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GEOTECHNICAL DESIGN REPORT LITTLE RIVER BRIDGE NO. 3112 ROUTE 237 OVER LITTLE RIVER GORHAM, MAINE

Prepared for:

Hoyle, Tanner & Associates, Inc.
Manchester, New Hampshire

October 2019
09.0025988.01

Prepared by:

GZA GeoEnvironmental, Inc.

477 Congress Street | Suite 700 | Portland, Maine 04101
207.879.9190

31 Offices Nationwide

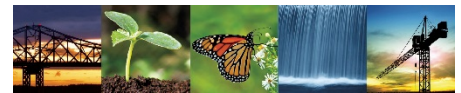
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Suite 700
Portland, ME 04101
T: 207.879.9190
F: 207.879.0099
www.gza.com



VIA EMAIL

October 1, 2019
File No. 09.0025988.01

Mr. Steven Hodgdon, P.E.
Hoyle, Tanner & Associates, Inc.
150 Dow Street
Manchester, New Hampshire 03101

Re: Geotechnical Design Report
Little River Bridge No. 3112, Route 237 over Little River
Maine Department of Transportation WIN 23909.00
Gorham, Maine

Dear Steve:

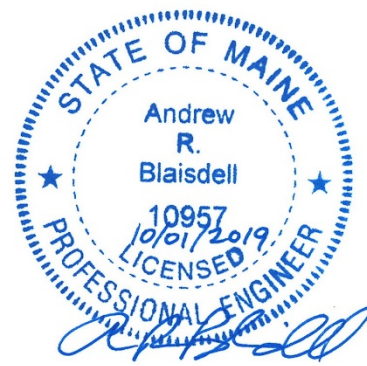
We are pleased to provide this Geotechnical Design Report (GDR) to Hoyle, Tanner & Associates, Inc. (Hoyle, Tanner) for the subject project. Our work was completed in accordance with the Agreement between Hoyle, Tanner and GZA GeoEnvironmental, Inc. (GZA) dated May 13, 2019, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to Hoyle, Tanner, who is under contract with the Maine Department of Transportation (MaineDOT) for final design of the proposed bridge.

It has been a pleasure serving Hoyle, Tanner on this 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.


Christopher L. Snow, P.E.
Consultant Reviewer


Andrew R. Blaisdell, P.E.
Associate Principal

ARB/CLS:erc

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Attachment: Geotechnical Design Report



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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the subject project. Our services were completed in accordance with the Subconsultant Agreement with Hoyle, Tanner and Associates, Inc. (Hoyle, Tanner) dated May 13, 2019. This report is subject to the *Limitations* included in **Appendix A**.

1.1 BACKGROUND

The project includes replacement of the Little River Bridge No. 3112 Carrying Route 237 over Little River in Gorham, Maine, the location of which is shown on **Figure 1**. Built in 1952, the existing bridge is comprised of a riveted steel through-plate girder superstructure with steel beams and stringers. The bridge spans 100 feet and is supported on concrete stub abutments with timber piles. Each abutment is supported by three rows of piles, where the front and back rows are battered in opposing directions at an inclination of 2 inches per foot. The plans indicate an expected tip elevation around El. 31¹. The concrete bridge deck was replaced in 1989.

The existing south and north approach embankments are at approximately El. 91 and El. 87, respectively. The approach embankments are about 10 to 20 feet above adjacent grades. The riverbed elevation beneath the bridge is at about El. 64.

Based on our review of the information provided by Hoyle, Tanner, plans are to construct a replacement bridge on the existing alignment consisting of a 135-foot-long, single span, integral abutment bridge with steel plate girder beams and approximately 13-foot-high backwalls. The bridge will be supported on driven piles. We understand the bridge length has been selected so that the existing battered piles can be cut off near existing grades, and the remaining piles will not interfere with installation of the new piles.

Each abutment will be supported on seven piles. Based on information provided by Hoyle, Tanner, the bridge designer, we understand the maximum factored axial pile load is 320 kips for the strength loading condition and a 7-pile configuration. Hoyle, Tanner estimated that thermal deformation of the bridge superstructure could result in 1.5 inches of lateral deflection at each abutment for the Strength I condition.

