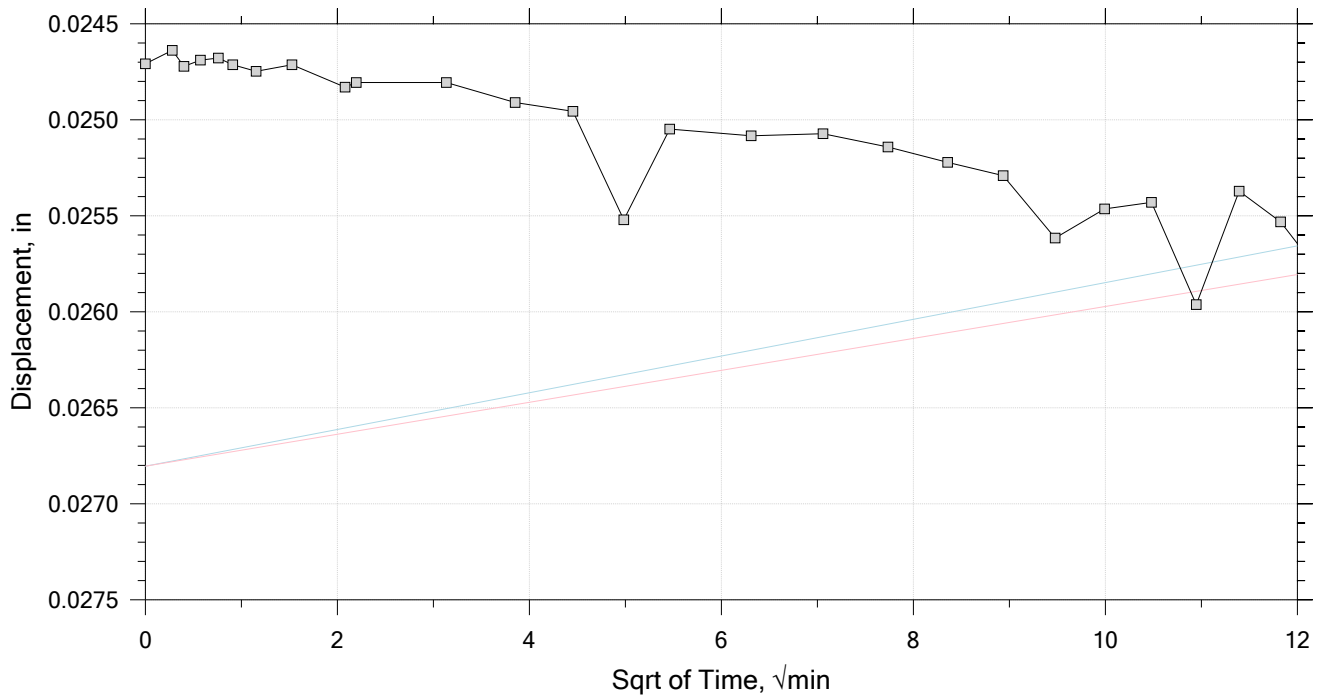
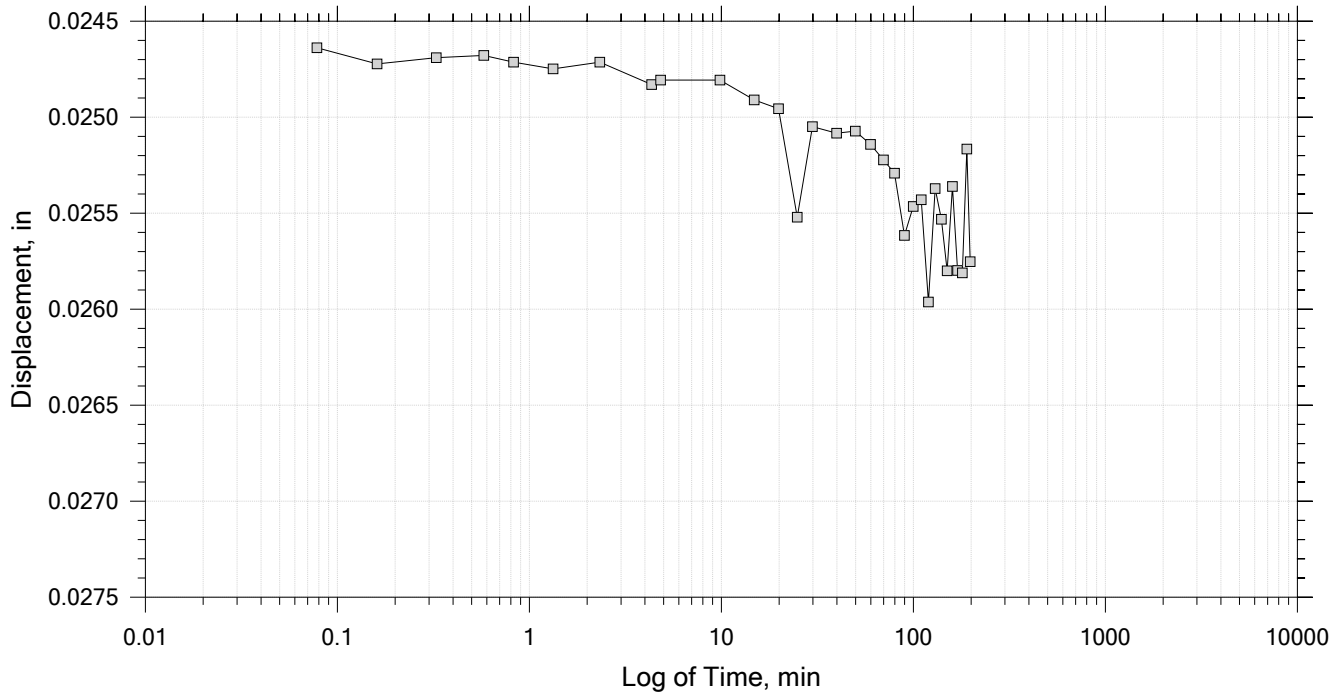
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	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 19

Constant Load Step

Stress: 1.35e+03 psf



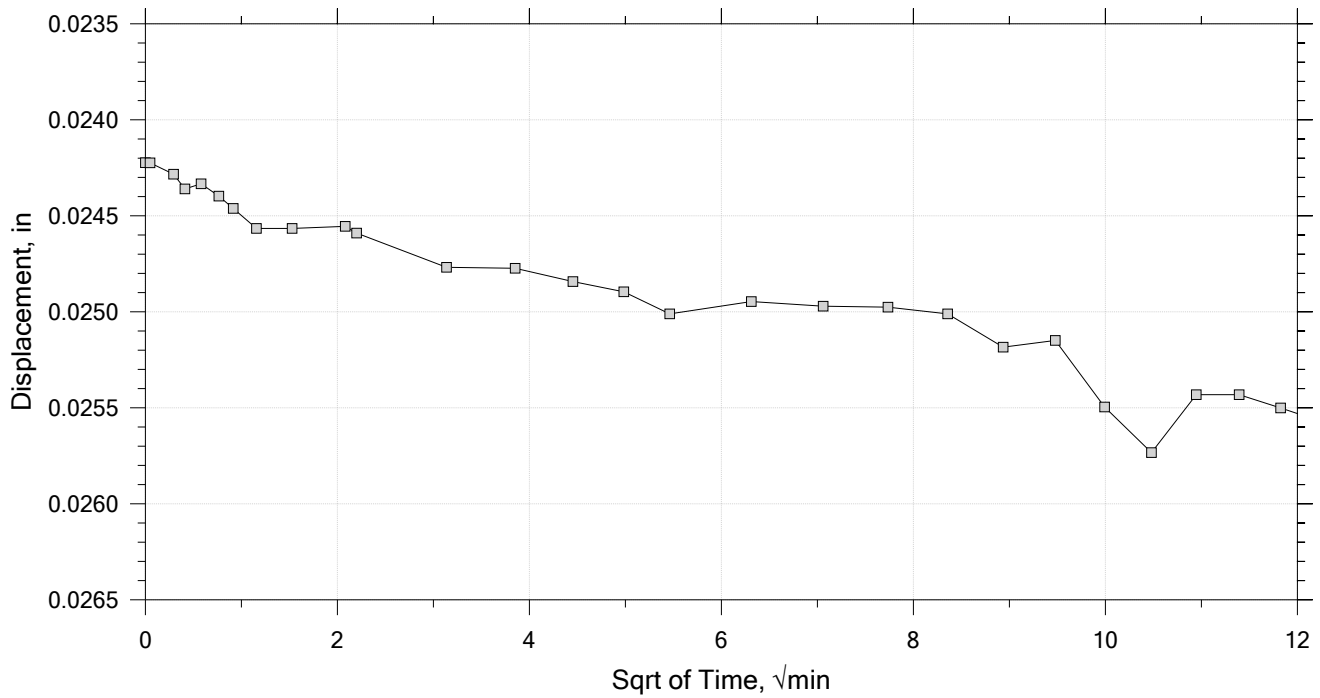
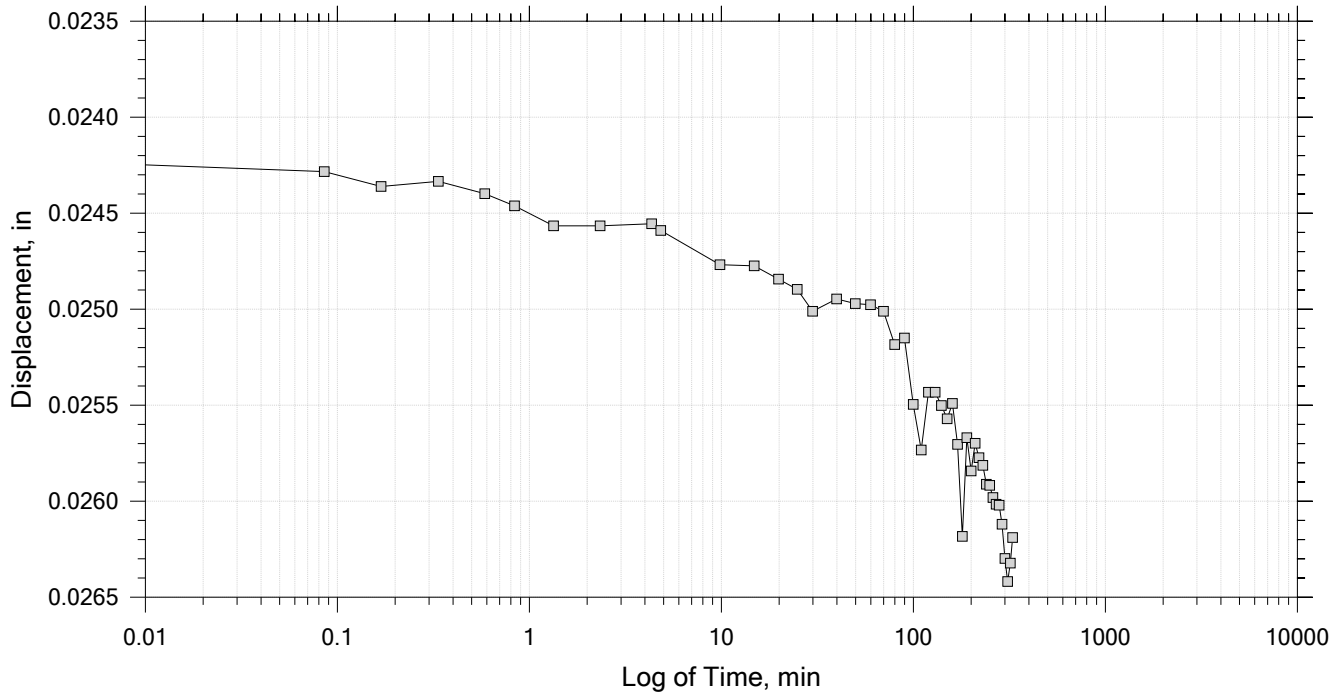
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 19

Constant Load Step

Stress: 2.02e+03 psf



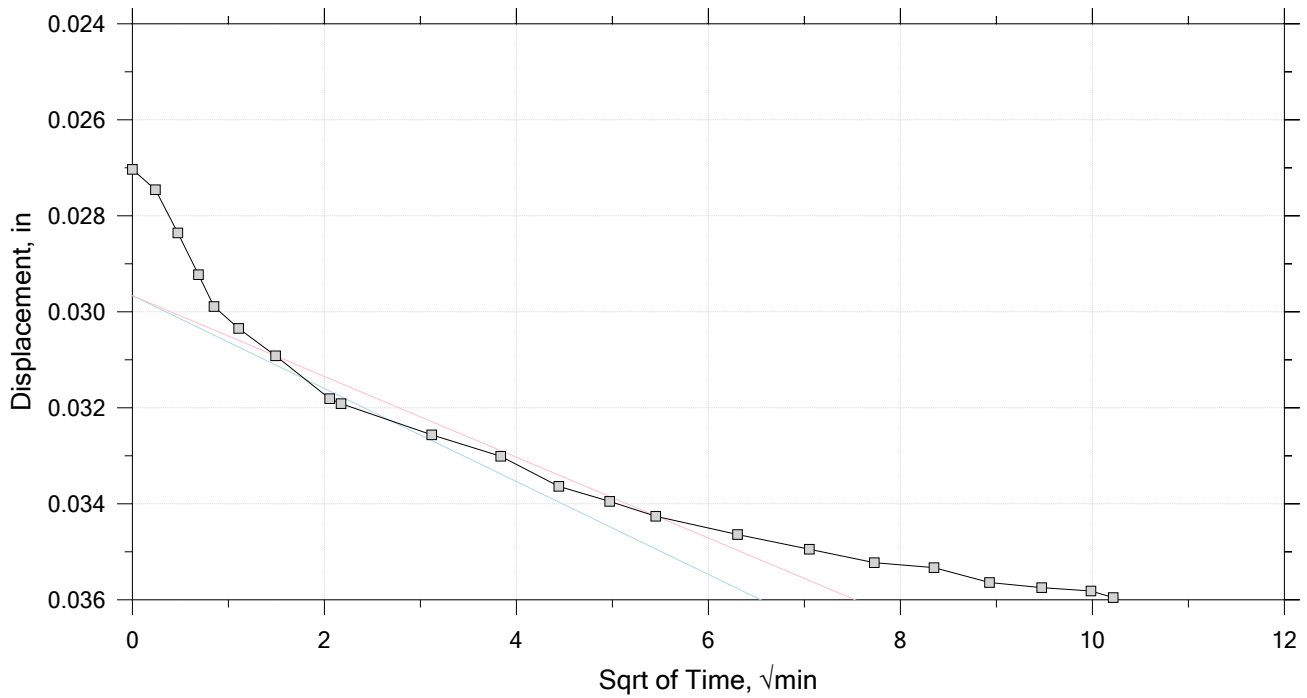
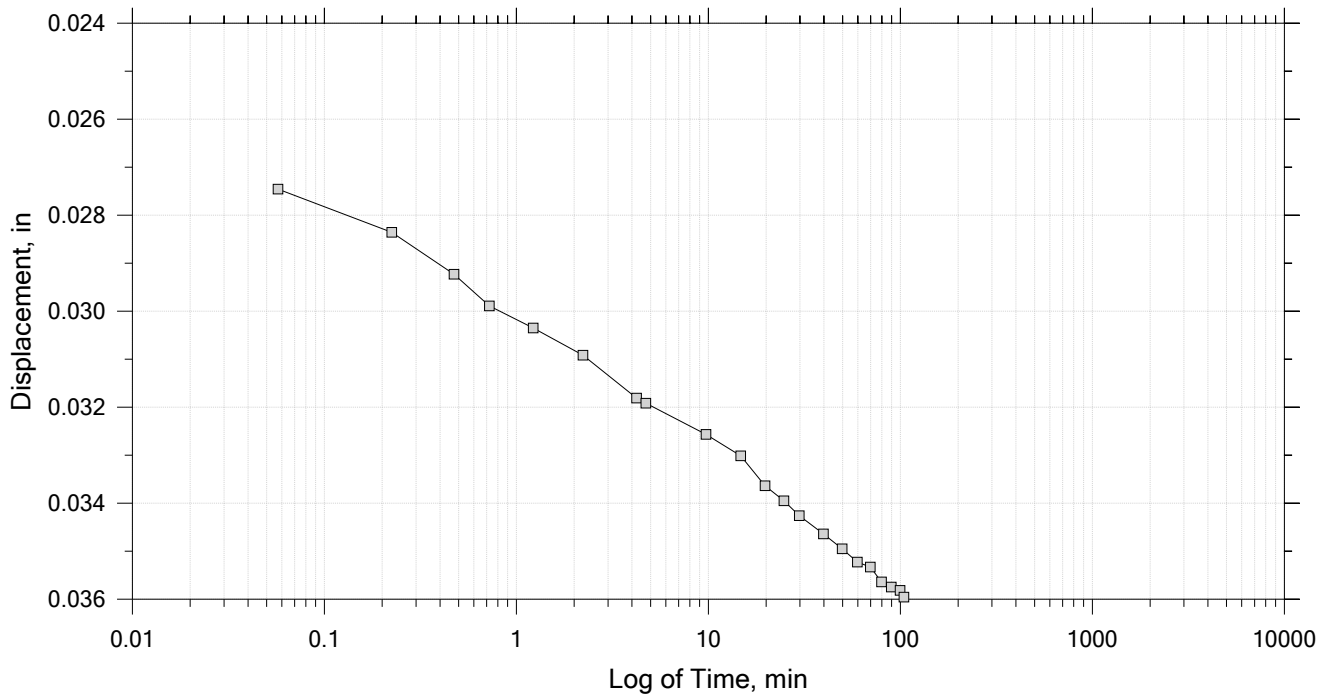
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 19

Constant Load Step

Stress: 4e+03 psf



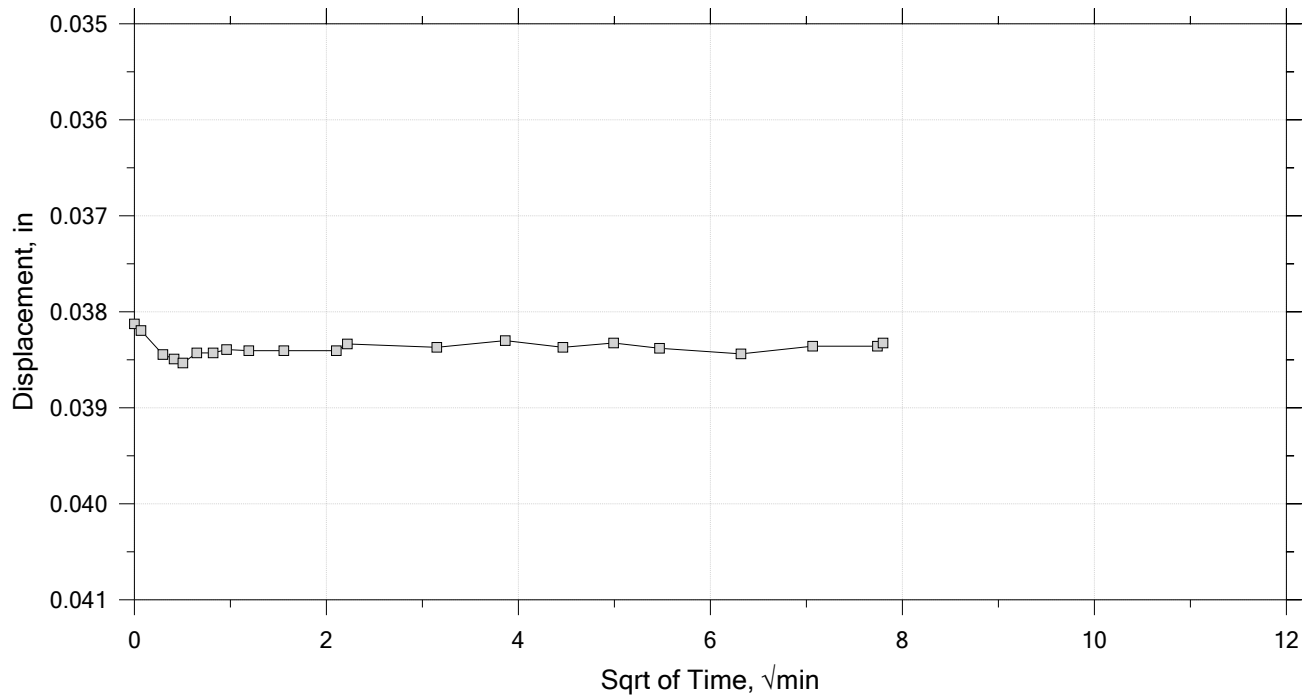
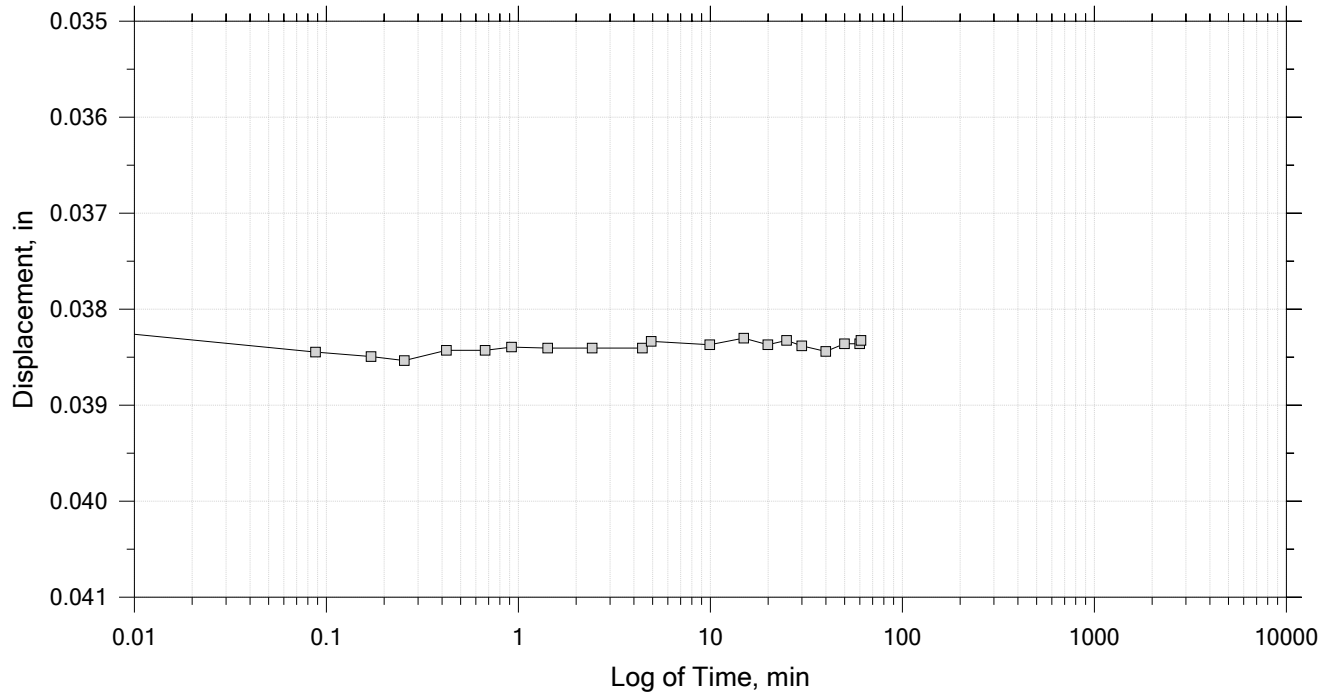
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 19