An approximate 4-foot raise in grade will be required at the existing abutments to facilitate estimated river flood levels and required freeboard.

It is planned to construct a temporary bridge to the east of the existing bridge to avoid a bridge closure and accelerated bridge construction (ABC). Two alternatives will be allowed for in the construction documents, including a single-lane temporary bridge with alternating, signalized one-way traffic or a two-lane temporary bridge. Temporary approach embankments will be constructed as a widening east of the existing embankments.

¹ Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).



Additional details of the proposed permanent and temporary approach embankments are provided in **Section 5.2** of this report.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Conducted site visits to observe surficial conditions during preliminary design and reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data;
- Coordinated and observed subsurface explorations, consisting of three test borings and two piezocone penetration tests (CPTs), to evaluate subsurface conditions during preliminary design, and one supplemental test boring during final design;
- Conducted a laboratory testing program to evaluate engineering and index properties of the site soils;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability and settlement of temporary and permanent approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD 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 structural elements to improve embankment stability, 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

Details of the previous and current subsurface explorations are provided in the following sections.

2.1 PREVIOUS (1950) BORINGS

In 1950, MaineDOT conducted five test borings, designated B-1 through B-5, to explore subsurface conditions for the current bridge. Boring B-1 met refusal at a depth of approximately 7 feet below grade on a boulder, and boring B-5 was conducted only to collect tubes of clay. Therefore, borings B-2 through B-4 provide data to characterize the subsurface stratification for design consideration. The locations of borings B-2 through B-4 were estimated by measuring from features of the existing bridge as shown on **Figure 2**. Therefore, the locations should be considered approximate. The borings were advanced using drive-and-wash drilling techniques into bedrock to depths ranging from approximately 46 to 65 feet below grade. The boring log sheet from the 1950 geotechnical report is included as **Appendix B**.



2.2 RECENT BORINGS

One boring was drilled behind each existing abutment near planned replacement bridge abutment locations during preliminary design (BB-GLR-101 and -102), and a third boring was drilled at proposed Abutment 1 during final design (BB-GLR-201). The as-drilled locations and elevations of BB-GLR-100 series borings were surveyed by Maine Department of Transportation (MaineDOT) and provided to GZA, and the as-drilled location of BB-GLR-201 was estimated by taping from known site features and should be considered approximate. The elevation was estimated by interpolation between contours on the site plan and is also considered approximate. The boring locations are shown on **Figure 2**.

The borings were drilled to depths of approximately 48 to 63 feet below ground surface (BGS) and terminated approximately 5 to 12 feet into bedrock. New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between August 16 and August 20, 2018 and on August 29, 2019 using a truck-mounted drill rig. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

The borings were drilled using 4-inch and 3-inch driven casing and drive-and-wash drilling techniques with a track-mounted drill rig. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler, driven with automatic hammers with rated hammer efficiency factors of 0.937 and 0.895, as noted on the logs. *In situ* field vanes were taken at approximately 10-foot typical intervals within the clay layer. Two field vanes were completed at each depth interval using a 65mm by 130mm rectangular vane, spaced 1 foot apart, and a total of eight pairs of vanes were completed. Additional vanes were attempted but could not be advanced into the clay. Five thin-walled tube samples were taken for laboratory testing to evaluate clay properties. Between 2.5 and 10 feet of 2-inch-diameter rock core was collected from the borings using an NQ or NX2 core barrel. At the completion of drilling, the borings were backfilled with cuttings and asphalt cold patch.

2.3 CONE PENETRATION TESTING

GZA retained Summit Geoenvironmental Services (SGS) to complete two CPTs, designated CPT-GLR-101 and -102, on August 16, 2018. CPT-GLR-101 was conducted to the west of the north approach embankment, near the riverbank level, and CPT-GLR-102 was conducted within a few feet of boring BB-GLR-102 after pre-augering through the pavement section. The as-drilled CPT locations were surveyed by MaineDOT and provided to GZA and are shown on **Figure 2**.

The CPTs were performed in accordance with ASTM D5778. Shear wave velocity measurements were performed at rod break intervals of 3.3 feet in accordance with ASTM D7400. CPTs were advanced using a rubber-track-mounted PowerProbe 9500 VTR with a Vertek digital cone to a depth of push refusal encountered at depths of 46 and 58 feet BGS for CPT-GLR-101 and CPT-GLR-102, respectively. Parameters obtained include cone resistance (q_c), sleeve friction (f_s), piezocone pore pressure (u_2), and shear wave velocity (V_s). A dissipation test was conducted at CPT-GLR-102 at a depth of 32 feet BGS.

Field vane shear tests were conducted at CPT-GLR-101 at 4-foot intervals from a depth of 16 to 40 feet BGS in accordance with ASTM D2573 using a Geonor field vane. A data report was submitted to GZA by SGS containing the CPT results, dated August 25, 2018, which is included in **Appendix D**. SGS also



provided GZA with excel files containing the actual data collected from the CPTs for use in our engineering evaluation. GZA used the computer program CPet-IT 3.0 by Geologismiki to develop interpreted engineering properties from the raw CPT data. The interpretation report generated from CPet-IT is also included in **Appendix D**.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering of Cranston, Rhode Island to complete a laboratory testing program to assess the gradation and index properties of the soil. The program included: seven (7) gradation analyses / AASHTO Classifications / USCS / MaineDOT Frost Classification, three (3) Atterberg limits tests and 14 moisture content tests.

GZA also retained GeoTesting Express of Acton, Massachusetts to complete a supplemental soil testing program on tube samples from the borings to assess the engineering characteristics of the clay encountered at the site. The program included: two (2) incremental consolidation tests, two (2) controlled rate of strain consolidation tests, four (4) moisture content tests, and four (4) Atterberg limits tests.

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

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping², the surficial unit in the vicinity of the bridges consists of the Presumpscot Formation, described as gray-blue, silty clay. Marine sand and gravel deposits and glacial till deposits were also mapped in the area of the bridges.

Bedrock in the vicinity of the site is mapped³ as the Vassalboro Formation, consisting primarily of metamorphic deposits of Schist and Quartzite.

4.2 SUBSURFACE PROFILE

Up to five soil units were encountered in the test borings and CPTs overlying bedrock: Fill (Sand), Fill (Clay), Clayey Sand, Marine Clay and Sand. Approximately 1 foot of asphalt pavement was encountered in the roadway borings. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. It is noted that stratum types and thicknesses were interpreted from previous boring logs when they could be determined. Soil descriptions are based on GZA borings and CPTs, except as noted in the table. Detailed descriptions of

² Thompson, Woodrow B., 2008. Surficial geology of the Portland West quadrangle, Maine: Maine Geological Survey, Open-File Map 08-16, map, scale 1:24,000. *Maine Geological Survey Maps*. 2019. http://digitalmaine.com/mgs_maps/2019

³ Osberg, P.H., Hussey, A.M., and Boone, G.M., 1985, Bedrock geologic map of Maine: Maine Geological Survey, Dept. of Conservation, scale 1:500,000.



GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill (Sand)	3 to 8	Brown, loose to very dense, fine to coarse SAND, some to little Gravel, little Silt, dry to moist. (USCS: SP-SM, SC). MaineDOT Frost Classification: II-III <i>Encountered in borings BB-GLR-101, BB-GLR-102, BB-GLR-201, B-2 and B-4.</i>
Fill (Clay)	6 to 10	Brown and grey, very soft to very stiff, Silty CLAY, trace to some Sand, trace Gravel. (USCS: CL) MaineDOT Frost Classification: IV <i>Encountered in borings BB-GLR-101, CPT-GLR-101, BB-GLR-102, BB-GLR-201, and potentially in previous borings.</i>
Clayey Sand	4 to 10	Gray, very loose, Clayey fine SAND, occasional root fragments and wood. (USCS: SC) MaineDOT Frost Classification: IV <i>Encountered in borings CPT-GLR-101, BB-GLR-102, BB-GLR-201, and B-3.</i>
Marine Clay	17 to 34	Gray, medium stiff to soft, CLAY (upper 5 to 10 ft) to Silty CLAY. (USCS: CH, CL) MaineDOT Frost Classification: IV <i>Encountered in all borings.</i>
Sand	3 to 37	Gray, red and black, medium dense, fine to coarse SAND, some gravel, little silt. (USCS: SM) <i>Encountered in borings BB-GLR-201, B-2, B-3 and B-4, and possibly at bottom of CPT-GLR-101.</i>
Top of Bedrock Elevation	<u>Encountered Top of Rock:</u> Abutment 1: Approximate El. 56 to El. 46 Abutment 2: Approximate El. 18 to El. 30	