Constant Load Step

Stress: 1.35e+03 psf



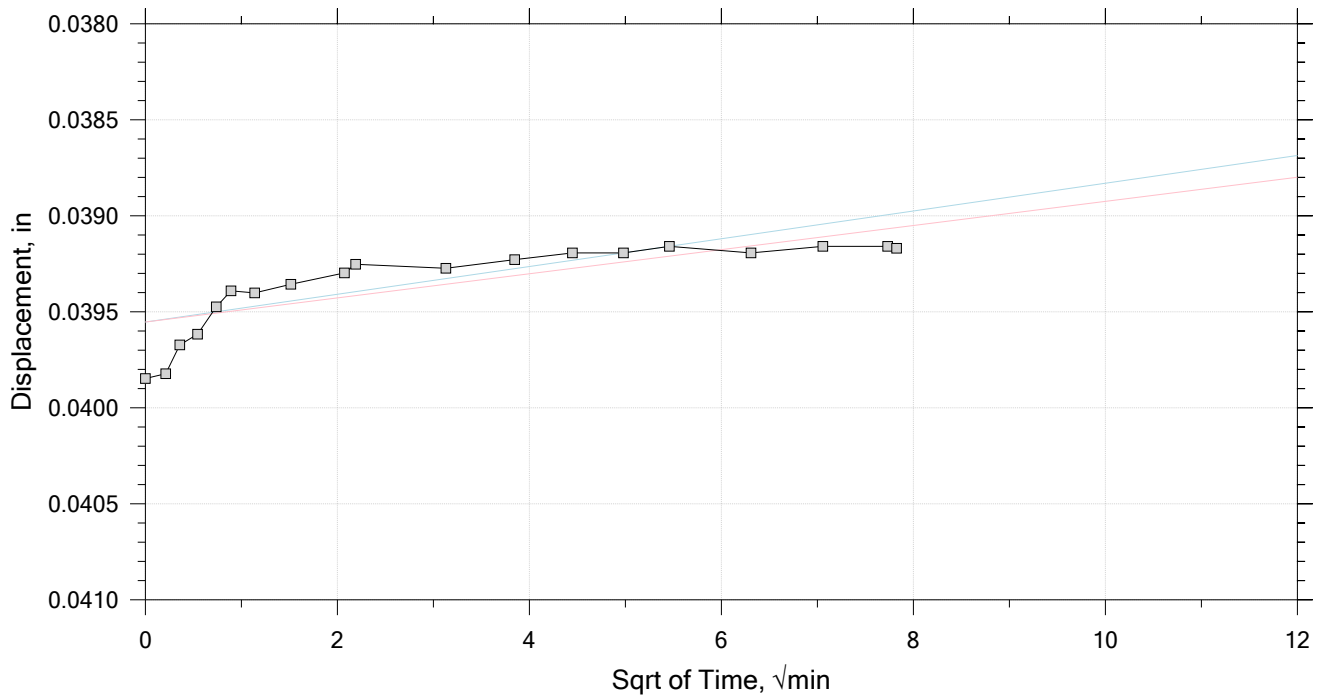
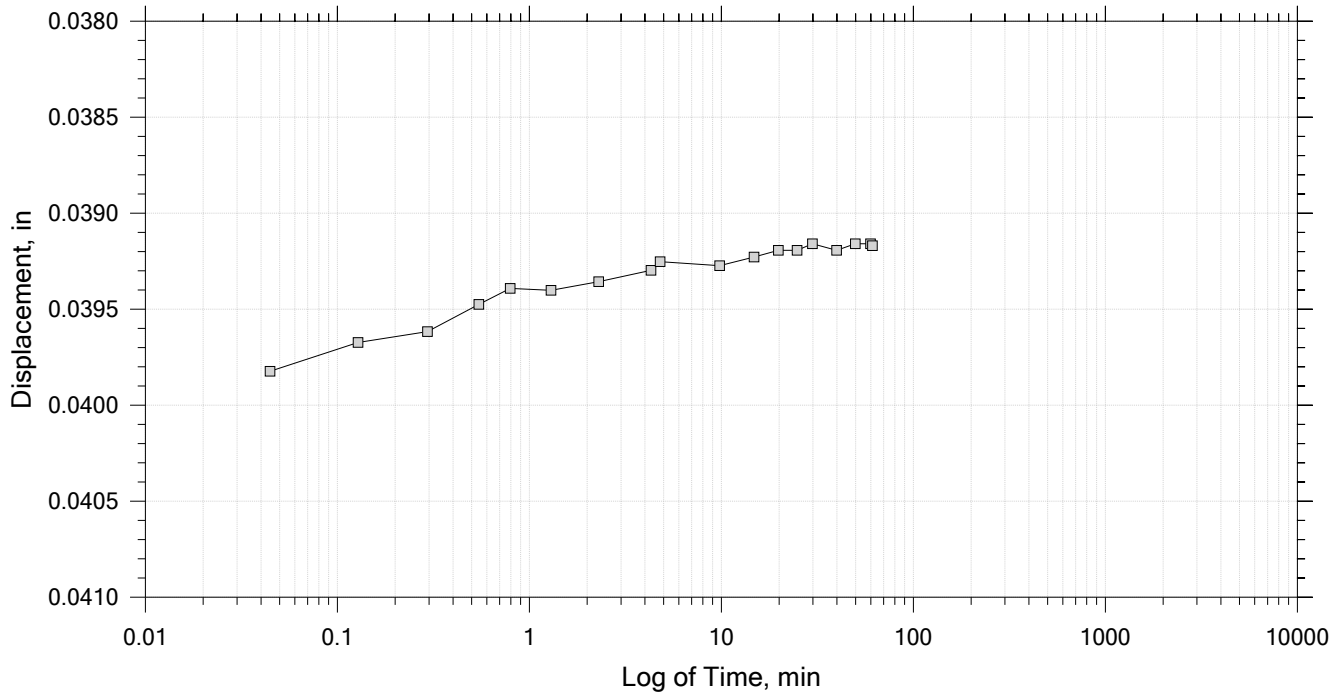
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 18 of 19

Constant Load Step

Stress: 600 psf



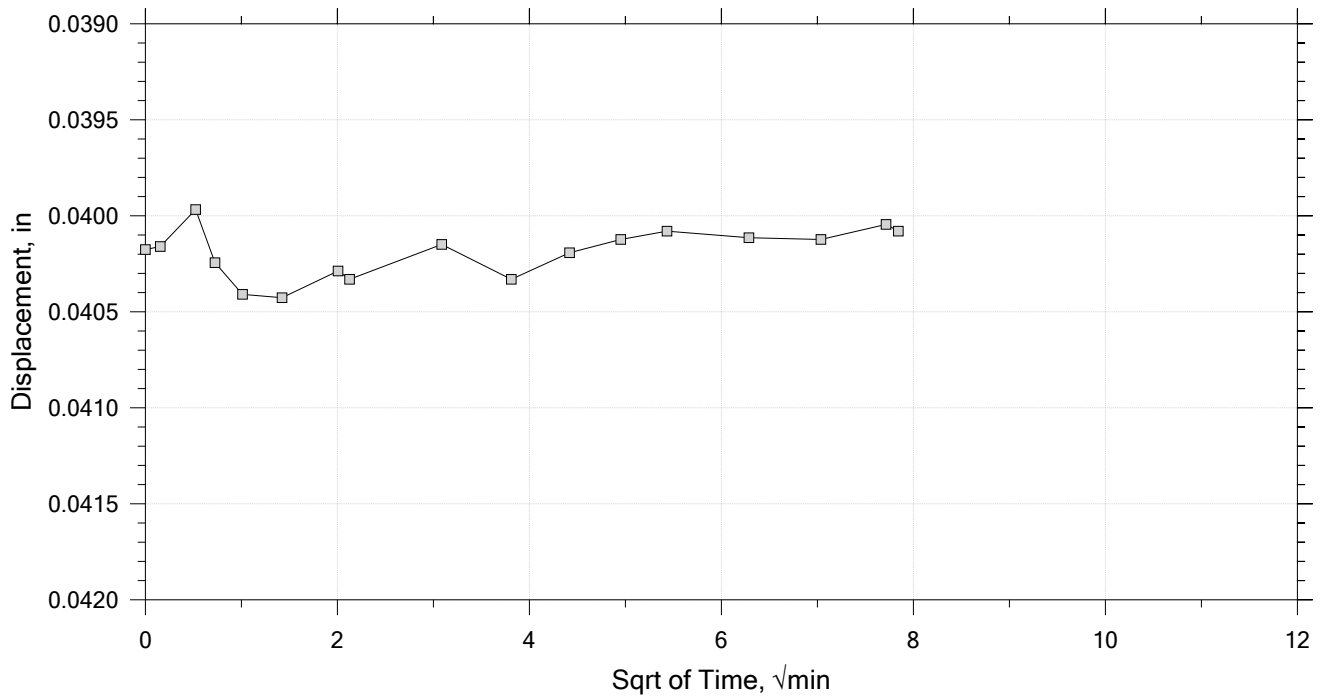
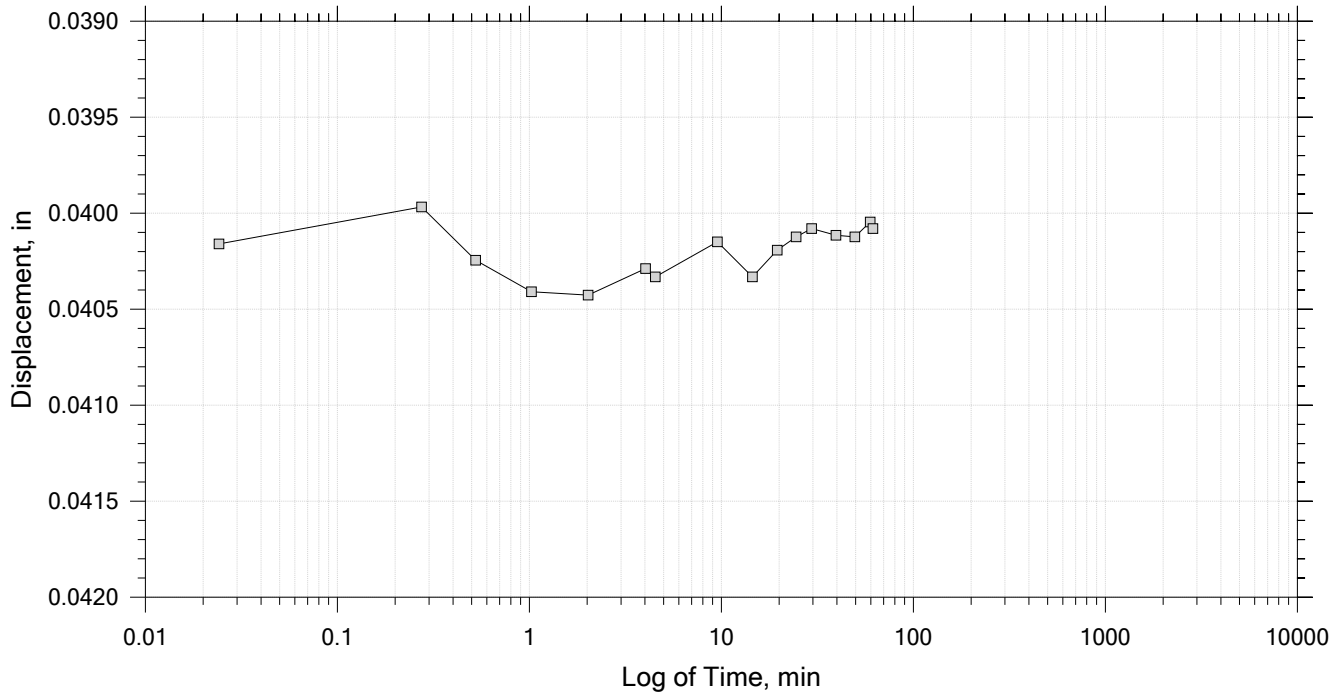
	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 19 of 19

Constant Load Step

Stress: 200 psf



	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		


One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter, in: 2.50	Specific Gravity: 2.74 (Implied)	Liquid Limit: 0
Specimen Height, in: 1.00	Initial Void Ratio: 0.851	Plastic Limit: 0
Final Height, in: 0.96	Final Void Ratio: 0.777	Plasticity Index: 0

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	223	---	"ring"	301
Mass Container, gm	37.04	111.11	111.11	60.57
Mass Container + Wet Soil, gm	126.61	267.39	264.88	214.31
Mass Container + Dry Soil, gm	106.58	230.88	230.88	180.32
Mass Dry Soil, gm	69.54	119.77	119.77	119.75
Water Content, %	28.80	30.48	28.38	28.38
Void Ratio	---	0.85	0.78	---
Degree of Saturation, %	---	98.07	100.00	---
Dry Unit Weight, pcf	---	92.344	96.179	---

Preconsolidation Stress, psf	---
Compression Ratio	0
Rebound Ratio	0
Compression Index	0
Rebound Index	0

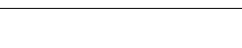
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		

One-Dimensional Consolidation by ASTM D2435 - Method B

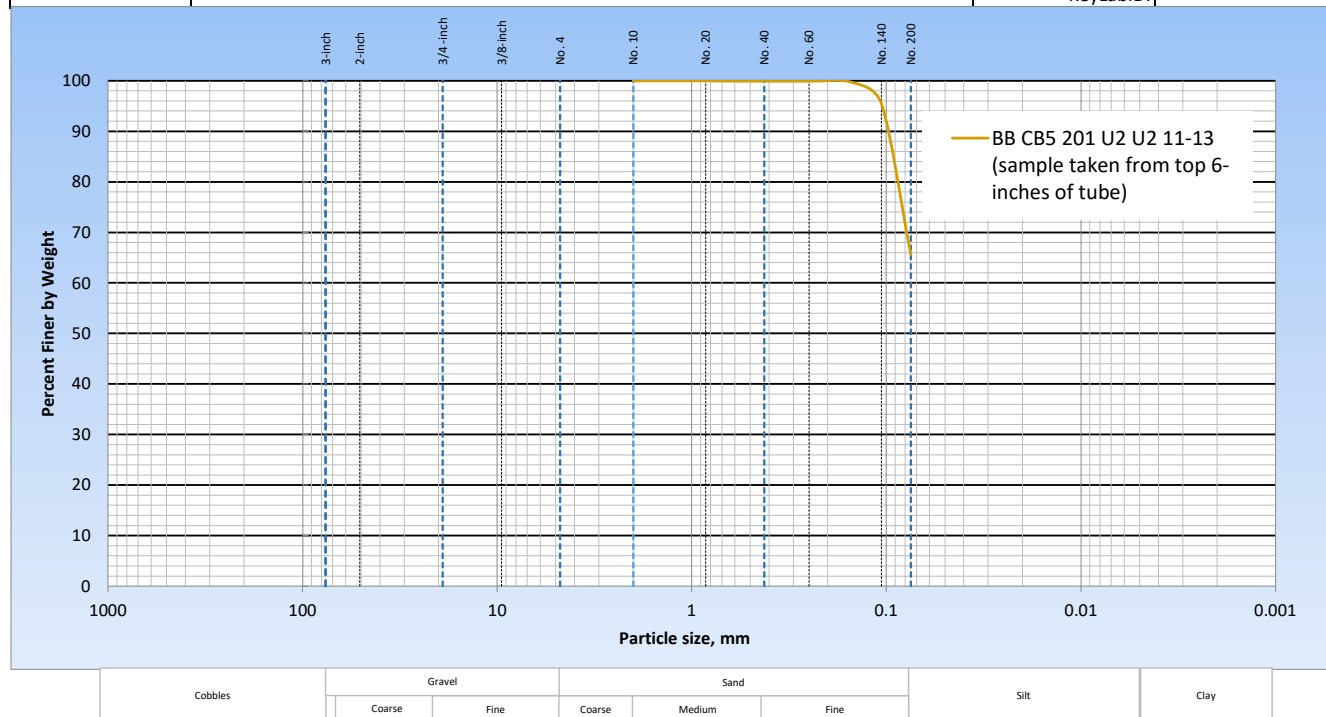
Sqrt of Time Coefficients

[illegible]

	Project Name: Caanan Bridge	Location: Caanan Maine	Project Number: 166-14
	Boring Number: EBB CB-5-201	Tester: SJR	Checker: SJR
	Sample Number: 2U	Test Date: 8/15/2020	Depth: 11.95
	Test Number: ICON 330	Preparation: Shelby Tube	Elevation:
	Description: Gray silt with sand layers		
	Remarks:		
Displacement at End of Primary			

Particle Size Distribution

	Particle Size Distribution	Job No.:	166-15
		Client:	GZA
Site Name	Caanan Bridge	Client Project No.:	
		Boring/Test Pit No:	BB CB 201
Site Location	Caanan, Maine	Sample No:	U2
		Depth:	11-13 (sample taken from top 6 inches of tube)
Test Method	AASHTO 311	Sample Type:	Shelby
		KeyLabID:	



Sieving			
Particle Size mm	Inch	% Passing	% passing 3/4 inch
100.00	3.9370	100.0	
75.00	2.9528	100.0	
50.00	1.9685	100.0	
37.50	1.4764	100.0	
25.00	0.9843	100.0	
19.00	0.7480	100.0	100.0
12.50	0.4921	100.0	100.0
9.50	0.3740	100.0	100.0
4.75	0.1870	100.0	100.0
2.00	0.0787	100.0	100.0
0.85	0.0335	100.0	100.0
0.43	0.0167	100.0	100.0
0.25	0.0098	99.9	99.9
0.15	0.0059	99.7	99.7
0.11	0.0042	95.5	95.5
0.075	0.0030	65.6	65.6

Sample Proportions	% Dry Mass
Very Coarse > 75 mm	0.0
Coarse Gravel 75 mm - 19 mm	0.0
Fine Gravel 19 mm - 4.75 mm	0.0
Coarse Sand 4.75 to 2.0 mm	0.0
Medium Sand 2.0 to 0.435 mm	0.0
Fine Sand: 0.425 to 0.075 mm	34.3
Fines: <0.075 mm	65.6

Soil Classification
Gray sandy SILT

Moisture Content:	26.7
-------------------	------

Tested by	Date	Checked by	Date	Approved by	Date	Printed Date	Figure No.
SJR	10/13/2020	sjr	10/14/2020	sjr	10/14/2020	11/27/2020	



3/15/2021

HALL BRIDGE NO. 3159 OVER BLACK STREAM

GEOTECHNICAL DESIGN REPORT

09.0026000.00

APPENDIX E – CALCULATIONS

Engineering Design Properties



Correlation of SPT N-Values to Internal Friction Angle (ϕ) Using LRFD

Rev. 0

Project: Halls Bridge (Route 23) over the Black River
Location: Canaan, Maine
Calculated By: NVW Date: 1/6/2021
Checked By: CLS Date: _____

Objective: To estimate the internal friction angle (ϕ) of subsurface strata for the Halls Bridge (Route 23) Bridge based on Standard Penetration Test (SPT) measurements and empirical correlations using N_{field} , N_{corr} and $(N_1)_{60}$ for use in LRFD analyses.

Method: Correlations between SPT N-values and the soil internal friction angle were considered based on two (2) equations presented in NCHRP Report 651 and in [Table 10.4.6.2.4-1 of the AASHTO, 8th Edition \(2017\)](#). The correlated friction angle using these two (2) correlations are presented in this document. The recommended internal friction angle to be used for design is based on the correlation presented by Peck, Hanson and Thornburn (PHT, 1974) as mentioned in Kulhawy and Mayne (1990) and recommended in NCHRP Report 651 (below).

$$\phi \approx 54 - 27.6034 * e^{-0.014 * (N_1)_{60}}$$

Given

Information: SPT measurements and subsurface conditions in borings BB-CBS-101 through BB-CBS-103, and -2010 performed by New England Boring Contractors between October 19, 2018 and November 1, 2018, and on July 29 2020, and

Assumptions: Split spoon samples and SPTs were performed using an automatic hammer. The calibrated automatic hammer efficiency of 89.5%, 71.3%, and 90.4% were used to calculate N_{60} .

Analysis

Results:

Layer	Range of SPT (N_1) ₆₀ - Values	Recomm. ϕ^1 (degrees)
Fill	7 to 73	32
Organics	6	--
Silt	2 to 19	28
Glacial Till	29 to 99	36

Attachments: 1) Figures
2) Calculations

References:

- 1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.
- 2) Boring Logs BB-CBS-101 through BB-CBS-103.
- 2) NCHRP Report 651, "LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures," 2010.
- 3) FHWA NHI-06-088, "Soils and Foundations," Volume 1, December 2006.
- 4) Youd, T.L., et. al., "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and GeoEnvironmental Engineering, October 2001.
- 5) Peck, Hanson, and Thornburn; "Foundation Engineering" 2nd ed., Wiley, New York, 1974
- 6) Kulhawy, F. and Mayne, P (1990). *Manual on Estimation of Soil Properties for Foundation Design*, Report EPRI-EL-6800. Electric Power Research Institute, Palo Alto, CA.



Estimation of Horizontal Modulus of Subgrade Reaction (k)

Rev. 0

Project: Halls Bridge (Route 23) over the Black River
 Location: Canaan, Maine
 Calculated By: NVW Date: 1/6/2021
 Checked By: CLS Date: 1/22/2021

Objective: To estimate the horizontal modulus of subgrade reaction (k) of subsurface strata for the Hall Bridge (Route 23) Bridge for use in lateral analyses. K values will be estimated using strata internal friction angles (ϕ).

Method(s): 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 2016 LPILE Technical Manual.