- Nine (9) water content tests: 58 percent to 39 percent (decreases with depth)
- Six (6) Atterberg limits tests: LL=56-38, PI=29-16 (decreases with depth)
- 16 In-situ Field Vanes; Peak Shear Strength=481 to 1,030 psf, Residual Shear Strength=27 to 253 psf
- Four consolidation tests:
 - Preconsolidation Pressure=4,200-6,600 psf
 - Virgin Compression Ratio (CR)=0.24-0.28 (average=0.26)
 - Recompression Ratio (RR)=0.013-0.028 (average=0.022)
 - Coefficient of Consolidation (Cv)=0.087-2.6 ft²/day (average=0.59 ft²/day for overconsolidated clay)



4.2.1 Bedrock

Bedrock was cored in all current borings and in previous borings B-2 and B-3. Bedrock descriptions were not provided for the previous borings. Bedrock was described as hard to very hard, fresh to slightly weathered, fine to coarse grained, gray and white Schist. In general, the joints are described as very close to moderately spaced, low to high angle, smooth to rough, planar to stepped, fresh to discolored, and tight to open with minor gray silt infilling. The Rock Quality Designation (RQD) in the core runs ranged from 18 to 100 percent. Wet and dry photographs of the rock core are presented in **Appendix G**.

4.2.2 Groundwater

The groundwater level was measured in the completed borings after drilling at depths of 7.5 to 20.4 feet BGS, corresponding approximately to El. 84 to 71. Water levels measured in the borings were likely influenced by the addition of drill water during rotary wash drilling. The groundwater level was also interpreted from the CPTs to be at depths of 5.7 to 14.4 feet BGS, corresponding approximately to El. 70 to El. 73. The CPT measurements are considered more representative of in-situ groundwater conditions.

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

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition (herein designated as AASHTO) and the MaineDOT Bridge Design Guide (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix F**.

5.2 APPROACH EMBANKMENTS

The roadway profile will be raised approximately 4 feet above existing grades in the vicinity of the bridge. The roadway will also be widened, but the top of the existing embankment is relatively level several feet past the outside edges of the existing pavement, so the maximum permanent new fill thickness is on the order of 4 feet over the embankment footprint.

Approach embankment side slopes will be constructed with an inclination of 2 horizontal to 1 vertical (2H:1V) or less, except in the northern 40 feet of the east (Right) side of the south approach, where a 1.75H:1V riprap-covered slope will wrap around from beneath the bridge to the approach side slope. The embankment will be up to approximately 28 feet high at the northeast corner of the south approach. Other than this location, the maximum proposed embankment height is approximately 17 feet.



The ground surface in front of each abutment will slope down to the river level at an inclination of 1.75H:1V and will be protected by riprap. Based on the proposed bridge length, this will require demolition of the existing abutments and removal of soil and/or rockfill extending approximately 20 feet into each existing riverbank, resulting in 18- and 13-foot-tall, 1.75H:1V riprap slopes in front of each abutment.