Given

Information: SPT measurements and subsurface conditions in borings BB-CBS-101 through BB-CBS-103, and -201 performed by New England Boring Contractors between October 19, 2018 and November 1, 2018, and on July 29 2020, and observed by GZA.

Layer	ϕ (degrees)
Fill	32
Organics	--
Silt	28
Glacial Till	36

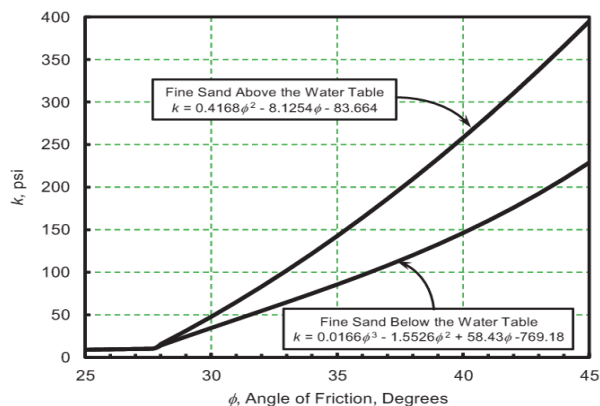


Figure 3-34 Value of k versus Friction Angle for Fine Sand Used in LPILE

Assumptions: Split spoon samples and SPTs were performed using an automatic hammer.

Analysis

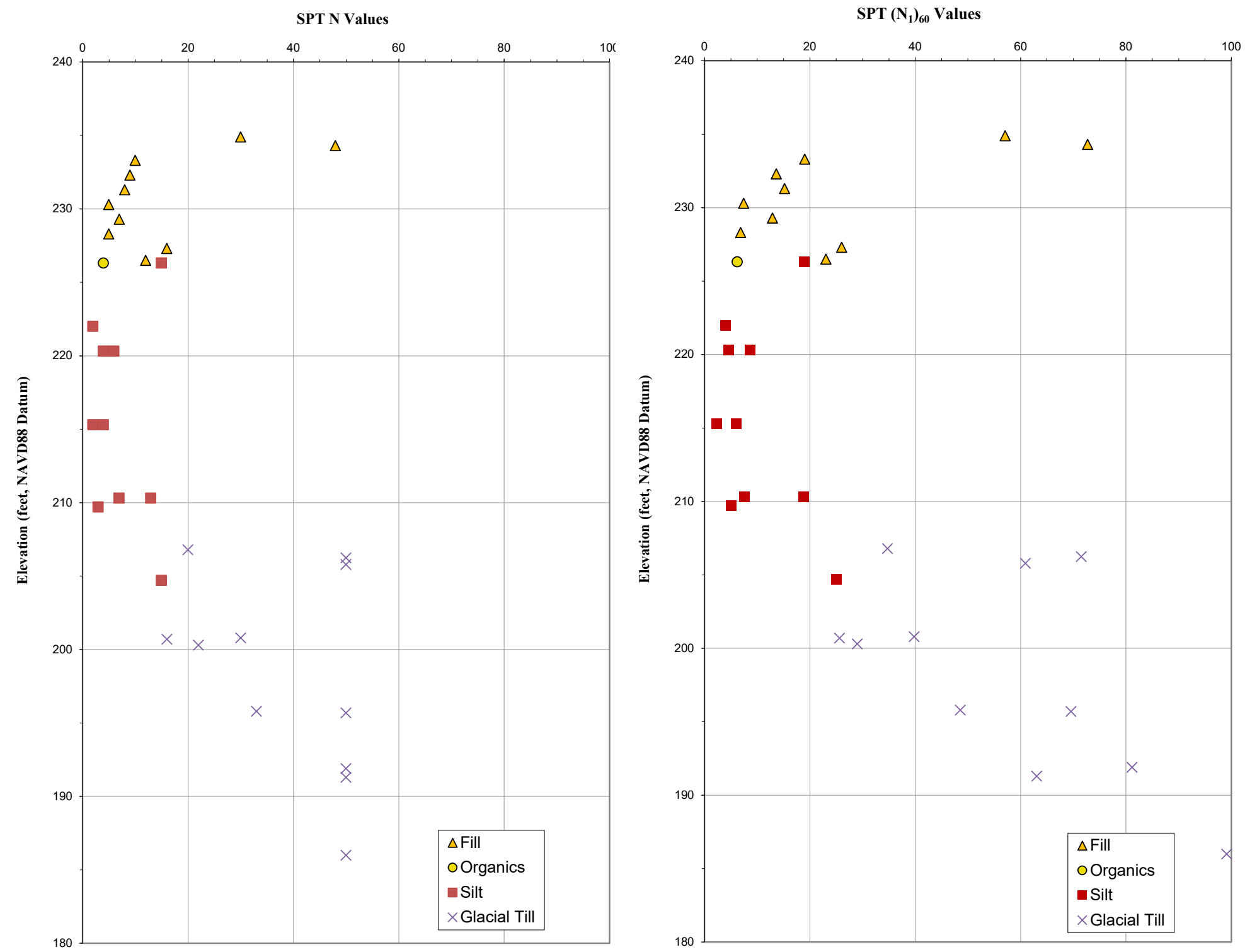
Results:

Layer	Unit Weight above GWL (pcf)	Unit Weight below GWL (pcf)	Recomm. k above GWL ² (lb/in ³)	Recomm. k below GWL (lb/in ³)
Fill	125	63	85	55
Organics	95	33	--	--
Silt	95	33	--	15
Glacial Till	130	68	--	95

Notes:

- ϕ is based on the Peck, Hanson, & Thornburn (1974) empirical correlation, as mentioned by Kulhawy & Mayne (1990).
- GWL = groundwater level.

References: 1) Technical Manual for LPILE 2016 by Ensoft, Inc., dated January 14, 2016.



- Notes:**
- 1. Project datum = North American Vertical Datum (NAVD 88).
 - 2. Standard Penetration Test (SPT) field values shown for all borings.
 - 3. N values greater than 100 have been plotted as 100 for clarity.
 - 5. Corrected, (N₁)₆₀ values plotted on the figure are based on the following:

$$N_{1(60)} = N_m C_N C_E C_R C_S C_B$$

where:
N_M = measured SPT 'N' Value
C_N = normalization factor for overburden stress
 $C_N = (P_a/\sigma'_v)^{0.5} \leq 1.7$
P_a = 2088 psf (100 kPa)
σ'_v = effective overburden stress
C_E = hammer energy ratio correction factor
C_B = borehole diameter correction factor
C_R = rod length correction factor
C_S = sampler liner correction factor



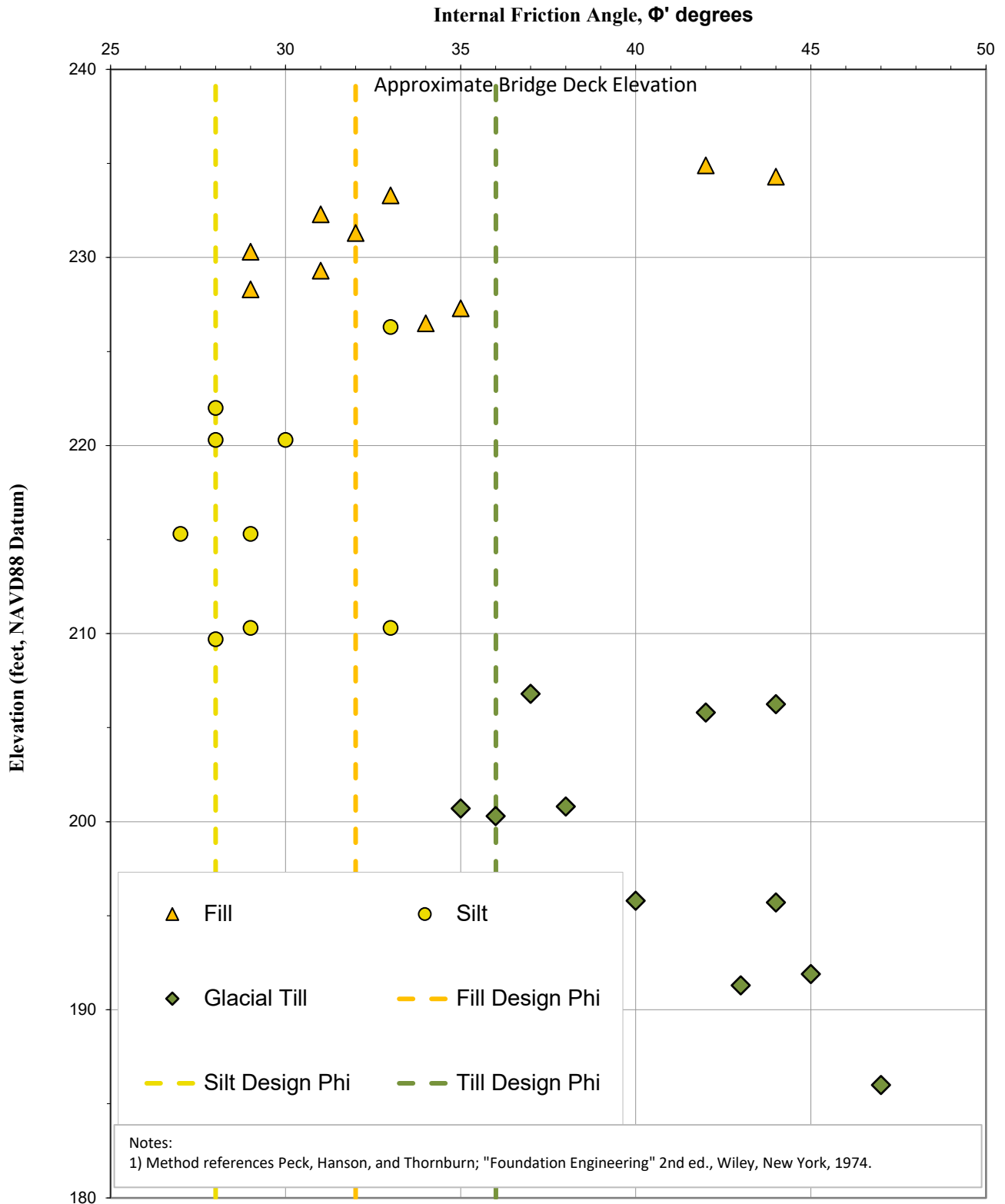
PROJ MGR: NVW
DESIGNED BY: NVW
REVIEWED BY:CLS

DRAWN BY: NVW
DATE: 4/5/2018

HALL BRIDGE(ROUTE 23) OVER THE BLACK STREAM
CANAAN, MAINE

UNCORRECTED AND CORRECTED N-VALUES VERSUS ELEVATION
BY STRATUM

FIGURE 1



PROJ MGR: NVW
 DESIGNED BY: NVW
 REVIEWED BY: CLS

DRAWN BY: NVW
 DATE: 4/5/2018



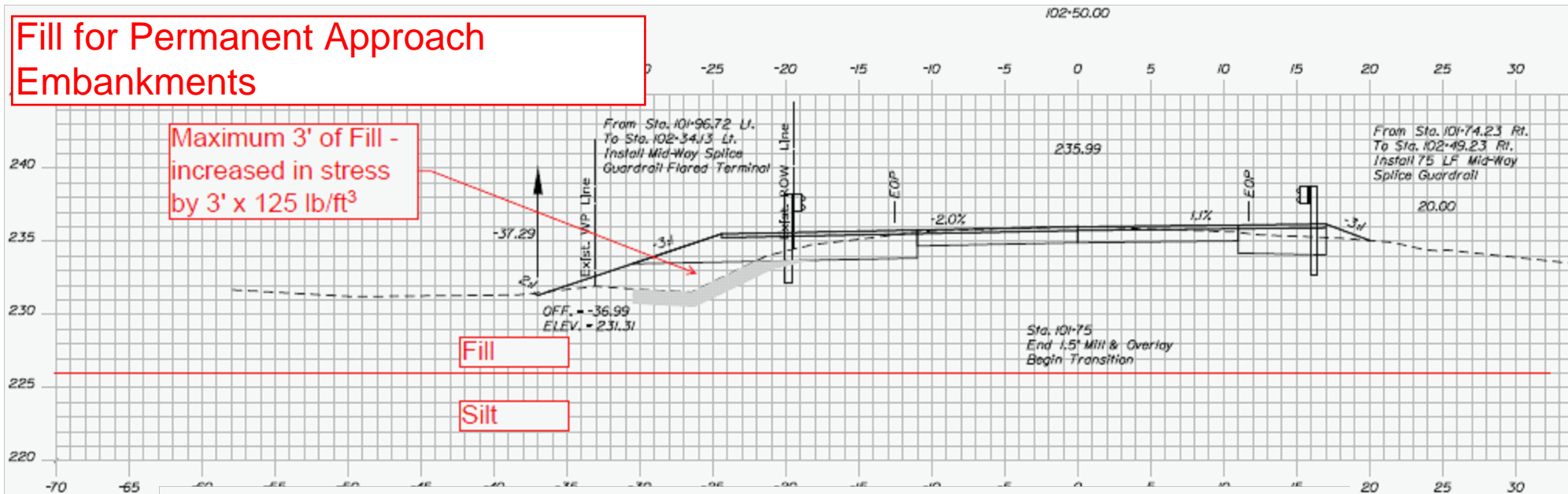
**HALL BRIDGE(ROUTE 23) OVER THE BLACK STREAM
 CANAAN, MAINE**

**INTERNAL FRICTION ANGLES BASED ON STANDARD
 PENETRATION TEST (SPT) N-VALUES FOR ALL
 STRATA**

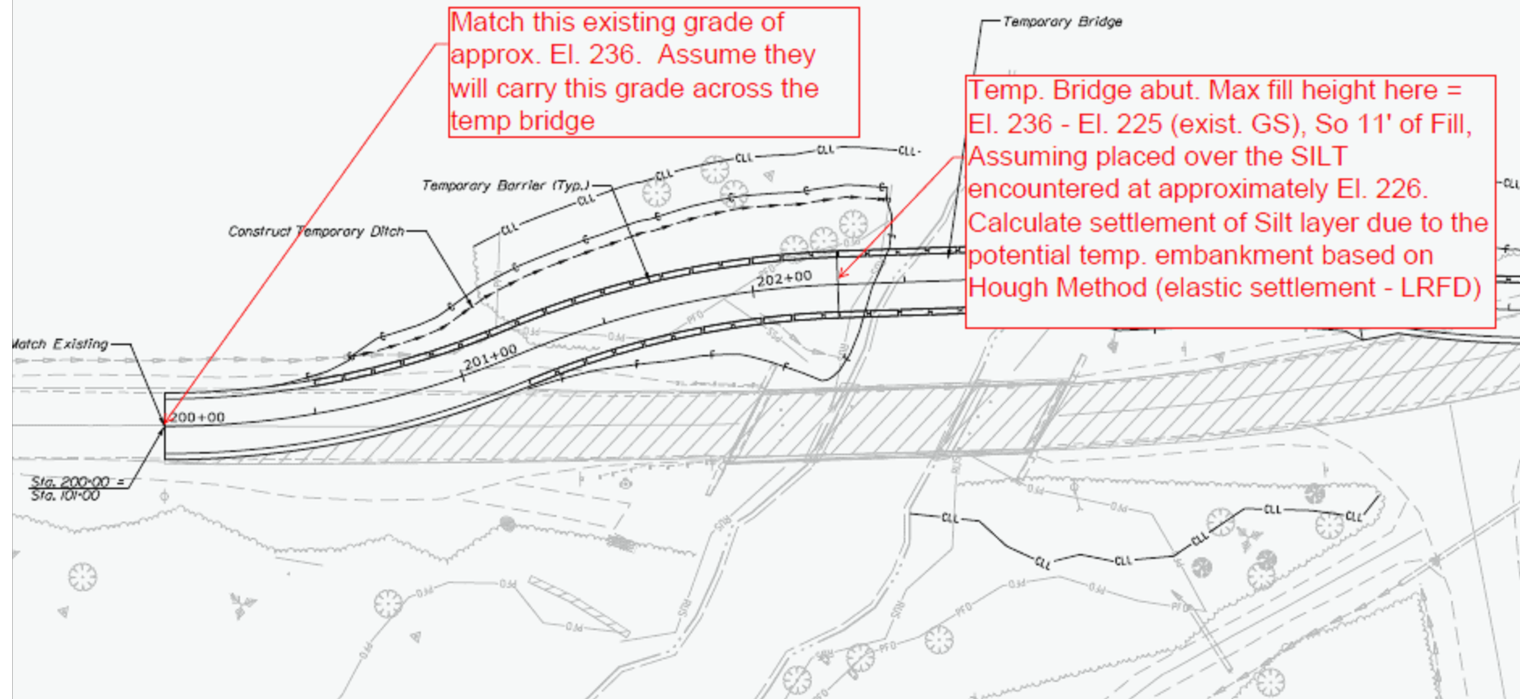
FIGURE 2

Elastic
Settlement -
Approach
Embankments
and Temp
Bridge

Fill for Permanent Approach Embankments



Fill for Permanent Approach Temporary Embankments





GZA
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 477 Congress Street
 Suite 700
 Portland, Maine 04101
 207-879-9190
 Fax 207-879-0099

*Engineers and
 Scientists*

JOB: 09.0026000.00 Hall Bridge
 SUBJECT: Settlement - Fill Placed on Existing Soil
 SHEET: 1 OF 5
 CALCULATED BY N. Williams 1/19/21
 CHECKED BY C. Snow 1/22/2021

Objective

Calculate amount of settlement of in-situ soil due to placement of fill at the approach embankment due to widening using the Hough Method and SPT data.

References

1. 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition with Interims

Soil Properties and Geotechnical Inputs

$N_{60} := 9$ N_{60} value from SPT of in-situ Silt encountered in borings
 $\gamma_w := 62.4 \cdot \text{pcf}$ Unit weight of water.

Soil properties taken from Table 6 of NAVFAC, based on Soft to medium stiff SILT from SPT information

$\gamma_{\text{SILT}} := 95 \cdot \text{pcf}$ Wet Unit weight of silt
 $C_{\text{Prime}} := 25$ C_{Prime} = Bearing Capacity Factor based on Corrected SPT
 Based on Table 10.6.2.4.2-1

Assume groundwater table at the top of the in-situ soil (Approx. El. 228 measured in BB-CBS-103)

Embankment Dimensions

$H_{\text{fill}} := 3 \cdot \text{ft}$ Max fill height occurs due to embankment widening
 $H_{\text{Exist. fill}} := 6 \cdot \text{ft}$ Existing fill height
 $\gamma_{\text{FILL}} := 125 \cdot \text{pcf}$ Total unit weight of soil based on Granular Borrow for Underwater Backfill - BDG Type 4 soil

$$\Delta H := H_c \cdot \left(\frac{1}{C_{\text{Prime}}} \right) \cdot \log \left[\frac{(\sigma_{v0} + \Delta \sigma_v)}{\sigma_{v0}} \right] \quad \text{Settlement equation based on Hough Method}$$

ΔH = settlement

C_{Prim} = Tangent value of constrained modulus of the soil (stress units – the same units as $\Delta \sigma_v$)

σ_{v0} = in-situ effective stress

$\Delta \sigma_v$ = Increase in effective stress within the sublayer due to the load.

H_c = sublayer thickness

Based on boring BB-CBS-103 at abutment 2, a 23' thick layer of soft to medium stiff SILT is used in the settlement analysis



$$H_c := 23 \cdot \text{ft}$$

Sublayer thickness

$$d := \frac{H_c}{2} = 11.5 \cdot \text{ft}$$

d = depth of the midpoint of the layer

$$\sigma_{vo} := H_{\text{Exist. fill}} \cdot \gamma_{\text{FILL}} + d \cdot (\gamma_{\text{SILT}} - \gamma_w) = 1124.9 \cdot \text{psf}$$

Initial vertical effective stress - at midpoint of the layer

$$\Delta \sigma_v := H_{\text{fill}} \cdot \gamma_{\text{FILL}} = 375 \cdot \text{psf}$$

Change in vertical effective stress due to the fill

$$i := \frac{8 \cdot \text{ft}}{8 \cdot \text{ft} + d} = 0.41$$

Influence Factor assuming a 1H:2V stress distribution starting at top of Silt, Assume 8' average width of new fill

$$\Delta H := i \cdot H_c \cdot \left(\frac{1}{C_{\text{Prime}}} \right) \cdot \log \left[\frac{(\sigma_{vo} + \Delta \sigma_v)}{\sigma_{vo}} \right] = 0.6 \cdot \text{in}$$

Settlement of Silt Layer

$$S_e = \sum_{i=1}^n \Delta H_i \quad (10.6.2.4.2-2)$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log \left(\frac{\sigma'_o + \Delta \sigma_v}{\sigma'_o} \right) \quad (10.6.2.4.2-3)$$

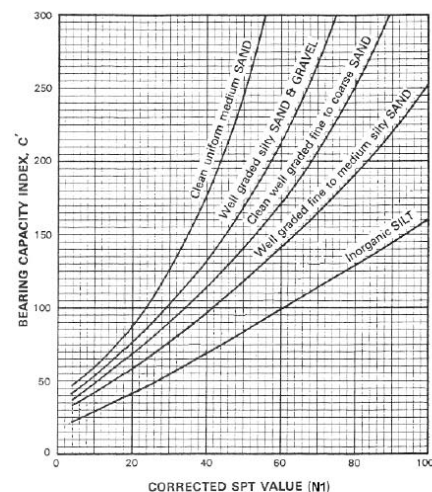
where:

- n = number of soil layers within zone of stress influence of the footing
- ΔH_i = elastic settlement of layer i (ft)
- H_c = initial height of layer i (ft)
- C' = bearing capacity index from Figure 10.6.2.4.2-1 (dim)
- σ'_o = initial vertical effective stress at the midpoint of layer i (ksf)
- $\Delta \sigma_v$ = increase in vertical stress at the midpoint of layer i (ksf)

In Figure 10.6.2.4.2-1, N_1 shall be taken as N_{160} , Standard Penetration Resistance, N (blows/ft), corrected for overburden pressure as specified in Article 10.4.6.2.4.

10-58

AASHTO



Reference: Hough, "Compressibility as a Basis for Soil Bearing Values" ASCE 1959

Figure 10.6.2.4.2-1—Bearing Capacity Index versus Corrected SPT (modified from Cheney and Chassie, 2000, after Hough, 1959)



GZA
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 477 Congress Street
 Suite 700
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 207-879-9190
 Fax 207-879-0099

*Engineers and
 Scientists*

JOB: 09.0026000.00 Hall Bridge
 SUBJECT: Settlement - Fill Placed on Existing Soil
 SHEET: 1 OF 5
 CALCULATED BY N. Williams 1/19/21
 CHECKED BY C. Snow 1/22/21

Objective

Calculate amount of settlement of in-situ soil due to placement of fill for temporary bridge using the Hough Method and SPT data.

References

1. 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition with Interims

Soil Properties and Geotechnical Inputs

$N_{60} := 9$ N_{60} value from SPT of in-situ Silt encountered in borings
 $\gamma_w := 62.4 \cdot \text{pcf}$ Unit weight of water.

Soil properties taken from Table 6 of NAVFAC, based on Soft to medium stiff SILT from SPT information

$\gamma_{\text{SILT}} := 100 \text{ pcf}$ Wet Unit weight of silt
 $C_{\text{Prime}} := 25$ C_{Prime} = Bearing Capacity Factor based on Corrected SPT
 Based on Table 10.6.2.4.2-1

Assume groundwater table at the top of the in-situ soil (Approx. El. 228 measured in BB-CBS-103)

Embankment Dimensions

$H_{\text{fill}} := 11 \cdot \text{ft}$ Max fill height occurs at Temporary Bridge embankment, El. 236 - GSEL. 225 = 11 feet.
 Silt is assumed at ground surface (encountered at EL. 225 in BB-CBS-103)

$\gamma_{\text{FILL}} := 125 \cdot \text{pcf}$ Total unit weight of soil based on Granular Borrow for Underwater Backfill - BDG Type 4 soil

$$\Delta H := H_c \cdot \left(\frac{1}{C_{\text{Prime}}} \right) \cdot \log \left[\frac{(\sigma_{v0} + \Delta \sigma_v)}{\sigma_{v0}} \right] \quad \text{Settlement equation based on Hough Method}$$

ΔH = settlement

C_{prim} = Tangent value of constrained modulus of the soil (stress units – the same units as $\Delta \sigma_v$)

σ_{v0} = in-situ effective stress

$\Delta \sigma_v$ = Increase in effective stress within the sublayer due to the load.

H_c = sublayer thickness

Based on boring BB-CBS-103 at abutment 2, a 23' thick layer of soft to medium stiff SILT is used in the settlement analysis



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 Fax 207-879-0099

Engineers and
 Scientists

JOB: 09.0026000.00 Hall Bridge
 SUBJECT: Settlement - Fill Placed on Existing Soil
 SHEET: 2 OF 5
 CALCULATED BY N. Williams 1/19/21
 CHECKED BY C. Snow 1/22/21

$$H_c := 23 \cdot \text{ft}$$

Sublayer thickness

$$d := \frac{H_c}{2}$$

d = depth of the midpoint of the layer

$$\sigma_{vo} := d \cdot (\gamma_{\text{SILT}} - \gamma_w) = 432.4 \cdot \text{psf}$$

Initial vertical effective stress - at midpoint of the layer

$$\Delta\sigma_v := H_{\text{fill}} \cdot \gamma_{\text{FILL}} = 1375 \cdot \text{psf}$$

Change in vertical effective stress due to the fill

$$i := \frac{45 \cdot \text{ft}}{45 \cdot \text{ft} + d} = 0.80$$

Influence Factor assuming a 1H:2V stress distribution starting at top of Silt, Assume 45' average width of temp embankment

$$\Delta H := i \cdot H_c \cdot \left(\frac{1}{C_{\text{Prime}}} \right) \cdot \log \left[\frac{(\sigma_{vo} + \Delta\sigma_v)}{\sigma_{vo}} \right] = 5.5 \cdot \text{in}$$

Settlement of Silt Layer

$$S_e = \sum_{i=1}^n \Delta H_i \quad (10.6.2.4.2-2)$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log \left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o} \right) \quad (10.6.2.4.2-3)$$

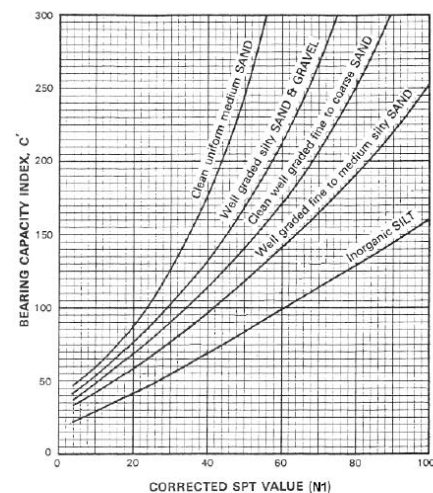
where:

- n = number of soil layers within zone of stress influence of the footing
- ΔH_i = elastic settlement of layer i (ft)
- H_c = initial height of layer i (ft)
- C' = bearing capacity index from Figure 10.6.2.4.2-1 (dim)
- σ'_o = initial vertical effective stress at the midpoint of layer i (ksf)
- $\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer i (ksf)

In Figure 10.6.2.4.2-1, N_1 shall be taken as N_{160} , Standard Penetration Resistance, N (blows/ft), corrected for overburden pressure as specified in Article 10.4.6.2.4.

10-58

AASHTO



Reference: Hough, "Compressibility as a Basis for Soil Bearing Values" ASCE 1959

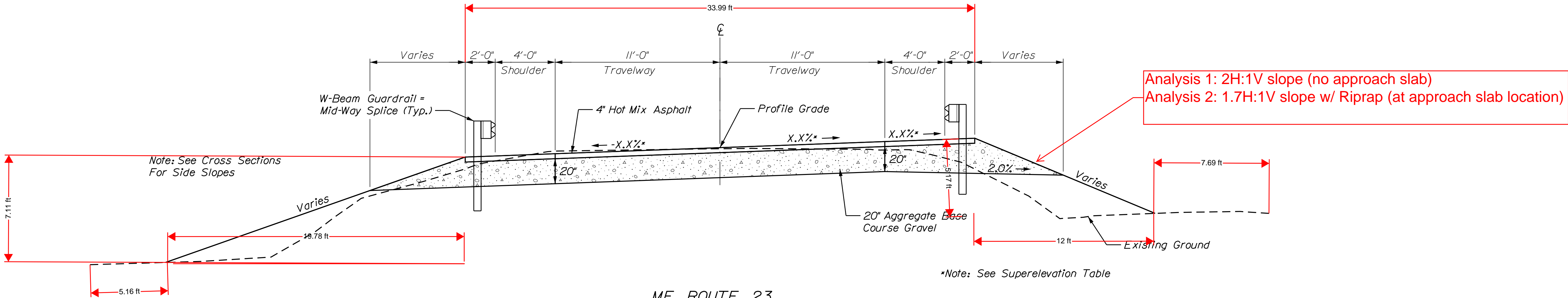
Figure 10.6.2.4.2-1—Bearing Capacity Index versus Corrected SPT (modified from Cheney and Chassie, 2000, after Hough, 1959)

Global
Stability

VHB - Typical Cross Section for use in Lateral global stability Analysis

TRAFFIC DATA

1. The pavement, base and subbase depths as shown on the plans are intended to be nominal.
2. When superelevation exceeds the slope of the low side shoulder, the low side shoulder shall have the same slope as the travelway.
3. Crowns for both normal and superelevation sections for all courses of subbase and pavement shall be straight.
4. The algebraic difference between the shoulder and travelway cross slopes "rollover" shall not exceed 8%.
5. The stationing shown under each typical is approximate.



ME ROUTE 23
RECONSTRUCTION TYPICAL SECTION

Sta. 102+25 to 102+84
Sta. 104+00 to 104+75

ROUTE 23				
SUPERELEVATION TABLE				
START				
LT. SHOULDER	LT. TRAVELWAY	STATION	RT. TRAVELWAY	RT. SHOULDER
MATCH	MATCH	101+00	MATCH	MATCH
-2.0	-2.0	101+50	-0.8	-0.8
-2.0	-2.0	102+00	1.6	1.6
-4.0	-4.0	102+50	4.0	4.0
-5.5	-5.5	102+84	5.5	5.5
BRIDGE				
-5.5	-5.5	104+00	5.5	5.5
-6.5	-6.5	104+50	5.5	2.5
-8.5	-8.5	105+00	3.2	-0.5
-11.0	-11.0	105+50	2.0	-2.0
MATCH	MATCH	106+00	MATCH	MATCH
END				

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	STP-2222(600)	
	BRIDGE NO. 3159	WIN 22226.00
	BRIDGE PLANS	

PRELIMINARY
NOT FOR
CONSTRUCTION

PROJ. MANAGER	M. KERSBERGEN	BY	DATE
DESIGN-DETAILED	ECF	BMD	10/22/18
CHECKED-REVIEWED	ECF	RSBLUNT	10/22/18
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HALL BRIDGE ROUTE 23 (HARTLAND ROAD)
OVER BLACK STREAM
CANAAN ME SOMERSET COUNTY

TYPICAL SECTIONS

SHEET NUMBER

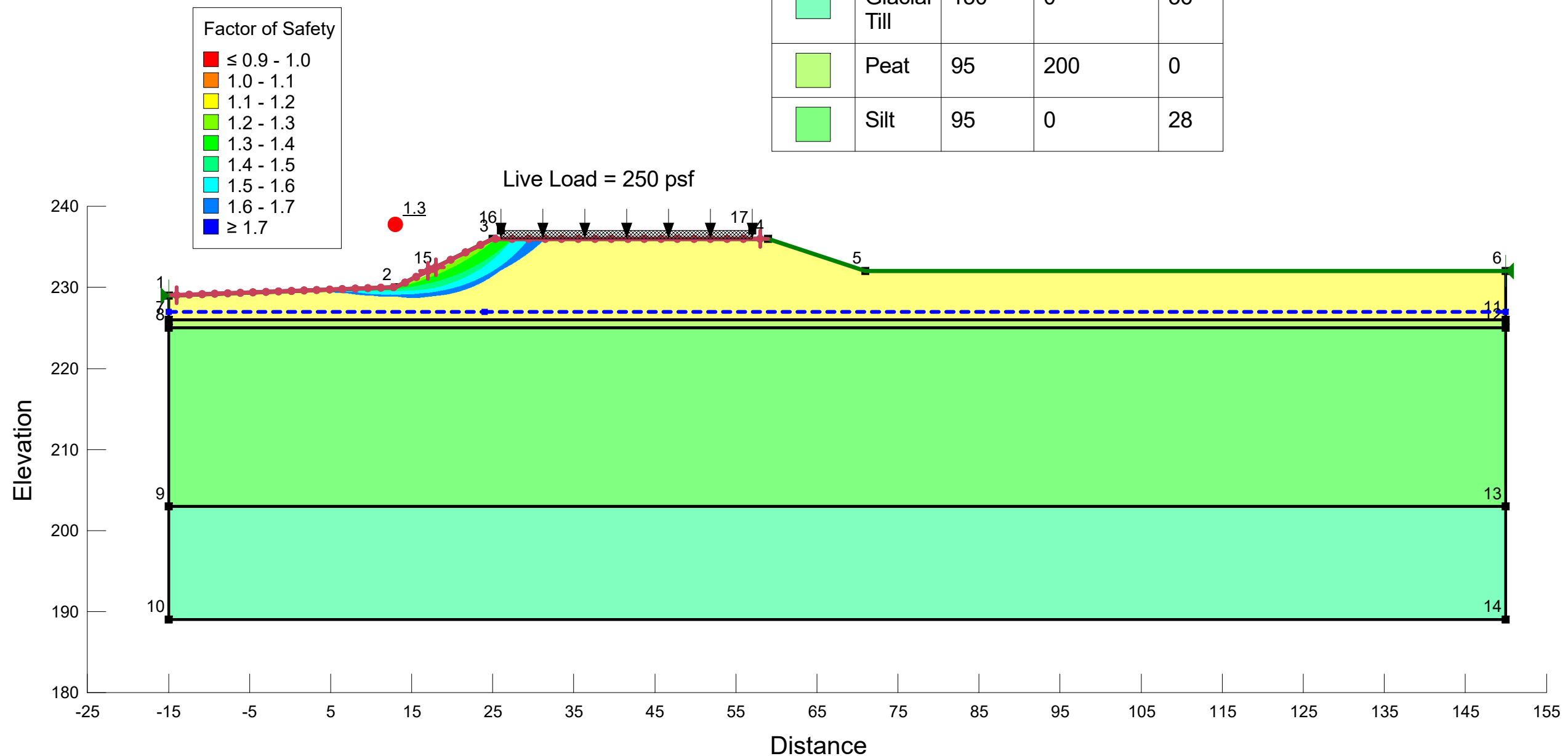
4

OF 7

Analysis 1

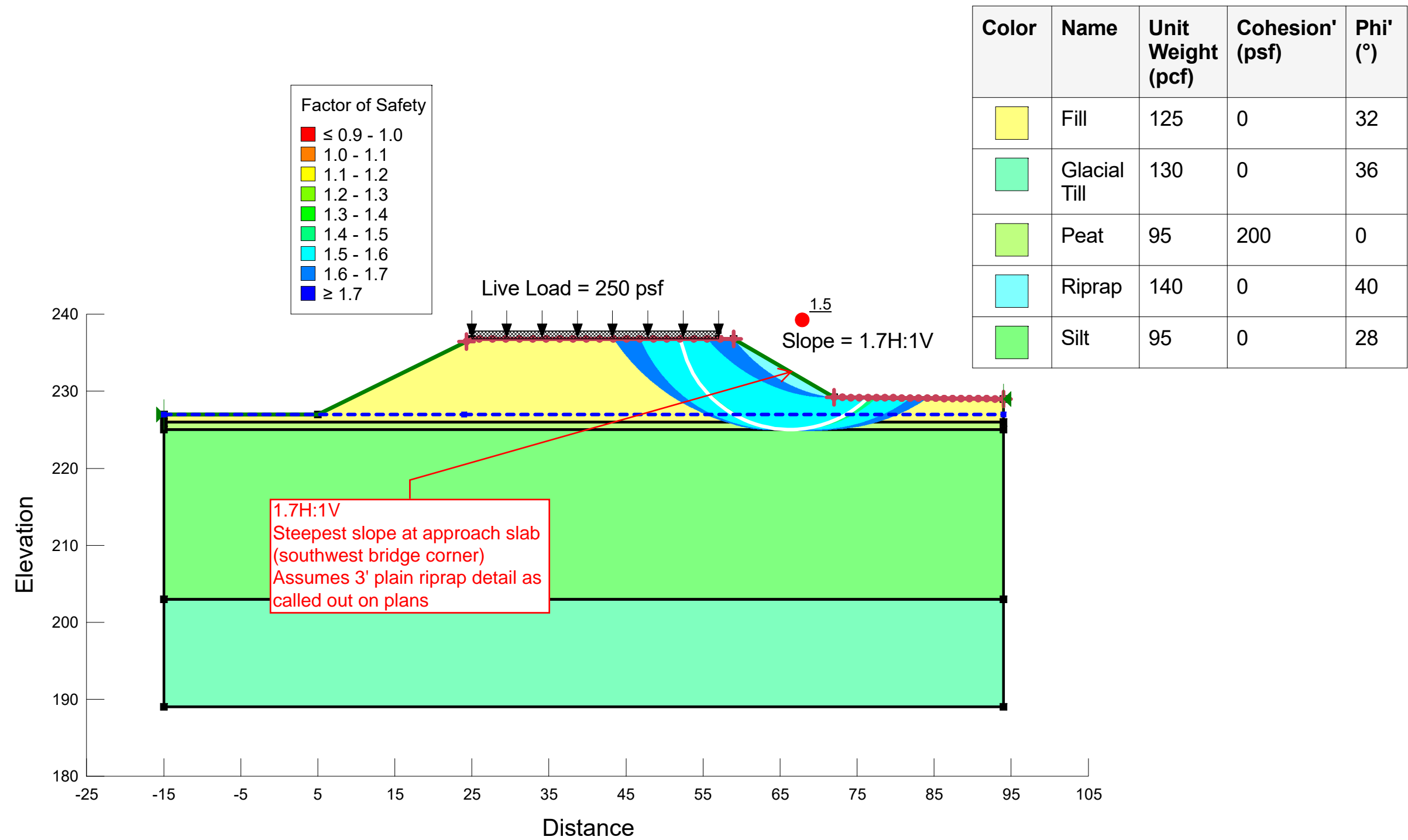
Lateral Global Stability for Approach (Typ. Cross Section) beyond approach slab (no riprap).

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
<div></div>	Fill	125	0	32
<div></div>	Glacial Till	130	0	36
<div></div>	Peat	95	200	0
<div></div>	Silt	95	0	28



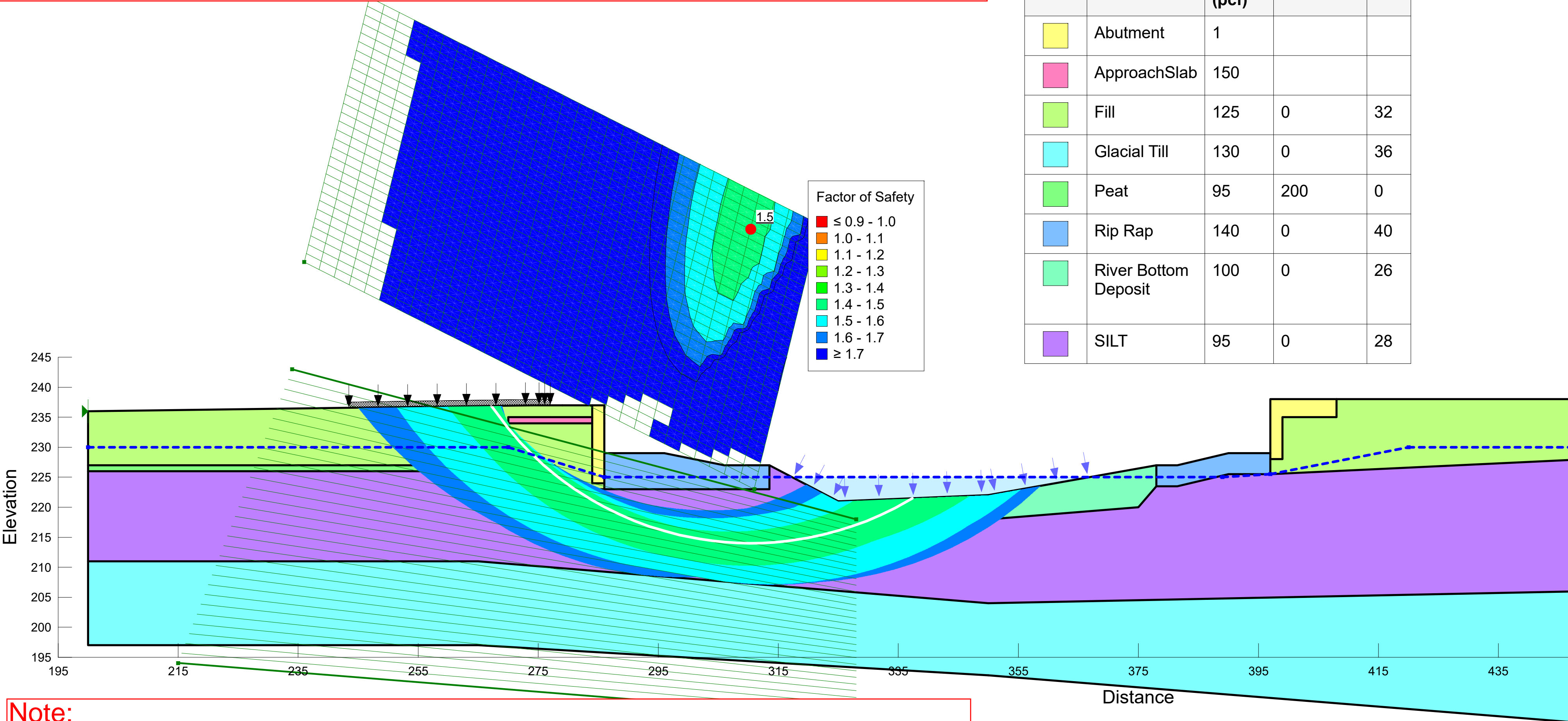
Analysis 2

Lateral Global Stability for Approach at approach slab (with riprap).



Global Stability Analysis

Longitudinal Global Stability - Through center of Abutment

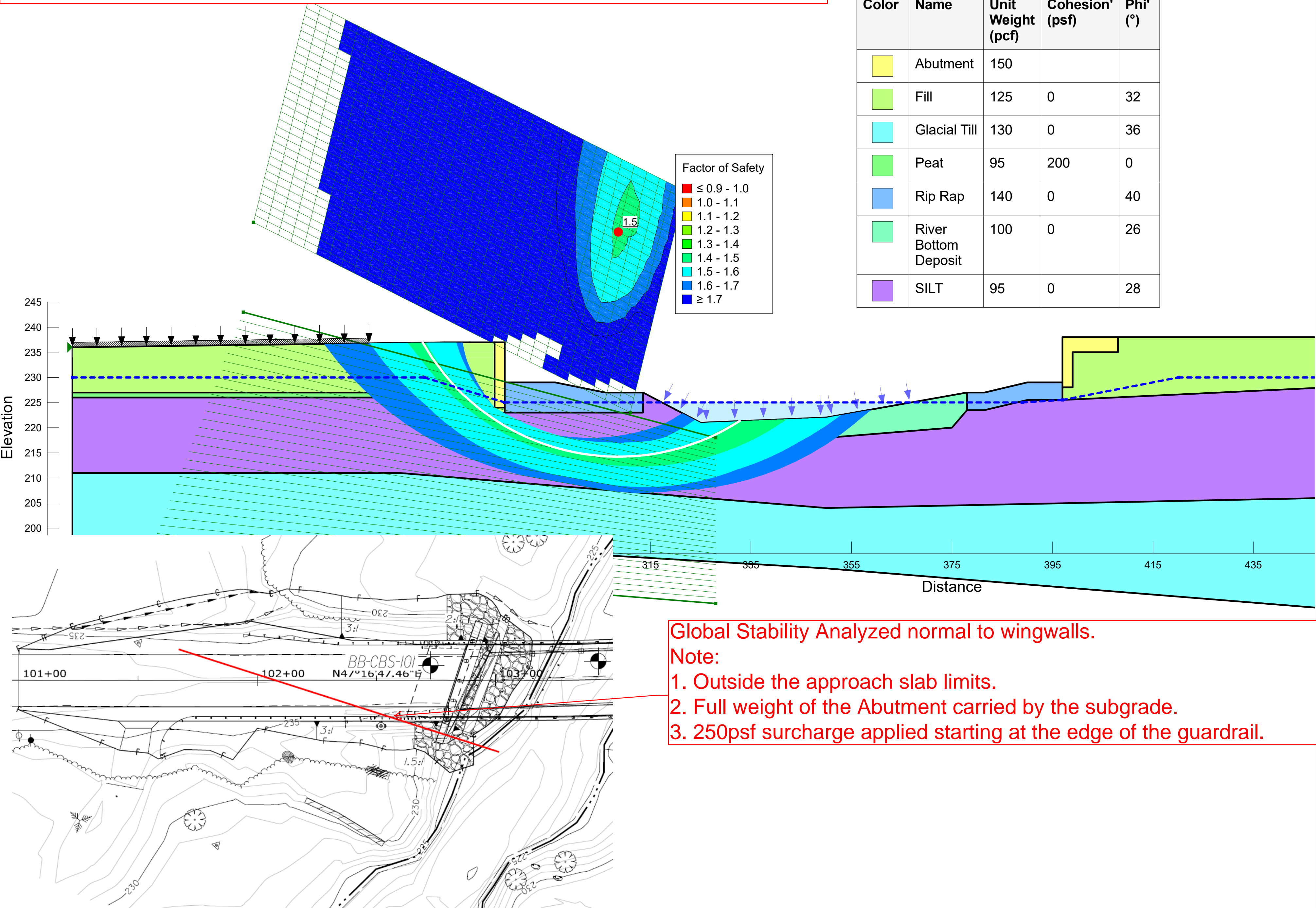


Note:

1. Doesn't take into account added Global Stability Shear resistance from Piles.
2. Abutment weight and the surcharge on 1/2 of the approach slab was assumed to be carried by Piles.