The temporary embankments will be contractor-designed. Hoyle, Tanner provided possible grading for a two-lane detour for GZA's design considerations. The plans provided by Hoyle, Tanner show approach fills for the temporary bridge up to approximately 2 feet above existing approach pavement grades, corresponding to 25 and 15 feet above grade to the east of the existing Abutments 1 and 2 approach embankments, respectively, which will temporarily widen the embankments by about 50 feet. We assume that the fill placed for the temporary bridge will be removed at the end of construction. We assume the embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over the existing embankment/side slopes. It is anticipated that temporary embankment slope inclinations of 2H:1V or flatter would be used.

5.2.1 Soil Profile and Properties

The typical interpreted soil profile beneath each existing embankment is summarized below. The layering is primarily based on the results of the recent borings and CPTs and varies in GZA's analyses that incorporate two- and three-dimensional subsurface models. The strength parameters for each soil unit were developed using the corrected blow count data for granular soil, *in situ* shear vane and CPT results for cohesive soil, and soil types from the borings.

DESIGN SOIL PROFILES, PROPOSED APPROACH FILLS NEAR ABUTMENTS				
Soil Unit	Total Unit Weight (pcf)	Effective Friction Angle (deg) / Undrained Shear Strength (psf)	Estimated Thickness ¹ (feet)	
			South Approach/ Abutment 1 (BB-GLR-101, 201, B-4)	North Approach/ Abutment 2 (BB-GLR-102, CPT-GLR-101)
Proposed Fill (Common Borrow, Sand, Gravel)	125	33	4	4
Existing Fill (Sand)	125	34	8 - 9	13
Existing Fill (Clay)	115	S _u = 400 psf	9 - 11	0
Clayey Sand	110	28	0	11
Marine Clay	105-115	S _u = 550-1,000 psf	17 - 18	34
Sand/Till	125	34	0 - 8	0

Note:

1. Estimated thicknesses of fill strata are measured from existing approach embankment pavement grades.

The measured and correlated shear strengths are plotted for the borings and CPTs on **Figure 4**. The undrained shear strength of the clay fill appears to vary widely within the stratum, and 400 psf is considered a suitable lower-bound strength to characterize the entire deposit. Measured and correlated undrained shear strengths in the marine clay vary from about 500 to 1,200 psf. At the south approach, the clay stratum is thinner and had higher field vane strengths than the north approach. The north approach includes a higher strength region in the upper portion of the deposit, a lower strength region approximately 10 to 20 feet below the top of the deposit, and increasing strengths up to about 1,000 to



1,200 psf at the bottom of the deposit. Considering the elevation difference between the top of clay at the north and south approaches, a single design shear strength profile was considered suitable for the site. Design undrained shear strength values were selected based on the field vane data and CPT interpretations and are presented on **Figure 4**. At the north approach, where CPTs and field vanes were conducted through and east of the existing embankment, the data indicate that the shear strength profile is generally consistent as a function of elevation beneath and outside (east) of the existing embankment footprint, which is reflected in our design shear strengths and assumed to be true at the south approach.

The results of four consolidation tests (one at the south approach and three at the north approach) and correlations of the CPT data were used to evaluate the compressibility characteristics of the marine clay. The measured and correlated maximum past pressure values are plotted along with the existing effective stress within and outside of the current embankment footprint on **Figure 4**. The available data indicate that the clay is lightly to moderately overconsolidated throughout the depth of the stratum at both approaches. As with the shear strength, the maximum past pressure appears to be approximately the same both beneath and outside the existing embankments at the north approach. The area outside the embankment footprint at the north approach was judged using the CPT correlations. We developed design maximum past pressure profiles, shown on **Figure 4**, that indicate the maximum past pressure is at least 200 psf greater than the effective overburden pressure beneath the existing embankment and at least 600 psf greater than the effective overburden pressure east of the existing embankment.

5.2.2 Embankment Settlement

Considering the maximum increase in effective stress will be approximately 500 psf under 4 feet of new embankment fill, and the temporary approach embankment will be lower than the final approach embankments, the final effective stress under new fill will be less than the maximum past pressure, and the clay will remain in the overconsolidated range. In addition, because the clay will remain overconsolidated, we anticipate that secondary compression will be negligible (approximately $\frac{1}{4}$ inch or less). Therefore, settlement will occur as recompression. However, recompression settlement could incur downdrag loading on new piles, and it would also result in differential settlement between the embankments and the temporary bridge and the replacement bridge.