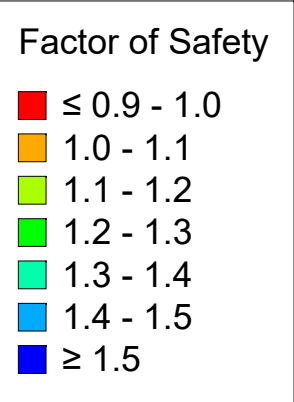
Global Stability Analysis

Longitudinal Global Stability - Through wingwall

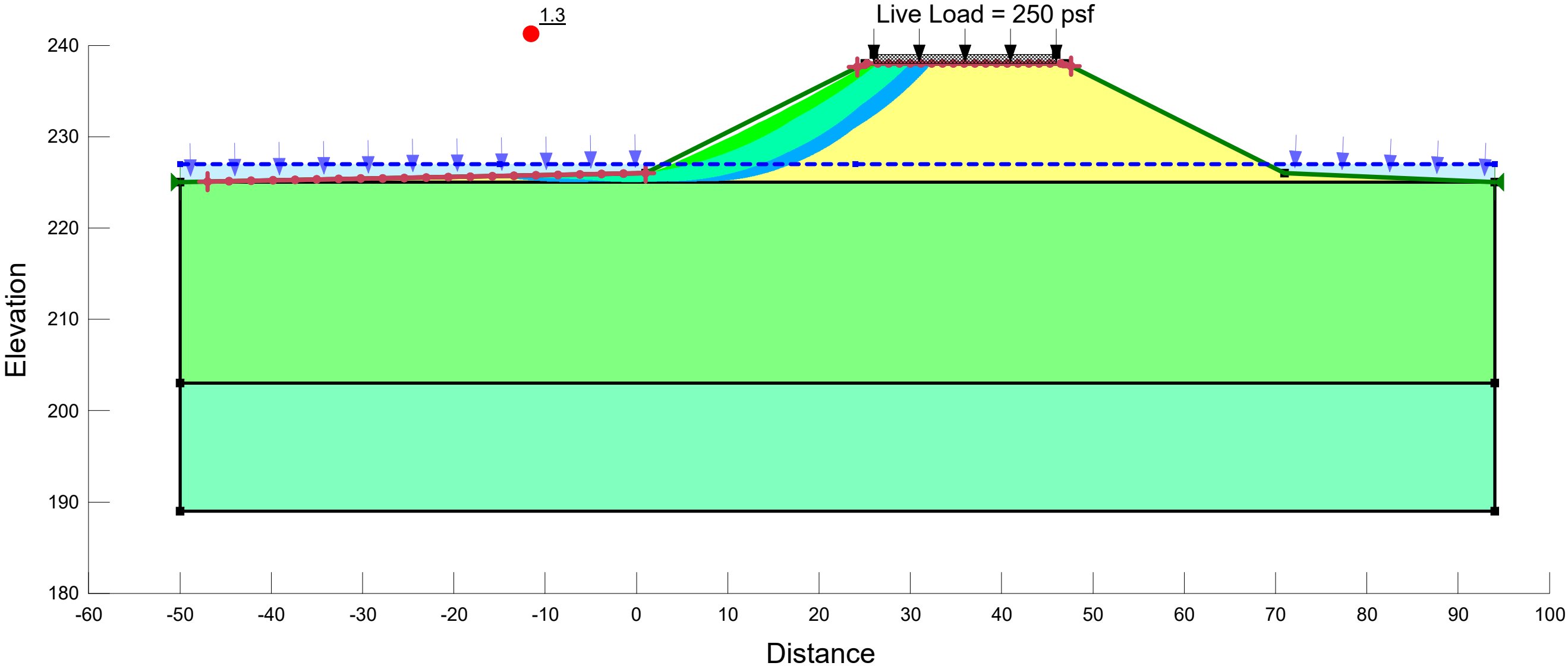


Global Stability Analyzed normal to wingwalls.
Note:
1. Outside the approach slab limits.
2. Full weight of the Abutment carried by the subgrade.
3. 250psf surcharge applied starting at the edge of the guardrail.

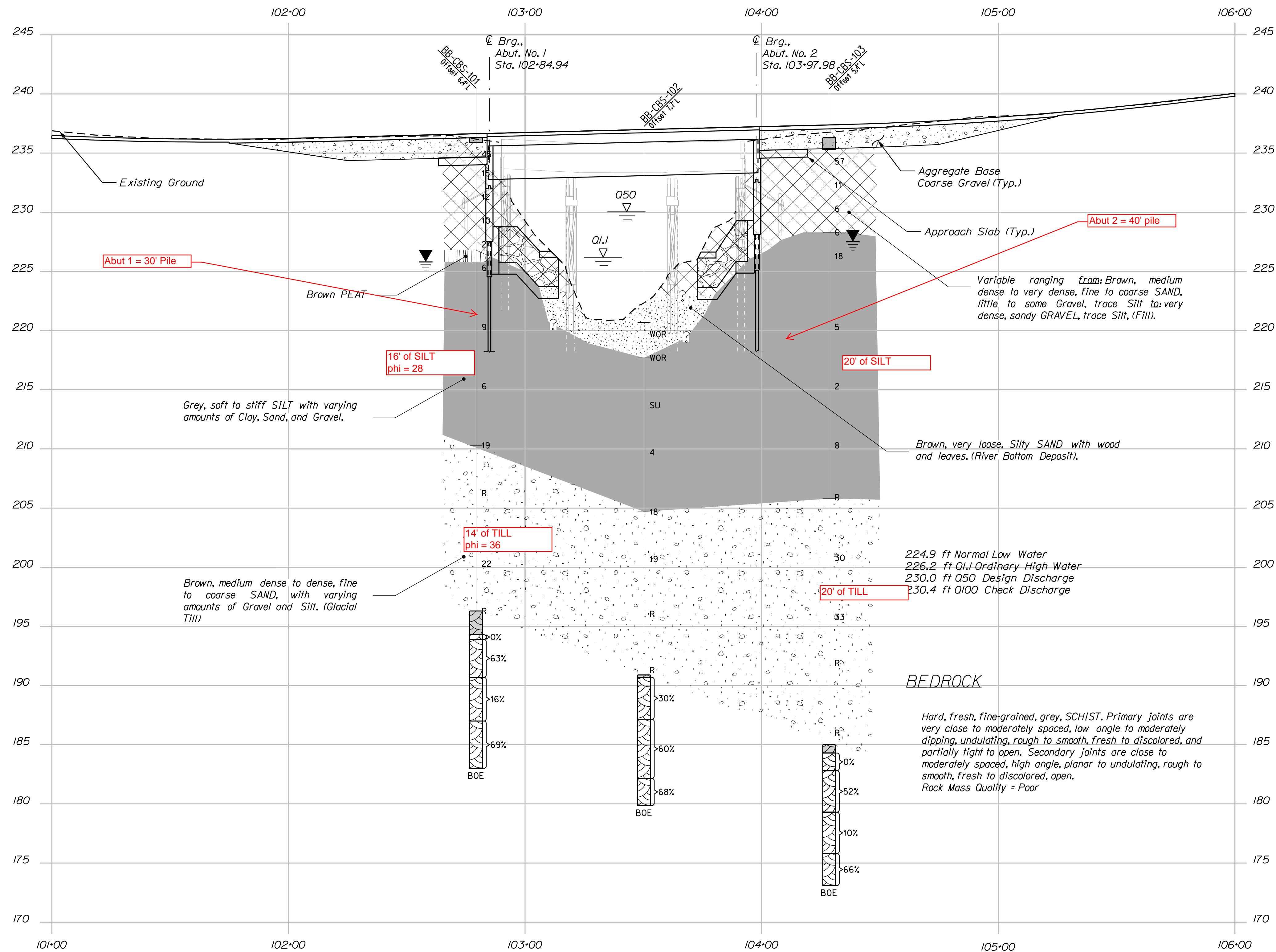
Lateral Global Stability for Assumed Temp Bridge Approach (2H:1V Side slopes)



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Fill	125	0	32
	Glacial Till	130	0	36
	Silt	95	500	0



Pile Analyses



NOTES

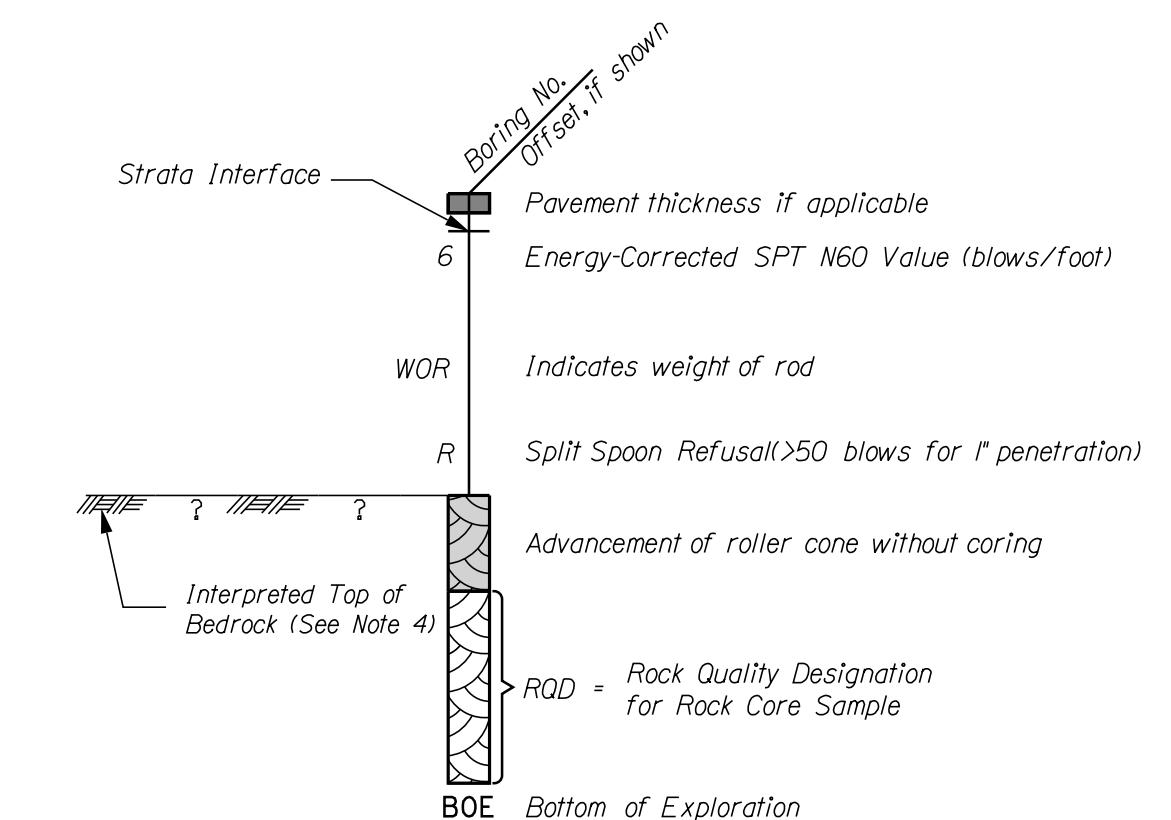
1) Base map developed from electronic files provided by VHB dated January 8, 2019 (Files included Profile_HWY.dgn and Z_Profile.dgn)

2) The as drilled locations of the test borings were surveyed by a MaineDOT survey crew and supplied to GZA.

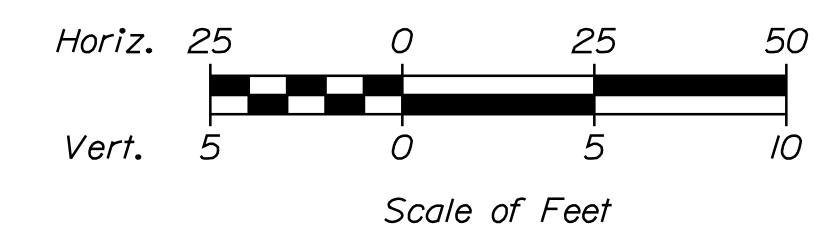
3) BB-CBS-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between October 19 and November 1, 2018.

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

INTERPRETIVE SUBSURFACE PROFILE LEGEND

 *Encountered Groundwater Level*

HALL BRIDGE PROFILE



PRELIMINARY
NOT FOR CONSTRUCTION

PREPARED BY:



<div> <div> <div>STATE OF MICHIGAN</div> <div>DEPARTMENT OF TRANSPORTATION</div> </div> <div> <div> <div>ROUTE 23 OVER THE BLACK STREAM CANAAAN</div> <div>OXFORD COUNTY</div> </div> <div> <div>INTEPRETIVE SUBSURFACE PROFILE</div> <div>STP-2222(600)</div> </div> </div> </div>	DESIGN-DETAILED		NW
	CHECKED-REVIEWED	RB	CLS
	DESIGN2-DETAILED2		
	DESIGN3-DETAILED3		
	REVISIONS 1		
	REVISIONS 2		
	REVISIONS 3		
	REVISIONS 4		
	FIELD CHANGES		
	<div> <div>SHEET NUMBER</div> <div>BRIDGE NO. 3159</div> <div>BRIDGE PLANS</div> </div>		

GZA
calc: N Williams 11-06-20
check: C Snow 01-22-21

Computations

Project: Canaan Hall Bridge
Location: Canaan
Calculated by: JRK
Checked by: BGP
Title: Superstructure Loads

Project #: 55216.01
Sheet:
Date: 6/15/2020
Date: 6/17/2020

Pile Loads

Number of Piles	Pile Spacing (ft.)	Service Axial Load (kip)	factored	Thermal Deflections	
			Strength Axial Load (kip)	Δ_T Exp. (in.)	Δ_T Con. (in.)
8	5	139	212	0.353	-0.529
7	5.75	158	243	0.353	-0.529
6	7	183	279	0.353	-0.529
5	8.75	216	329	0.353	-0.529

of Piles with Strength Load and deflections provided by VHB

Based on conversations with Carl Ayers (VHB) - wanted to use 5 pile configuration using 14x89.
Step 1 = Lpile to make sure not overstressing pile with 0.529" deflection.
Step 2 = Run WEAP analysis to make sure we can get the nominal geotech resistance (strength + downdrag)/0.65

Downdrag Analysis



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<http://www.gza.com>

*Engineers and
 Scientists*

Hall Bridge -
 JOB: 09.0026000.00
 SUBJECT: Downdrag, Abutments 1 and 2
 SHEET: 1
 CALCULATED BY: N. Williams, 5/2/2019
 CHECKED BY: C. Snow 01/22/21

OBJECTIVE: Evaluate downdrag on HP14x89 piles supporting abutments.

EVALUATION: The settlement evaluation assumes that >0.4 inches of total settlement may occur adjacent to the new abutments under the stress increase imposed by new fills. We anticipate a portion of the settlement will occur as the load is applied. Considering the variability of the deposits, it's possible that greater than 0.4 inches of settlement may occur at some locations within the new abutments after pile installation. Therefore the piles should be designed for downdrag loading from movement of the fill and silt relative to the piles.

LFRD Section C3.11.8 indicates that downdrag loading should be calculated using either the alpha, lambda, or beta method for clay and an effective stress method for sand. Based on historical Maine practice, the beta method was used for Silt. Beta values were selected for Silt equal to 0.23, It was assumed that the entire thickness of the Silt layer will contribute to downdrag.

A load factor of 1.0 was applied to the nominal downdrag load

CONCLUSION: The nominal and factored downdrag load is estimated at 52 kips which will be assumed at both abutments. Factored pile loads and required driving resistance are summarized below.

Down Drag Calculation:

Layer	Thickness (ft)	Unit Weight (pcf)	β	Effective Stress at Midpoint of Layer (psf)	Unit Side Friction (ksf)	Downdrag (kips)
Subgrade	6	125	--	375	NA (Above pile)	--
Existing Fill	8	125	--	1250	NA (Above pile)	--
Silt	22.5	95	0.23	2117	0.49	52.1
Total						52

Abutment 1/2		
Maximum factored pile load (kips)	329	
Factored Downdrag Load (kips)	52	(Load factor=1.0)
Required Factored Driving Resistance (kips)	381	
Required Nominal Driving Resistance (kips)	586	(Resistance factor=0.65)

Lpile Analysis

GZA

calc: N Williams 11-06-20

check: C Snow 01-22-21

Abutment 1
14x89
30' length
0.529" of deflection
Total Stress = Approx. 40 ksi
Fixity = approx. 24'

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=====

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is a violation of the software license agreement.

Files Used for Analysis

Path to file locations:
\\09 Jobs\0026000s\09.0026000.00 - MEDOT - Hall Bridge - Canaan\Work\Calcs\LPILE\Abut 1\14x89\0.529InchDefl\

Name of input data file:
Abut1_14x89.lp11

Name of output report file:
Abut1_14x89.lp11

Name of plot output file:
Abut1_14x89.lp11

Name of runtime message file:
Abut1_14x89.lp11

Page 1

Abut1_14x89.lp11o

Date and Time of Analysis

Date: July 8, 2020 Time: 8:31:49

Problem Title

Project Name: Hall Bridge

Job Number: 09.0026000.00

Client:

Engineer: N.Williams

Description:

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

Page 2

Abut1_14x89.lp11o

- 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.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.7000
2	30.000	14.7000

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
Page 3

Abut1_14x89.lp11o

Pile width = 13.800000 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 2 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 = 16.000000 ft
Effective unit weight at top of layer = 32.600000 pcf
Effective unit weight at bottom of layer = 32.600000 pcf
Friction angle at top of layer = 28.000000 deg.
Friction angle at bottom of layer = 28.000000 deg.
Subgrade k at top of layer = 15.000000 pci
Subgrade k at bottom of layer = 15.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 16.000000 ft
Distance from top of pile to bottom of layer = 30.000000 ft
Effective unit weight at top of layer = 67.600000 pcf
Effective unit weight at bottom of layer = 67.600000 pcf
Friction angle at top of layer = 35.000000 deg.
Friction angle at bottom of layer = 35.000000 deg.
Subgrade k at top of layer = 95.000000 pci
Subgrade k at bottom of layer = 95.000000 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	Angle of Friction deg.	kpy pci
1	Sand	0.00	32.6000	28.0000	15.0000
	(Reese, et al.)	16.0000	32.6000	28.0000	15.0000
2	Sand	16.0000	67.6000	35.0000	95.0000
	(Reese, et al.)	30.0000	67.6000	35.0000	95.0000

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 = 0.529000 in	S = 0.0000 in/in	381000.	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	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	45.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.811550 sq. in.
Moment of Inertia	=	325.837265 in^4
Elastic Bending Stiffness	=	9449281. kip-in^2
Plastic Modulus, Z	=	67.636247in^3
Plastic Moment Capacity = Fy Z	=	3044.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1161.520 kips
Nominal Axial Tensile Capacity	=	-1161.520 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	381.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 381.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000386	36.4402599	9448389.	139.3242518	15.5746835	
0.00000771	72.8805198	9448389.	73.3371259	16.3885333	
0.00001157	109.3207797	9448389.	51.3414173	17.2023828	
0.00001543	145.7610396	9448389.	40.3435630	18.0162328	
0.00001928	182.2012995	9448389.	33.7448504	18.8300824	
0.00002314	218.6415594	9448389.	29.3457086	19.6439322	
0.00002700	255.0818192	9448389.	26.2034645	20.4577819	
0.00003085	291.5220791	9448389.	23.8467815	21.2716317	
0.00003471	327.9623390	9448389.	22.0138058	22.0854813	
0.00003857	364.4025989	9448389.	20.5474252	22.8993312	
0.00004242	400.8428588	9448389.	19.3476593	23.7131807	
0.00004628	437.2831187	9448389.	18.3478543	24.5270305	
0.00005014	473.7233786	9448389.	17.5018655	25.3408802	
0.00005399	510.1636385	9448389.	16.7767323	26.1547300	
0.00005785	546.6038984	9448389.	16.1482835	26.9685796	
0.00006171	583.0441583	9448389.	15.5983907	27.7824293	
0.00006557	619.4844182	9448389.	15.1131913	28.5962791	
0.00006942	655.9246781	9448389.	14.6819029	29.4101288	
0.00007328	692.3649380	9448389.	14.2960133	30.2239786	
0.00007714	728.8051978	9448389.	13.9487126	31.0378283	
0.00008099	765.2454577	9448389.	13.6344882	31.8516780	
0.00008485	801.6857176	9448389.	13.3488296	32.6655277	
0.00008871	838.1259775	9448389.	13.0880109	33.4793775	
0.00009256	874.5662374	9448389.	12.8489272	34.2932272	
0.00009642	911.0064973	9448389.	12.6289701	35.1070769	
0.0001003	947.4467572	9448389.	12.4259328	35.9209267	
0.0001041	983.8870171	9448389.	12.2379353	36.7347764	
0.0001080	1020.	9448389.	12.0633661	37.5486261	
0.0001118	1057.	9448389.	11.9008363	38.3624759	
0.0001157	1093.	9448389.	11.7491417	39.1763255	
0.0001196	1130.	9448389.	11.6072339	39.9901753	
0.0001234	1166.	9448389.	11.4741954	40.8040250	
0.0001273	1203.	9448389.	11.3492198	41.6178747	
0.0001311	1239.	9448389.	11.2315956	42.4317245	
0.0001350	1275.	9448389.	11.1206929	43.2455742	

Page 7

0.0001388	1312.	9448389.	11.0159514	44.0594239	
0.0001427	1348.	9448389.	10.9168717	44.8732736	
0.0001466	1384.	9441578.	10.8242594	45.0000000	Y
0.0001504	1418.	9425583.	10.7381862	45.0000000	Y
0.0001581	1482.	9373599.	10.5829769	45.0000000	Y
0.0001658	1542.	9300644.	10.4474242	45.0000000	Y
0.0001736	1599.	9212372.	10.3285448	45.0000000	Y
0.0001813	1652.	9113345.	10.2238420	45.0000000	Y
0.0001890	1702.	9007258.	10.1312052	45.0000000	Y
0.0001967	1750.	8897099.	10.0488321	45.0000000	Y
0.0002044	1796.	8784041.	9.9754974	45.0000000	Y
0.0002121	1839.	8670126.	9.9098731	45.0000000	Y
0.0002198	1881.	8555747.	9.8511508	45.0000000	Y
0.0002275	1921.	8442555.	9.7982592	45.0000000	Y
0.0002353	1960.	8330891.	9.7505498	45.0000000	Y
0.0002430	1997.	8220673.	9.7075685	45.0000000	Y
0.0002507	2034.	8112600.	9.6686762	45.0000000	Y
0.0002584	2069.	8006993.	9.6333869	45.0000000	Y
0.0002661	2103.	7903982.	9.6013098	45.0000000	Y
0.0002738	2137.	7803672.	9.5720954	45.0000000	Y
0.0002815	2170.	7705938.	9.5455040	45.0000000	Y
0.0002893	2201.	7610685.	9.5213184	45.0000000	Y
0.0002970	2233.	7518212.	9.4991957	45.0000000	Y
0.0003047	2263.	7426854.	9.4784117	45.0000000	Y
0.0003124	2291.	7333932.	9.4579312	45.0000000	Y
0.0003201	2317.	7239380.	9.4380631	45.0000000	Y
0.0003278	2342.	7144504.	9.4185122	45.0000000	Y
0.0003355	2366.	7049978.	9.3991848	45.0000000	Y
0.0003433	2387.	6954953.	9.3804804	45.0000000	Y
0.0003510	2408.	6860717.	9.3617415	45.0000000	Y
0.0003587	2427.	6767066.	9.3435941	45.0000000	Y
0.0003664	2446.	6675020.	9.3256747	45.0000000	Y
0.0003741	2463.	6583982.	9.3083194	45.0000000	Y
0.0003818	2480.	6493990.	9.2907889	45.0000000	Y
0.0003895	2495.	6405305.	9.2739970	45.0000000	Y
0.0003972	2510.	6318801.	9.2573475	45.0000000	Y
0.0004050	2524.	6232812.	9.2409172	45.0000000	Y
0.0004127	2538.	6149058.	9.2249176	45.0000000	Y
0.0004204	2550.	6066812.	9.2089116	45.0000000	Y
0.0004281	2563.	5985796.	9.1935006	45.0000000	Y
0.0004358	2574.	5906787.	9.1782401	45.0000000	Y
0.0004435	2585.	5829217.	9.1629758	45.0000000	Y
0.0004512	2596.	5753220.	9.1484327	45.0000000	Y
0.0004590	2606.	5678577.	9.1337797	45.0000000	Y
0.0004668	2643.	5396200.	9.0778694	45.0000000	Y
0.0005207	2675.	5136759.	9.0253558	45.0000000	Y

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0.0005515	2702.	4898387.	8.9758553	45.0000000	Y
0.0005824	2725.	4679382.	8.9290907	45.0000000	Y
0.0006132	2746.	4477690.	8.8852574	45.0000000	Y
0.0006441	2764.	4291629.	8.8437143	45.0000000	Y
0.0006749	2780.	4119497.	8.8042909	45.0000000	Y
0.0007058	2795.	3959920.	8.7668005	45.0000000	Y
0.0007366	2808.	3811604.	8.7312830	45.0000000	Y
0.0007675	2820.	3673715.	8.6972526	45.0000000	Y
0.0007984	2830.	3545098.	8.6651982	45.0000000	Y
0.0008292	2840.	3425079.	8.6348131	45.0000000	Y
0.0008601	2849.	3312378.	8.6052605	45.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	381.0000000000	2849.

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	Equivalent Top Depth Below Grnd Surf	Same Layer Type As Layer	Layer is Rock or is Below	F0 Integral for Layer	F1 Integral for Layer
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	ft	ft	Above	Rock Layer	Abut1_14x89.1p110 lbs	lbs
1	0.00	0.00	N.A.	No	0.00	74542.
2	16.0000	13.0288	Yes	No	74542.	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 = 0.529000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 381000.0 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	0.5290	-1105486.	15387.	0.00	39698.	9.45E+09	0.00	0.00	0.00
0.3000	0.5282	-1049866.	15357.	-4.11E-04	38443.	9.45E+09	-7.2924	49.6982	0.00
0.6000	0.5260	-993792.	15316.	-8.00E-04	37178.	9.45E+09	-15.5117	106.1549	0.00
0.9000	0.5225	-937400.	15244.	-0.00117	35906.	9.45E+09	-24.1392	166.3233	0.00
1.2000	0.5176	-880830.	15142.	-0.00151	34630.	9.45E+09	-32.7404	227.7002	0.00
1.5000	0.5116	-824225.	15009.	-0.00184	33353.	9.45E+09	-41.0662	288.9843	0.00
1.8000	0.5044	-767721.	14848.	-0.00214	32079.	9.45E+09	-48.6260	347.0574	0.00
2.1000	0.4962	-711446.	14659.	-0.00242	30809.	9.45E+09	-56.0277	406.5260	0.00
2.4000	0.4869	-655525.	14446.	-0.00268	29548.	9.45E+09	-62.4850	461.9585	0.00
2.7000	0.4768	-600072.	14211.	-0.00292	28297.	9.45E+09	-68.2191	515.0488	0.00
3.0000	0.4659	-545189.	13957.	-0.00314	27059.	9.45E+09	-72.6717	561.5464	0.00
3.3000	0.4542	-490963.	13687.	-0.00334	25836.	9.45E+09	-77.4096	613.5449	0.00
3.6000	0.4418	-437483.	13401.	-0.00352	24629.	9.45E+09	-81.3320	662.6647	0.00
3.9000	0.4289	-384830.	13099.	-0.00367	23442.	9.45E+09	-86.5773	726.7144	0.00
4.2000	0.4154	-333097.	12778.	-0.00381	22275.	9.45E+09	-91.8255	795.7903	0.00
4.5000	0.4015	-282380.	12439.	-0.00393	21131.	9.45E+09	-96.0833	861.6104	0.00
4.8000	0.3871	-232760.	12088.	-0.00403	20011.	9.45E+09	-99.1833	922.3327	0.00
5.1000	0.3725	-184305.	11721.	-0.00410	18918.	9.45E+09	-104.6220	1011.	0.00

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5.4000	0.3576	-137109.	11332.	-0.00417	17854.	9.45E+09	-111.3914	1121.	0.00
5.7000	0.3425	-91285.	10920.	-0.00421	16820.	9.45E+09	-117.7769	1238.	0.00
6.0000	0.3273	-46939.	10485.	-0.00424	15820.	9.45E+09	-123.7110	1361.	0.00
6.3000	0.3120	-4173.	10028.	-0.00425	14855.	9.45E+09	-130.0152	1500.	0.00
6.6000	0.2967	36911.	9548.	-0.00424	15593.	9.45E+09	-136.6298	1658.	0.00
6.9000	0.2815	76205.	9045.	-0.00422	16480.	9.45E+09	-142.9582	1828.	0.00
7.2000	0.2663	113606.	8520.	-0.00418	17323.	9.45E+09	-148.9787	2014.	0.00
7.5000	0.2514	149017.	7973.	-0.00413	18122.	9.45E+09	-154.6747	2215.	0.00
7.8000	0.2366	182346.	7407.	-0.00407	18874.	9.45E+09	-160.0115	2435.	0.00
8.1000	0.2221	213505.	6822.	-0.00399	19577.	9.45E+09	-164.8618	2673.	0.00
8.4000	0.2078	242417.	6221.	-0.00391	20229.	9.45E+09	-169.1759	2930.	0.00
8.7000	0.1939	269009.	5605.	-0.00381	20829.	9.45E+09	-172.9191	3210.	0.00
9.0000	0.1804	293219.	4977.	-0.00370	21375.	9.45E+09	-176.0600	3513.	0.00
9.3000	0.1673	314995.	4338.	-0.00359	21866.	9.45E+09	-178.5708	3843.	0.00
9.6000	0.1546	334291.	3692.	-0.00346	22302.	9.45E+09	-180.4273	4201.	0.00
9.9000	0.1424	351075.	3041.	-0.00333	22680.	9.45E+09	-181.6091	4592.	0.00
10.2000	0.1306	365322.	2386.	-0.00319	23001.	9.45E+09	-182.0997	5019.	0.00
10.5000	0.1194	377017.	1731.	-0.00305	23265.	9.45E+09	-181.8863	5485.	0.00
10.8000	0.1086	386158.	1078.	-0.00291	23472.	9.45E+09	-180.9604	5997.	0.00
11.1000	0.09843	392753.	429.0540	-0.00276	23620.	9.45E+09	-179.3171	6558.	0.00
11.4000	0.08877	396818.	-212.2366	-0.00261	23712.	9.45E+09	-176.9554	7177.	0.00
11.7000	0.07965	398382.	-832.6776	-0.00246	23747.	9.45E+09	-167.7341	7582.	0.00
12.0000	0.07107	397564.	-1411.	-0.00231	23729.	9.45E+09	-153.5143	7776.	0.00
12.3000	0.06304	394549.	-1938.	-0.00216	23661.	9.45E+09	-139.5758	7970.	0.00
12.6000	0.05555	389519.	-2417.	-0.00201	23547.	9.45E+09	-125.9976	8165.	0.00
12.9000	0.04860	382653.	-2846.	-0.00186	23392.	9.45E+09	-112.8514	8359.	0.00
13.2000	0.04217	374123.	-3230.	-0.00171	23200.	9.45E+09	-100.2012	8554.	0.00
13.5000	0.03626	364100.	-3569.	-0.00157	22974.	9.45E+09	-88.1037	8748.	0.00
13.8000	0.03084	352745.	-3865.	-0.00144	22718.	9.45E+09	-76.6080	8942.	0.00
14.1000	0.02591	340212.	-4122.	-0.00131	22435.	9.45E+09	-65.7553	9137.	0.00
14.4000	0.02144	326650.	-4340.	-0.00118	22129.	9.45E+09	-55.5795	9331.	0.00
14.7000	0.01743	312196.	-4523.	-0.00106	21803.	9.45E+09	-46.1069	9526.	0.00
15.0000	0.01384	296982.	-4673.	-9.40E-04	21460.	9.45E+09	-37.3567	9720.	0.00
15.3000	0.01065	281128.	-4793.	-8.30E-04	21102.	9.45E+09	-29.3407	9914.	0.00
15.6000	0.00786	264747.	-4886.	-7.26E-04	20733.	9.45E+09	-22.0638	10109.	0.00
15.9000	0.00542	247942.	-4954.	-6.29E-04	20354.	9.45E+09	-15.5243	10303.	0.00
16.2000	0.00333	230806.	-5086.	-5.37E-04	19967.	9.45E+09	-8.0450	10490.	0.00
16.5000	0.00155	212798.	-5243.	-4.53E-04	19561.	9.45E+09	-29.2423	67716.	0.00
16.8000	7.00E-05	194299.	-5298.	-3.75E-04	19144.	9.45E+09	-1.3400	68947.	0.00
17.1000	-0.00115	175681.	-5260.	-3.05E-04	18724.	9.45E+09	22.3823	70178.	0.00
17.4000	-0.00213	157261.	-5144.	-2.41E-04	18308.	9.45E+09	42.1579	71410.	0.00
17.7000	-0.00289	139306.	-4963.	-1.85E-04	17903.	9.45E+09	58.2493	72641.	0.00
18.0000	-0.00346	122033.	-4731.	-1.35E-04	17514.	9.45E+09	70.9405	73872.	0.00
18.3000	-0.00386	105615.	-4458.	-9.18E-05	17143.	9.45E+09	80.5298	75103.	0.00
18.6000	-0.00412	90186.	-4156.	-5.45E-05	16795.	9.45E+09	87.3230	76334.	0.00
18.9000	-0.00425	75841.	-3834.	-2.29E-05	16472.	9.45E+09	91.6274	77566.	0.00

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19.2000	-0.00428	62644.	-3500.	3.50E-06	16174.	9.45E+09	93.7467	78797.	0.00
19.5000	-0.00423	50630.	-3162.	2.51E-05	15903.	9.45E+09	93.9767	80028.	0.00
19.8000	-0.00410	39807.	-2826.	4.23E-05	15659.	9.45E+09	92.6011	81259.	0.00
20.1000	-0.00392	30163.	-2498.	5.56E-05	15441.	9.45E+09	89.8889	82490.	0.00
20.4000	-0.00370	21668.	-2181.	6.55E-05	15250.	9.45E+09	86.0916	83722.	0.00
20.7000	-0.00345	14278.	-1880.	7.24E-05	15083.	9.45E+09	81.4419	84953.	0.00
21.0000	-0.00318	7936.	-1596.	7.66E-05	14940.	9.45E+09	76.1518	86184.	0.00
21.3000	-0.00290	2577.	-1332.	7.86E-05	14819.	9.45E+09	70.4125	87415.	0.00
21.6000	-0.00262	-1871.	-1090.	7.87E-05	14803.	9.45E+09	64.3939	88646.	0.00
21.9000	-0.00233	-5484.	-868.8068	7.73E-05	14885.	9.45E+09	58.2446	89878.	0.00
22.2000	-0.00206	-8339.	-670.1997	7.47E-05	14949.	9.45E+09	52.0927	91109.	0.00
22.5000	-0.00180	-10514.	-493.5495	7.11E-05	14998.	9.45E+09	46.0463	92340.	0.00
22.8000	-0.00155	-12087.	-338.3153	6.68E-05	15033.	9.45E+09	40.1949	93571.	0.00
23.1000	-0.00131	-13133.	-203.6669	6.20E-05	15057.	9.45E+09	34.6098	94802.	0.00
23.4000	-0.00110	-13724.	-88.5454	5.69E-05	15070.	9.45E+09	29.3466	96034.	0.00
23.7000	-9.05E-04	-13927.	8.2802	5.16E-05	15075.	9.45E+09	24.4454	97265.	0.00
24.0000	-7.29E-04	-13806.	88.1620	4.63E-05	15072.	9.45E+09	19.9333	98496.	0.00
24.3000	-5.71E-04	-13419.	152.5276	4.11E-05	15064.	9.45E+09	15.8253	99727.	0.00
24.6000	-4.32E-04	-12820.	202.8398	3.61E-05	15050.	9.45E+09	12.1259	100958.	0.00
24.9000	-3.11E-04	-12058.	240.5616	3.14E-05	15033.	9.45E+09	8.8307	102190.	0.00
25.2000	-2.06E-04	-11175.	267.1265	2.70E-05	15013.	9.45E+09	5.9276	103421.	0.00
25.5000	-1.17E-04	-10209.	283.9133	2.29E-05	14991.	9.45E+09	3.3985	104652.	0.00
25.8000	-4.15E-05	-9193.	292.2266	1.92E-05	14968.	9.45E+09	1.2200	105883.	0.00
26.1000	2.13E-05	-8157.	293.2798	1.59E-05	14945.	9.45E+09	-0.6349	107114.	0.00
26.4000	7.30E-05	-7125.	288.1844	1.30E-05	14922.	9.45E+09	-2.1959	108346.	0.00
26.7000	1.15E-04	-6118.	277.9411	1.05E-05	14899.	9.45E+09	-3.4948	109577.	0.00
27.0000	1.48E-04	-5153.	263.4350	8.31E-06	14877.	9.45E+09	-4.5641	110808.	0.00
27.3000	1.75E-04	-4244.	245.4345	6.52E-06	14857.	9.45E+09	-5.4362	112039.	0.00
27.6000	1.95E-04	-3403.	224.5913	5.07E-06	14838.	9.45E+09	-6.1433	113270.	0.00
27.9000	2.11E-04	-2641.	201.4447	3.91E-06	14820.	9.45E+09	-6.7159	114502.	0.00
28.2000	2.23E-04	-1964.	176.4266	3.04E-06	14805.	9.45E+09	-7.1830	115733.	0.00
28.5000	2.33E-04	-1379.	149.8695	2.40E-06	14792.	9.45E+09	-7.5710	116964.	0.00
28.8000	2.41E-04	-891.3479	122.0156	1.97E-06	14781.	9.45E+09	-7.9034	118195.	0.00
29.1000	2.47E-04	-505.7723	93.0287	1.70E-06	14772.	9.45E+09	-8.2005	119426.	0.00
29.4000	2.53E-04	-226.2110	63.0059	1.56E-06	14766.	9.45E+09	-8.4788	120658.	0.00
29.7000	2.58E-04	-56.4167	31.9931	1.51E-06	14762.	9.45E+09	-8.7505	121889.	0.00
30.0000	2.64E-04	0.00	0.00	1.50E-06	14761.	9.45E+09	-9.0234	123160.	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 = 0.52900000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -1105486. inch-lbs
 Maximum shear force = 15387. 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 = 6
 Number of zero deflection points = 2

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	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	0.5290	S, rad	0.00	381000.	0.5290	0.00	15387.	-1105486.

Maximum pile-head deflection = 0.529000000 inches
 Maximum pile-head rotation = 0.000000000 radians = 0.000000 deg.

The analysis ended normally.

Abutment 2

14x89

40' length

0.529" of deflection

Total Stress = Approx. 40 ksi

Fixity = approx. 23.5'

=====

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Files Used for Analysis

Path to file locations:
\\09 Jobs\0026000s\09.0026000.00 - MEDOT - Hall Bridge - Canaan\Work\Calcs\LPILE\Abut 2\14x89\0.529 Deflection\

Name of input data file:
Abut 2_14x89_0.529 in_5 Pile.lp11

Name of output report file:
Abut 2_14x89_0.529 in_5 Pile.lp11

Name of plot output file:
Abut 2_14x89_0.529 in_5 Pile.lp11

Name of runtime message file:
Abut 2_14x89_0.529 in_5 Pile.lp11

Page 1

Abut 2_14x89_0.529 in_5 Pile.lp110

Date and Time of Analysis

Date: July 8, 2020 Time: 8:46:21

Problem Title

Project Name: Hall Bridge

Job Number: 09.0026000.00

Client: MaineDOT

Engineer: N.Williams

Description:

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

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- Abut_2_14x89_.529 in_5 Pile.lp11o
- 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 = 40.000 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.

Point No.	Depth Below Pile Head feet	Pile Diameter inches
1	0.000	14.7000
2	40.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is a H weak axis steel pile
 Length of section = 40.000000 ft
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Pile width = 13.800000 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 2 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 = 20.000000 ft
 Effective unit weight at top of layer = 32.600000 pcf
 Effective unit weight at bottom of layer = 32.600000 pcf
 Friction angle at top of layer = 28.000000 deg.
 Friction angle at bottom of layer = 28.000000 deg.
 Subgrade k at top of layer = 15.000000 pci
 Subgrade k at bottom of layer = 15.000000 pci

Layer 2 is sand, p-y criteria by Reese et al., 1974

Distance from top of pile to top of layer = 20.000000 ft
 Distance from top of pile to bottom of layer = 40.000000 ft
 Effective unit weight at top of layer = 67.600000 pcf
 Effective unit weight at bottom of layer = 67.600000 pcf
 Friction angle at top of layer = 35.000000 deg.
 Friction angle at bottom of layer = 35.000000 deg.
 Subgrade k at top of layer = 95.000000 pci
 Subgrade k at bottom of layer = 95.000000 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	Angle of Friction deg.	kpy pci
1	Sand	0.00	32.6000	28.0000	15.0000
	(Reese, et al.)	20.0000	32.6000	28.0000	15.0000
2	Sand	20.0000	67.6000	35.0000	95.0000
	(Reese, et al.)	40.0000	67.6000	35.0000	95.0000

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 = 0.529000 in	S = 0.0000 in/in	381000.	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	=	40.000000 ft
Flange Width	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Yield Stress of Pipe	=	45.000000 ksi
Elastic Modulus	=	29000. ksi
Cross-sectional Area	=	25.811550 sq. in.
Moment of Inertia	=	325.837265 in^4
Elastic Bending Stiffness	=	9449281. kip-in^2
Plastic Modulus, Z	=	67.636247in^3
Plastic Moment Capacity = Fy Z	=	3044.in-kip

Axial Structural Capacities:

Nom. Axial Structural Capacity = Fy As	=	1161.520 kips
Nominal Axial Tensile Capacity	=	-1161.520 kips

Number of Axial Thrust Force Values Determined from Pile-head Loadings = 1

Number	Axial Thrust Force kips
1	381.000

Definition of Run Messages:

Y = part of pipe section has yielded.

Axial Thrust Force = 381.000 kips

Bending Curvature rad/in.	Bending Moment in-kip	Bending Stiffness kip-in2	Depth to N Axis in	Max Total Stress ksi	Run Msg
0.00000386	36.4402599	9448389.	139.3242518	15.5746835	
0.00000771	72.8805198	9448389.	73.3371259	16.3885333	
0.00001157	109.3207797	9448389.	51.3414173	17.2023828	
0.00001543	145.7610396	9448389.	40.3435630	18.0162328	
0.00001928	182.2012995	9448389.	33.7448504	18.8300824	
0.00002314	218.6415594	9448389.	29.3457086	19.6439322	
0.00002700	255.0818192	9448389.	26.2034645	20.4577819	
0.00003085	291.5220791	9448389.	23.8467815	21.2716317	
0.00003471	327.9623390	9448389.	22.0138058	22.0854813	
0.00003857	364.4025989	9448389.	20.5474252	22.8993312	
0.00004242	400.8428588	9448389.	19.3476593	23.7131807	
0.00004628	437.2831187	9448389.	18.3478543	24.5270305	
0.00005014	473.7233786	9448389.	17.5018655	25.3408802	
0.00005399	510.1636385	9448389.	16.7767323	26.1547300	
0.00005785	546.6038984	9448389.	16.1482835	26.9685796	
0.00006171	583.0441583	9448389.	15.5983907	27.7824293	
0.00006557	619.4844182	9448389.	15.1131913	28.5962791	
0.00006942	655.9246781	9448389.	14.6819029	29.4101288	
0.00007328	692.3649380	9448389.	14.2960133	30.2239786	
0.00007714	728.8051978	9448389.	13.9487126	31.0378283	
0.00008099	765.2454577	9448389.	13.6344882	31.8516780	
0.00008485	801.6857176	9448389.	13.3488296	32.6655277	
0.00008871	838.1259775	9448389.	13.0880109	33.4793775	
0.00009256	874.5662374	9448389.	12.8489272	34.2932272	
0.00009642	911.0064973	9448389.	12.6289701	35.1070769	
0.0001003	947.4467572	9448389.	12.4259328	35.9209267	
0.0001041	983.8870171	9448389.	12.2379353	36.7347764	
0.0001080	1020.	9448389.	12.0633661	37.5486261	
0.0001118	1057.	9448389.	11.9008363	38.3624759	
0.0001157	1093.	9448389.	11.7491417	39.1763255	
0.0001196	1130.	9448389.	11.6072339	39.9901753	
0.0001234	1166.	9448389.	11.4741954	40.8040250	
0.0001273	1203.	9448389.	11.3492198	41.6178747	
0.0001311	1239.	9448389.	11.2315956	42.4317245	
0.0001350	1275.	9448389.	11.1206929	43.2455742	

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0.0001388	1312.	9448389.	11.0159514	44.0594239	
0.0001427	1348.	9448389.	10.9168717	44.8732736	
0.0001466	1384.	9441578.	10.8242594	45.0000000	Y
0.0001504	1418.	9425583.	10.7381862	45.0000000	Y
0.0001581	1482.	9373599.	10.5829769	45.0000000	Y
0.0001658	1542.	9300644.	10.4474242	45.0000000	Y
0.0001736	1599.	9212372.	10.3285448	45.0000000	Y
0.0001813	1652.	9113345.	10.2238420	45.0000000	Y
0.0001890	1702.	9007258.	10.1312052	45.0000000	Y
0.0001967	1750.	8897099.	10.0488321	45.0000000	Y
0.0002044	1796.	8784041.	9.9754974	45.0000000	Y
0.0002121	1839.	8670126.	9.9098731	45.0000000	Y
0.0002198	1881.	8555747.	9.8511508	45.0000000	Y
0.0002275	1921.	8442555.	9.7982592	45.0000000	Y
0.0002353	1960.	8330891.	9.7505498	45.0000000	Y
0.0002430	1997.	8220673.	9.7075685	45.0000000	Y
0.0002507	2034.	8112600.	9.6686762	45.0000000	Y
0.0002584	2069.	8006993.	9.6333869	45.0000000	Y
0.0002661	2103.	7903982.	9.6013098	45.0000000	Y
0.0002738	2137.	7803672.	9.5720954	45.0000000	Y
0.0002815	2170.	7705938.	9.5455040	45.0000000	Y
0.0002893	2201.	7610685.	9.5213184	45.0000000	Y
0.0002970	2233.	7518212.	9.4991957	45.0000000	Y
0.0003047	2263.	7426854.	9.4784117	45.0000000	Y
0.0003124	2291.	7333932.	9.4579312	45.0000000	Y
0.0003201	2317.	7239380.	9.4380631	45.0000000	Y
0.0003278	2342.	7144504.	9.4185122	45.0000000	Y
0.0003355	2366.	7049978.	9.3991848	45.0000000	Y
0.0003433	2387.	6954953.	9.3804804	45.0000000	Y
0.0003510	2408.	6860717.	9.3617415	45.0000000	Y
0.0003587	2427.	6767066.	9.3435941	45.0000000	Y
0.0003664	2446.	6675020.	9.3256747	45.0000000	Y
0.0003741	2463.	6583982.	9.3083194	45.0000000	Y
0.0003818	2480.	6493990.	9.2907889	45.0000000	Y
0.0003895	2495.	6405305.	9.2739970	45.0000000	Y
0.0003972	2510.	6318801.	9.2573475	45.0000000	Y
0.0004050	2524.	6232812.	9.2409172	45.0000000	Y
0.0004127	2538.	6149058.	9.2249176	45.0000000	Y
0.0004204	2550.	6066812.	9.2089116	45.0000000	Y
0.0004281	2563.	5985796.	9.1935006	45.0000000	Y
0.0004358	2574.	5906787.	9.1782401	45.0000000	Y
0.0004435	2585.	5829217.	9.1629758	45.0000000	Y
0.0004512	2596.	5753220.	9.1484327	45.0000000	Y
0.0004590	2606.	5678577.	9.1337797	45.0000000	Y
0.0004668	2643.	5396200.	9.0778694	45.0000000	Y
0.0005207	2675.	5136759.	9.0253558	45.0000000	Y

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0.0005515	2702.	4898387.	8.9758553	45.0000000	Y
0.0005824	2725.	4679382.	8.9290907	45.0000000	Y
0.0006132	2746.	4477690.	8.8852574	45.0000000	Y
0.0006441	2764.	4291629.	8.8437143	45.0000000	Y
0.0006749	2780.	4119497.	8.8042909	45.0000000	Y
0.0007058	2795.	3959920.	8.7668005	45.0000000	Y
0.0007366	2808.	3811604.	8.7312830	45.0000000	Y
0.0007675	2820.	3673715.	8.6972526	45.0000000	Y
0.0007984	2830.	3545098.	8.6651982	45.0000000	Y
0.0008292	2840.	3425079.	8.6348131	45.0000000	Y
0.0008601	2849.	3312378.	8.6052605	45.0000000	Y

Summary of Results for Nominal Moment Capacity for Section 1

Load No.	Axial Thrust kips	Nominal Moment Capacity in-kips
1	381.0000000000	2849.

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	Equivalent Top Depth Below Grnd Surf	Same Layer Type As Layer	Layer is Rock or is Below	F0 Integral for Layer	F1 Integral for Layer
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	ft	ft	Above	Rock Layer	
1	0.00	0.00	N.A.	No	0.00
2	20.0000	15.9400	Yes	No	131831.

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 = 0.529000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 381000.0 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	0.5290	-1105557.	15372.	0.00	39699.	9.45E+09	0.00	0.00	0.00
0.4000	0.5277	-1031408.	15317.	-5.43E-04	38027.	9.45E+09	-9.9505	90.5188	0.00
0.8000	0.5238	-956530.	15242.	-0.00105	36338.	9.45E+09	-21.2440	194.6796	0.00
1.2000	0.5176	-881253.	15112.	-0.00151	34640.	9.45E+09	-32.7392	303.6132	0.00
1.6000	0.5092	-805912.	14929.	-0.00194	32940.	9.45E+09	-43.6906	411.8120	0.00
2.0000	0.4989	-730828.	14695.	-0.00233	31246.	9.45E+09	-53.6197	515.8435	0.00
2.4000	0.4868	-656301.	14417.	-0.00269	29565.	9.45E+09	-62.4792	616.0044	0.00
2.8000	0.4732	-582603.	14099.	-0.00300	27903.	9.45E+09	-69.8622	708.7285	0.00
3.2000	0.4580	-509974.	13750.	-0.00328	26264.	9.45E+09	-75.8005	794.3428	0.00
3.6000	0.4417	-438618.	13372.	-0.00352	24655.	9.45E+09	-81.3164	883.7025	0.00
4.0000	0.4243	-368727.	12965.	-0.00372	23078.	9.45E+09	-88.4044	1000.	0.00
4.4000	0.4059	-300531.	12526.	-0.00389	21540.	9.45E+09	-94.7537	1120.	0.00
4.8000	0.3869	-234239.	12060.	-0.00403	20045.	9.45E+09	-99.1415	1230.	0.00
5.2000	0.3672	-170013.	11566.	-0.00413	18596.	9.45E+09	-106.8612	1397.	0.00
5.6000	0.3472	-108092.	11032.	-0.00420	17199.	9.45E+09	-115.6224	1598.	0.00
6.0000	0.3269	-48734.	10458.	-0.00424	15860.	9.45E+09	-123.6167	1815.	0.00
6.4000	0.3065	7822.	9844.	-0.00425	14937.	9.45E+09	-132.1306	2069.	0.00
6.8000	0.2861	61325.	9189.	-0.00424	16144.	9.45E+09	-140.7318	2361.	0.00

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7.2000	0.2658	111529.	8494.	-0.00419	17277.	9.45E+09	-148.7922	2687.	0.00
7.6000	0.2458	158202.	7762.	-0.00412	18329.	9.45E+09	-156.2695	3051.	0.00
8.0000	0.2262	201127.	6996.	-0.00403	19298.	9.45E+09	-163.0137	3459.	0.00
8.4000	0.2071	240109.	6199.	-0.00392	20177.	9.45E+09	-168.8131	3912.	0.00
8.8000	0.1886	274979.	5378.	-0.00379	20964.	9.45E+09	-173.5815	4418.	0.00
9.2000	0.1707	305594.	4536.	-0.00364	21654.	9.45E+09	-177.2425	4983.	0.00
9.6000	0.1536	331841.	3679.	-0.00348	22246.	9.45E+09	-179.7303	5616.	0.00
10.0000	0.1373	353639.	2813.	-0.00331	22738.	9.45E+09	-180.9900	6326.	0.00
10.4000	0.1219	370939.	1944.	-0.00312	23128.	9.45E+09	-180.9778	7127.	0.00
10.8000	0.1074	383724.	1079.	-0.00293	23417.	9.45E+09	-179.6613	8033.	0.00
11.2000	0.09376	392013.	222.8666	-0.00273	23604.	9.45E+09	-177.0182	9062.	0.00
11.6000	0.08112	395860.	-608.4791	-0.00253	23690.	9.45E+09	-169.3759	10022.	0.00
12.0000	0.06944	395436.	-1375.	-0.00233	23681.	9.45E+09	-149.9980	10368.	0.00
12.4000	0.05873	391189.	-2050.	-0.00213	23585.	9.45E+09	-131.0911	10714.	0.00
12.8000	0.04898	383558.	-2635.	-0.00194	23413.	9.45E+09	-112.8397	11059.	0.00
13.2000	0.04015	372971.	-3135.	-0.00174	23174.	9.45E+09	-95.4056	11405.	0.00
13.6000	0.03224	359840.	-3553.	-0.00156	22878.	9.45E+09	-78.9276	11750.	0.00
14.0000	0.02521	344555.	-3895.	-0.00138	22533.	9.45E+09	-63.5215	12096.	0.00
14.4000	0.01901	327487.	-4166.	-0.00121	22148.	9.45E+09	-49.2802	12442.	0.00
14.8000	0.01362	308979.	-4371.	-0.00105	21731.	9.45E+09	-36.2743	12787.	0.00
15.2000	0.00897	289349.	-4517.	-8.94E-04	21288.	9.45E+09	-24.5528	13133.	0.00
15.6000	0.00504	268884.	-4610.	-7.52E-04	20826.	9.45E+09	-14.1440	13478.	0.00
16.0000	0.00176	247843.	-4656.	-6.21E-04	20351.	9.45E+09	-5.0567	13824.	0.00
16.4000	-9.21E-04	226455.	-4662.	-5.00E-04	19869.	9.45E+09	2.7190	14170.	0.00
16.8000	-0.00305	204920.	-4633.	-3.91E-04	19383.	9.45E+09	9.2103	14515.	0.00
17.2000	-0.00467	183406.	-4576.	-2.92E-04	18898.	9.45E+09	14.4605	14861.	0.00
17.6000	-0.00585	162056.	-4497.	-2.04E-04	18416.	9.45E+09	18.5278	15206.	0.00
18.0000	-0.00663	140981.	-4401.	-1.27E-04	17941.	9.45E+09	21.4844	15552.	0.00
18.4000	-0.00707	120271.	-4293.	-6.09E-05	17474.	9.45E+09	23.4150	15898.	0.00
18.8000	-0.00722	99988.	-4178.	-4.91E-06	17016.	9.45E+09	24.4163	16243.	0.00
19.2000	-0.00712	80175.	-4061.	4.09E-05	16569.	9.45E+09	24.5959	16589.	0.00
19.6000	-0.00682	60855.	-3944.	7.67E-05	16134.	9.45E+09	24.0717	16934.	0.00
20.0000	-0.00638	42032.	-3670.	1.03E-04	15709.	9.45E+09	90.2869	67919.	0.00
20.4000	-0.00584	25250.	-3157.	1.20E-04	15330.	9.45E+09	123.1673	101302.	0.00
20.8000	-0.00523	11283.	-2566.	1.29E-04	15015.	9.45E+09	123.0946	112979.	0.00
21.2000	-0.00460	141.3805	-2004.	1.32E-04	14764.	9.45E+09	111.0743	116006.	0.00
21.6000	-0.00396	-8441.	-1504.	1.30E-04	14951.	9.45E+09	97.5542	118195.	0.00
22.0000	-0.00335	-14768.	-1068.	1.24E-04	15094.	9.45E+09	83.9721	120384.	0.00
22.4000	-0.00277	-19147.	-696.5789	1.15E-04	15193.	9.45E+09	70.7500	122573.	0.00
22.8000	-0.00224	-21878.	-387.0636	1.05E-04	15254.	9.45E+09	58.2147	124762.	0.00
23.2000	-0.00176	-23247.	-135.4933	9.36E-05	15285.	9.45E+09	46.6062	126950.	0.00
23.6000	-0.00134	-23521.	62.9715	8.17E-05	15291.	9.45E+09	36.0874	129139.	0.00
24.0000	-9.78E-04	-22941.	213.7909	6.99E-05	15278.	9.45E+09	26.7540	131328.	0.00
24.4000	-6.70E-04	-21724.	322.7491	5.86E-05	15251.	9.45E+09	18.6452	133517.	0.00
24.8000	-4.16E-04	-20057.	395.7064	4.79E-05	15213.	9.45E+09	11.7537	135706.	0.00
25.2000	-2.10E-04	-18101.	438.3991	3.82E-05	15169.	9.45E+09	6.0350	137894.	0.00

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25.6000	-4.85E-05	-15988.	456.2836	2.96E-05	15121.	9.45E+09	1.4169	140083.	0.00
26.0000	7.40E-05	-13829.	454.4211	2.20E-05	15073.	9.45E+09	-2.1929	142272.	0.00
26.4000	1.63E-04	-11706.	437.3989	1.55E-05	15025.	9.45E+09	-4.8996	144461.	0.00
26.8000	2.23E-04	-9686.	409.2835	1.01E-05	14979.	9.45E+09	-6.8152	146650.	0.00
27.2000	2.60E-04	-7814.	373.5993	5.65E-06	14937.	9.45E+09	-8.0532	148838.	0.00
27.6000	2.77E-04	-6120.	333.3310	2.11E-06	14899.	9.45E+09	-8.7252	151027.	0.00
28.0000	2.80E-04	-4622.	290.9422	-6.18E-07	14865.	9.45E+09	-8.9368	153216.	0.00
28.4000	2.71E-04	-3325.	248.4076	-2.64E-06	14836.	9.45E+09	-8.7859	155405.	0.00
28.8000	2.55E-04	-2228.	207.2549	-4.05E-06	14811.	9.45E+09	-8.3610	157594.	0.00
29.2000	2.33E-04	-1321.	168.6124	-4.95E-06	14791.	9.45E+09	-7.7400	159782.	0.00
29.6000	2.07E-04	-590.8579	133.2600	-5.43E-06	14774.	9.45E+09	-6.9902	161971.	0.00
30.0000	1.80E-04	-21.5240	101.6806	-5.59E-06	14761.	9.45E+09	-6.1679	164160.	0.00
30.4000	1.53E-04	405.7217	74.1112	-5.49E-06	14770.	9.45E+09	-5.3194	166349.	0.00
30.8000	1.28E-04	710.0323	50.5902	-5.21E-06	14777.	9.45E+09	-4.4811	168538.	0.00
31.2000	1.03E-04	910.4394	31.0018	-4.80E-06	14781.	9.45E+09	-3.6808	170726.	0.00
31.6000	8.16E-05	1025.	15.1157	-4.31E-06	14784.	9.45E+09	-2.9384	172915.	0.00
32.0000	6.22E-05	1071.	2.6219	-3.77E-06	14785.	9.45E+09	-2.2673	175104.	0.00
32.4000	4.53E-05	1064.	-6.8396	-3.23E-06	14785.	9.45E+09	-1.6750	177293.	0.00
32.8000	3.11E-05	1017.	-13.6540	-2.70E-06	14784.	9.45E+09	-1.1643	179482.	0.00
33.2000	1.94E-05	942.9701	-18.2115	-2.20E-06	14782.	9.45E+09	-0.7346	181670.	0.00
33.6000	9.98E-06	850.6839	-20.8923	-1.75E-06	14780.	9.45E+09	-0.3823	183859.	0.00
34.0000	2.63E-06	748.7982	-22.0543	-1.34E-06	14778.	9.45E+09	-0.1018	186048.	0.00
34.4000	-2.90E-06	643.8705	-22.0257	-9.88E-07	14775.	9.45E+09	0.1138	188237.	0.00
34.8000	-6.86E-06	540.9658	-21.0996	-6.87E-07	14773.	9.45E+09	0.2721	190426.	0.00
35.2000	-9.50E-06	443.8282	-19.5317	-4.37E-07	14771.	9.45E+09	0.3812	192614.	0.00
35.6000	-1.11E-05	355.0601	-17.5401	-2.34E-07	14769.	9.45E+09	0.4487	194803.	0.00
36.0000	-1.17E-05	276.2995	-15.3063	-7.38E-08	14767.	9.45E+09	0.4821	196992.	0.00
36.4000	-1.18E-05	208.3893	-12.9778	4.93E-08	14766.	9.45E+09	0.4882	199181.	0.00
36.8000	-1.13E-05	151.5324	-10.6712	1.41E-07	14764.	9.45E+09	0.4729	201370.	0.00
37.2000	-1.04E-05	105.4308	-8.4764	2.06E-07	14763.	9.45E+09	0.4416	203558.	0.00
37.6000	-9.30E-06	69.4053	-6.4604	2.50E-07	14762.	9.45E+09	0.3984	205747.	0.00
38.0000	-8.01E-06	42.4950	-4.6716	2.79E-07	14762.	9.45E+09	0.3469	207936.	0.00
38.4000	-6.62E-06	23.5384	-3.1436	2.96E-07	14761.	9.45E+09	0.2897	210125.	0.00
38.8000	-5.17E-06	11.2349	-1.8994	3.04E-07	14761.	9.45E+09	0.2287	212314.	0.00
39.2000	-3.70E-06	4.1902	-0.9542	3.08E-07	14761.	9.45E+09	0.1651	214502.	0.00
39.6000	-2.21E-06	0.9462	-0.3185	3.10E-07	14761.	9.45E+09	0.09977	216691.	0.00
40.0000	-7.22E-07	0.00	0.00	3.10E-07	14761.	9.45E+09	0.03294	109440.	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 = 0.52900000 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -1105557. inch-lbs
 Maximum shear force = 15372. 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 = 6
 Number of zero deflection points = 4

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	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	0.5290	S, rad	0.00	381000.	0.5290	0.00	15372.	-1105557.

Maximum pile-head deflection = 0.529000000 inches
 Maximum pile-head rotation = 0.000000000 radians = 0.000000 deg.

The analysis ended normally.

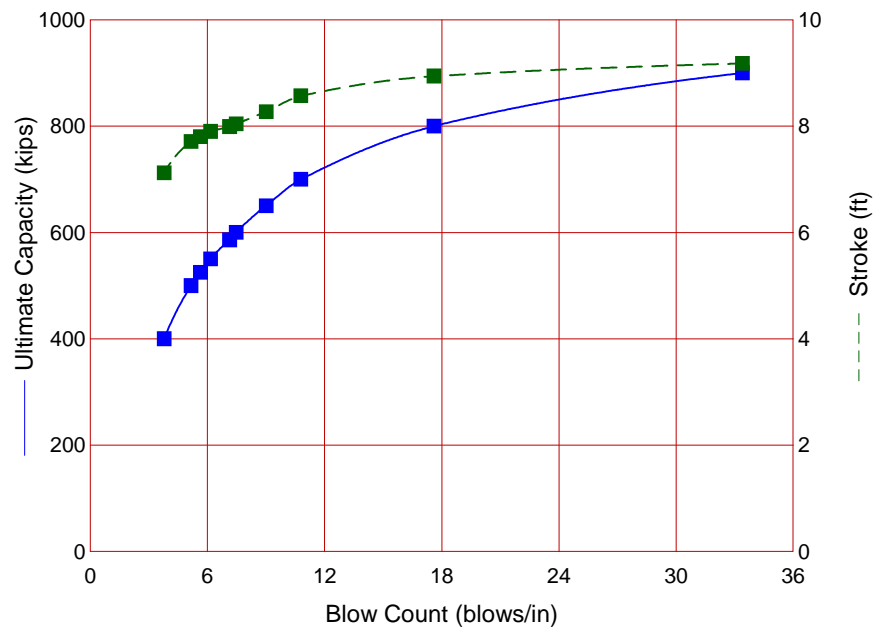
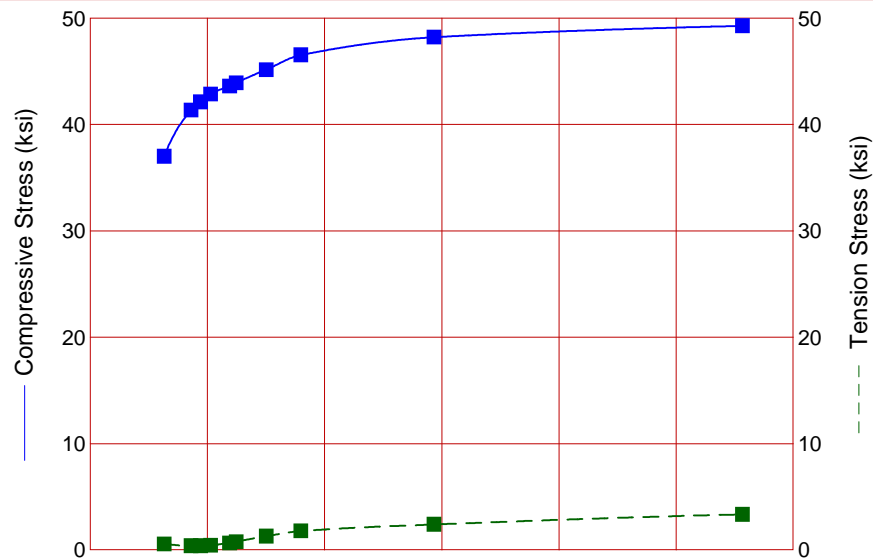
WEAP Analysis

Abutment 1

									Weap Analysis					
Pile Size	Length = Total + 5' stickup	Side Friction (k) From Apile	Nominal Capacity ([A-Pile /.65, kips)	% Side Friction	Skin Quake	Toe Quake	Skin Damping	Toe Damping	Hammer	Ram Weight (kips)	Max Compression Stress (ksi)	BPI	Stroke	Notes
Pier Piles														
14x89	35	53.6	586	10	0.1	0.1	0.2	0.15	Delmag D30	6.6	41.8	7.3	8	Fuel Setting 1 (100%)
14x89	35	53.6	586	10	0.1	0.04	0.2	0.15	Delmag D30	6.6	46.6	5.9	8	Fuel Setting 1 (100%)
14x89	35	53.6	586	10	0.1	0.04	0.2	0.15	Delmag D30	6.6	44.3	7.2	7.2	Fuel Setting 2 (90%)

Abutment 2

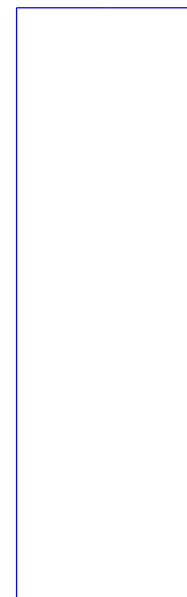
Pier Piles														
14x89	45	73.8	586	13	0.1	0.1	0.2	0.15	Delmag D30	6.6	41.9	7.8	8.1	Fuel Setting 1 (100%)
14x89	45	73.8	586	13	0.1	0.04	0.2	0.15	Delmag D30	6.6	44.3	6.5	8	Fuel Setting 1 (100%)



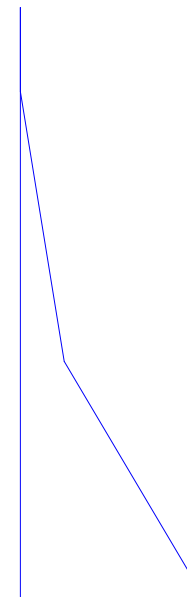
DELMAG D 30

Ram Weight	6.60 kips
Efficiency	0.800
Pressure	1415 (100%) psi
Helmet Weight	1.90 kips
Hammer Cushion	60155 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	35.00 ft
Pile Penetration	30.00 ft
Pile Top Area	26.10 in ²

Pile Model



Skin Friction Distribution

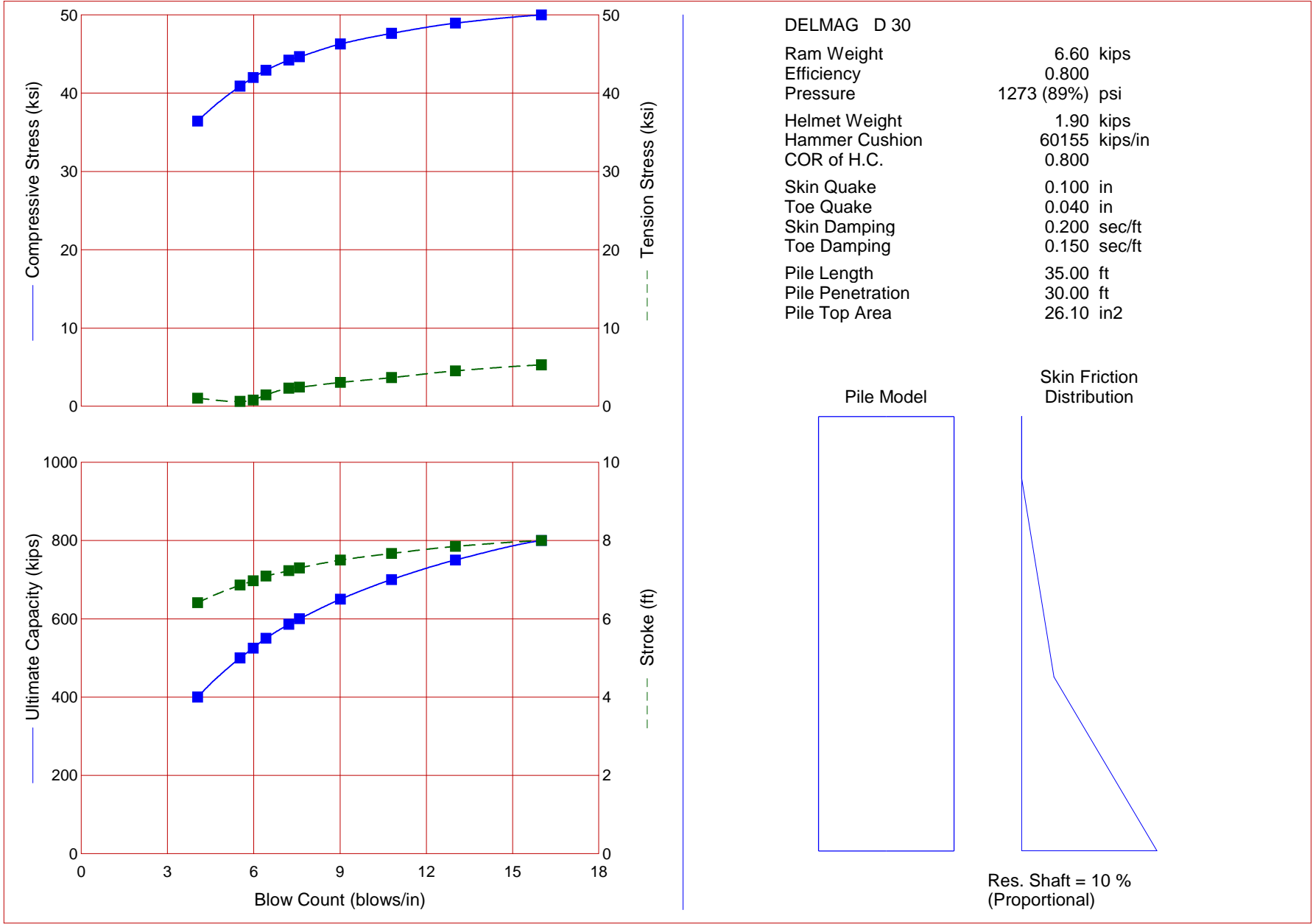


Res. Shaft = 10 %
(Proportional)

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Hall Bridge - Abutment 1 - 14x89

14-Jul-2020
GRLWEAP Version 2010

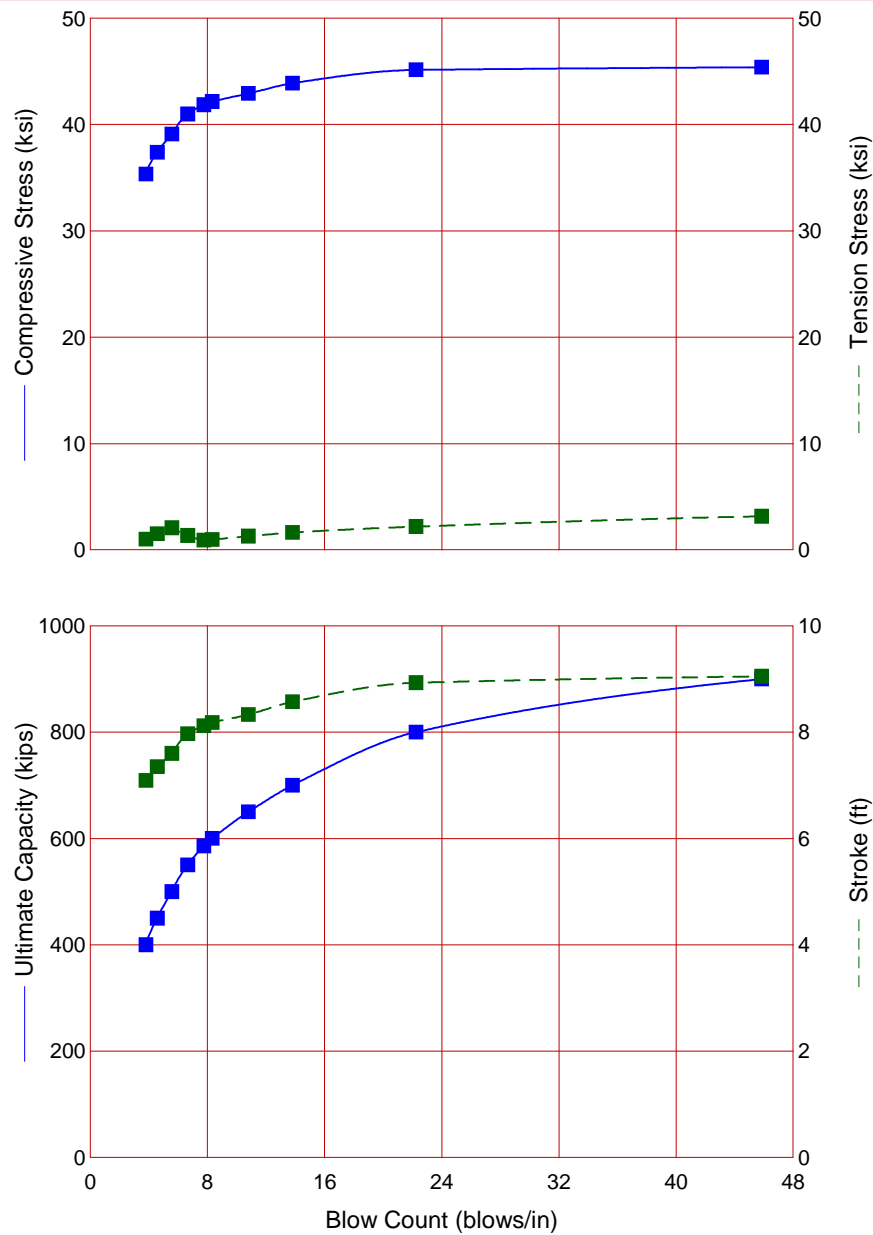
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	37.01	0.55	3.8	7.12	24.19
500.0	41.35	0.39	5.2	7.71	25.59
525.0	42.13	0.39	5.6	7.80	25.77
550.0	42.85	0.43	6.2	7.90	25.95
586.0	43.61	0.65	7.1	7.99	25.95
600.0	43.91	0.77	7.5	8.04	26.13
650.0	45.14	1.30	9.0	8.27	26.96
700.0	46.54	1.78	10.8	8.57	28.11
800.0	48.20	2.40	17.6	8.94	29.44
900.0	49.27	3.34	33.4	9.18	30.38



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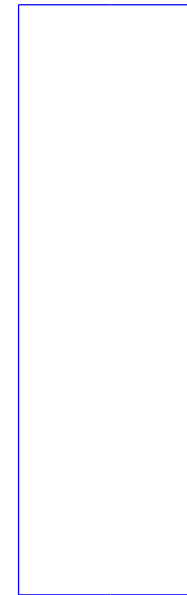
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	36.41	1.03	4.0	6.41	20.40
500.0	40.89	0.64	5.5	6.86	21.26
525.0	41.97	0.80	6.0	6.97	21.48
550.0	42.92	1.46	6.4	7.09	21.83
586.0	44.22	2.32	7.2	7.23	22.32
600.0	44.64	2.45	7.6	7.30	22.51
650.0	46.27	3.06	9.0	7.50	23.28
700.0	47.62	3.67	10.8	7.67	23.98
750.0	48.94	4.53	13.0	7.85	24.64
800.0	49.98	5.30	16.0	8.00	25.19



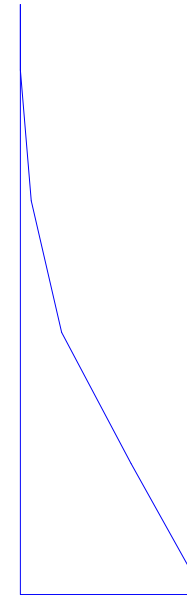
DELMAG D 30

Ram Weight	6.60 kips
Efficiency	0.800
Pressure	1415 (100%) psi
Helmet Weight	1.90 kips
Hammer Cushion	60155 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	45.00 ft
Pile Penetration	40.00 ft
Pile Top Area	26.10 in ²

Pile Model



Skin Friction Distribution

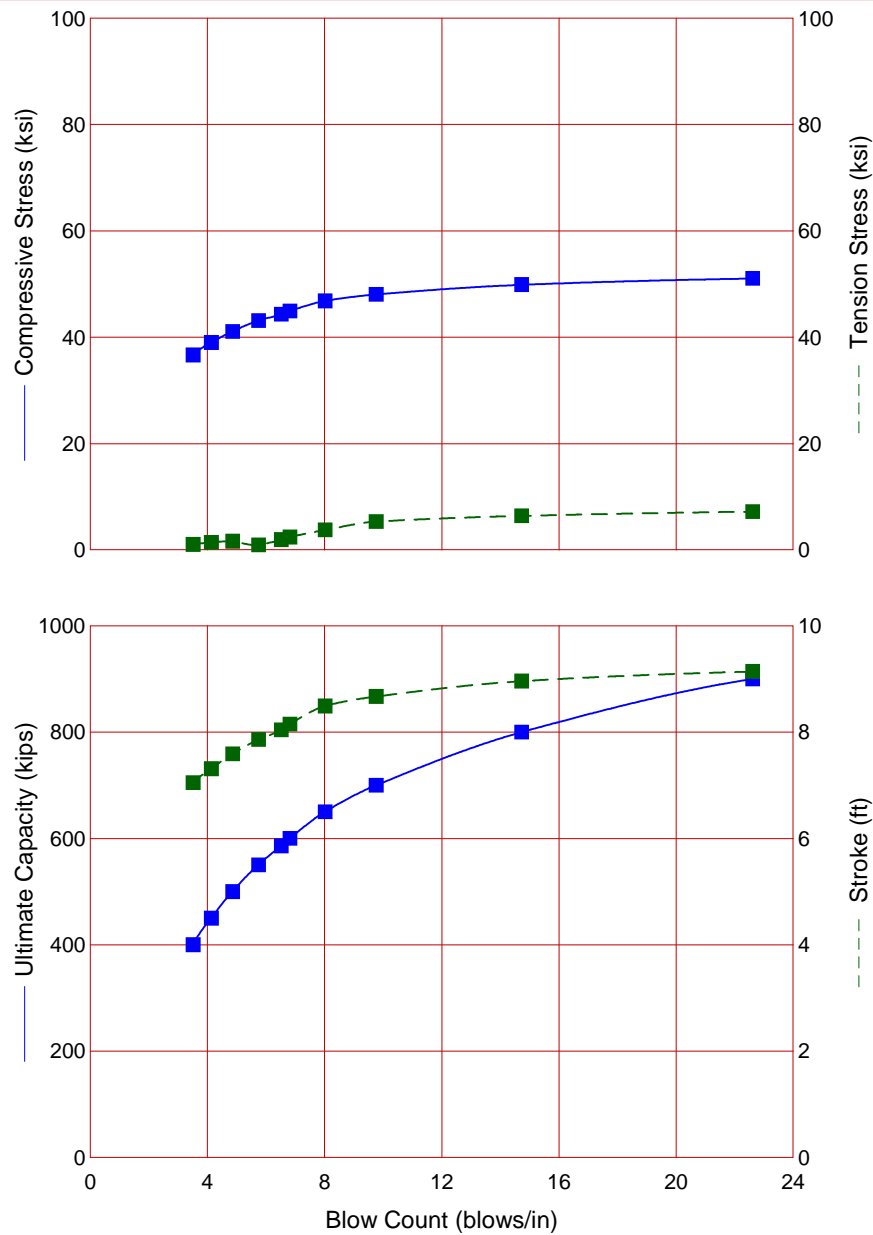


Res. Shaft = 13 %
(Proportional)

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GRLWEAP Version 2010

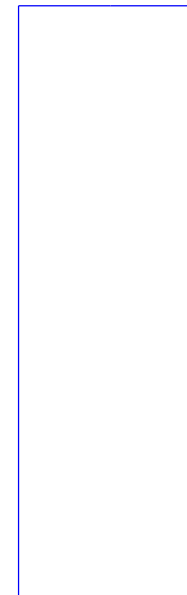
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	35.34	1.00	3.8	7.09	25.04
450.0	37.39	1.52	4.6	7.35	25.60
500.0	39.10	2.07	5.6	7.60	26.24
550.0	40.98	1.35	6.7	7.97	27.64
586.0	41.85	0.92	7.8	8.12	28.28
600.0	42.15	0.98	8.3	8.18	28.44
650.0	42.92	1.29	10.8	8.33	29.01
700.0	43.88	1.64	13.8	8.57	29.99
800.0	45.13	2.19	22.3	8.93	31.37
900.0	45.38	3.17	45.9	9.05	31.75



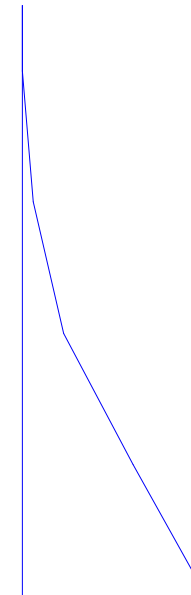
DELMAG D 30

Ram Weight	6.60 kips
Efficiency	0.800
Pressure	1415 (100%) psi
Helmet Weight	1.90 kips
Hammer Cushion	60155 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	45.00 ft
Pile Penetration	40.00 ft
Pile Top Area	26.10 in ²

Pile Model



Skin Friction Distribution



Res. Shaft = 13 %
(Proportional)

GZA Geo Environmental, Inc.
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14-Jul-2020
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	36.63	1.03	3.5	7.05	24.45
450.0	39.00	1.41	4.1	7.31	25.04
500.0	41.06	1.62	4.9	7.59	25.80
550.0	43.11	0.94	5.8	7.86	26.84
586.0	44.30	1.92	6.5	8.04	27.55
600.0	44.91	2.41	6.8	8.15	27.97
650.0	46.81	3.74	8.0	8.49	29.33
700.0	48.04	5.34	9.8	8.67	30.03
800.0	49.86	6.41	14.7	8.96	31.20
900.0	51.05	7.21	22.6	9.14	31.94

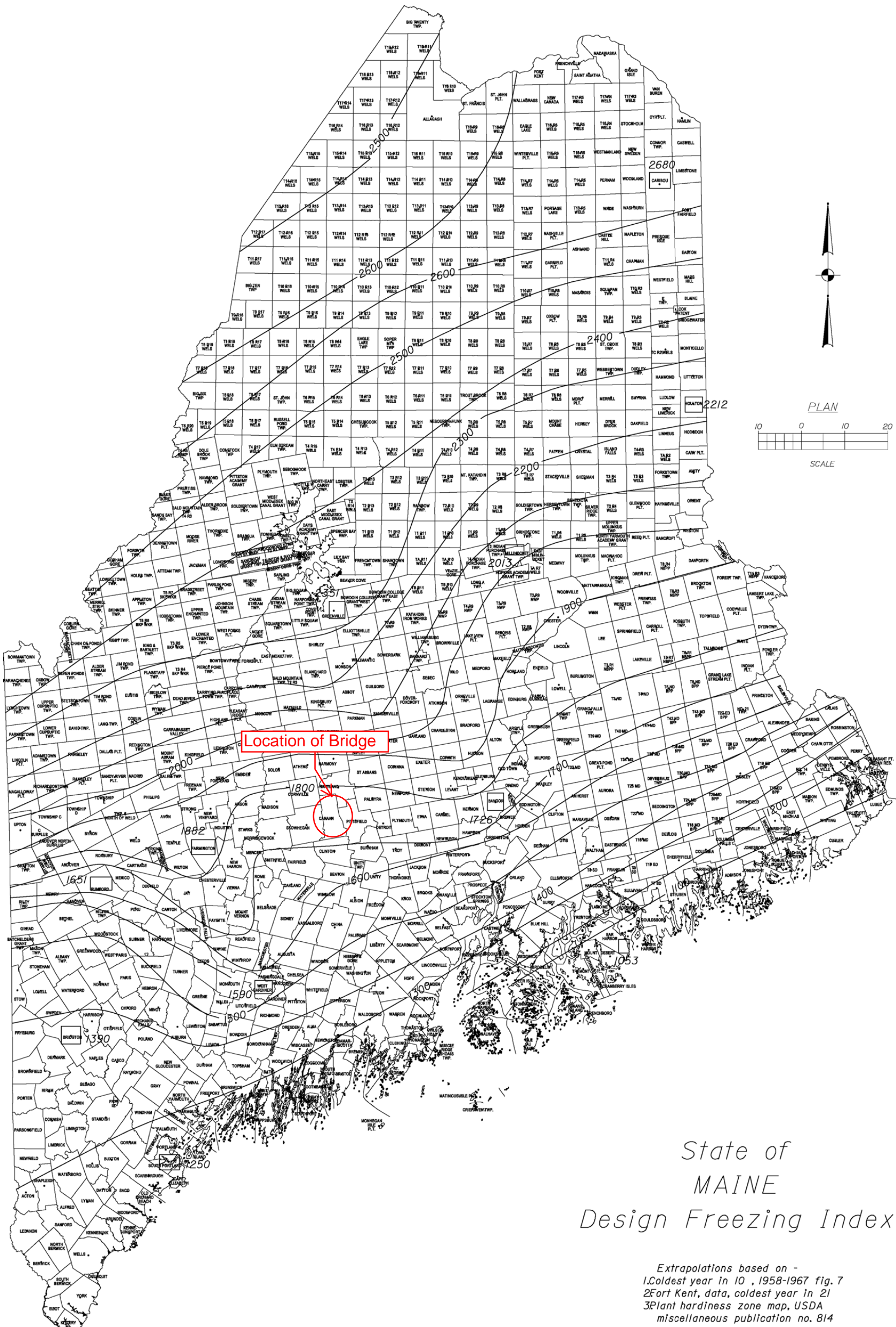
Frost Depth

GZA

calc: N Williams 11-06-20

check: C Snow 01-22-21

Figure 5-1 Maine Design Freezing Index Map



5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

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	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	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

Note: Testing and descriptions typically showed Fine to Coarse Sand with some Silt, so the average of fine-grained and coarse grained for 1750 Freezing Index was used for Frost depth = 6.3'. We recommend 6.5' for frost.

MC results show 10% or less for the most part.

- 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.