GZA developed a three-dimensional model for analysis of future settlement using existing and proposed grading supplied by Hoyle, Tanner, subsurface stratification based on the test boring logs and CPTs, and soil properties developed from the laboratory and field testing as previously described. Where necessary, the subsurface conditions beyond the limits of the test borings were extrapolated to provide a complete subsurface model in the areas of interest.

GZA evaluated the approach fill settlement using the analytical software Settle3D by Rocscience. Based on our discussion with Hoyle, Tanner, the following stages were considered in the settlement analyses:

1. Existing Condition, Temporary Approach Fill Complete and Temporary Bridge Open: Begins with existing approach embankments in place; stress history is based on current conditions (see **Figure 4**). The change in effective stress during this stage is based on the 15- to 25-foot embankments for the two-lane temporary bridge. Elastic settlement is assumed to occur during fill placement; after that only consolidation settlement is considered.



2. Permanent Fill Complete and Piles Installed: 4± feet of permanent fill is added over existing embankments, with temporary embankment fill still in place. This is modeled to cause settlement immediately after the temporary bridge is open. Settlement that affects new pile foundations for downdrag begins with the fill placement in this stage. This is complete about 65 days after beginning of construction. *Please note that this assumes the temporary bridge is pile supported and does not induce a stress increase in the soil.*
3. New Bridge Open and Temporary Approach Fill Removed: Based on discussion with Hoyle, Tanner, it is assumed that the temporary fill will be removed 335 days after start of construction, leaving the embankment in the final condition.
4. End of 75-year Design Life: Final time considered for downdrag.

Our settlement evaluation considered three locations beneath each approach fill, including:

1. Center of the temporary bridge approach embankment, representing the maximum anticipated settlement during use of the temporary bridge (relevant to roadway serviceability during construction);
2. Edge of the new roadway nearest to the temporary bridge approach at the abutment, representing the largest settlement that would occur at the location of a new pile (relevant to downdrag considerations); and
3. Center of the new roadway near/behind the abutment, representing differential settlement between the abutment and roadway (relevant to long-term roadway serviceability).

The results of the Settle3D analyses are presented in **Appendix F** and summarized in the table below:

RESULTS OF SETTLEMENT ANALYSES		
Abutment / Approach	Location (Stage/Time)	Estimated Settlement (inches, nearest 0.25")
South Temporary Approach	Center of Embankment (End of Temp Bridge)	5
Abutment 1	East Side of Abutment (End of Temp Bridge)	0.5
Abutment 1	East Side of Abutment (75 yrs)	0.25
Abutment 1 Approach	Center of Embankment, south of Abutment (75 yrs)	0.25
North Temporary Approach	Center of Embankment (End of Temp Bridge)	1.75
Abutment 2	East Side of Abutment (End of Temp Bridge)	0.75
Abutment 2	East Side of Abutment (75 yrs)	0.25
Abutment 2 Approach	Center of Embankment, North of Abutment (75 yrs)	0.5

Considering that greater than 0.4 inches of settlement is predicted at new pile locations supporting both abutments at the end of the temporary bridge phase (when the piles will be installed), the piles will be subject to downdrag loading from movement of a portion of the marine clay and overlying strata relative to the piles. The upper approximately 20 feet of the profile at Abutment 1 and 35 feet at Abutment 2 will settle 0.4 inches or more and will impose downdrag loading. Downdrag loads are discussed further in **Section 5.4**.



Based on the estimated settlement of the temporary and permanent embankments adjacent to the foundations, we conclude differential settlement will be less than 1 inch between the permanent paved roadway and the bridge, which is considered acceptable, assuming an approach slab is used to distribute differential settlement at the permanent bridge. Settlement ranging from 2 to 5 inches may occur at the temporary bridge approaches.

5.2.3 Embankment Slope Stability

GZA evaluated the stability of both approaches at critical cross-sections near the proposed abutments and in the longitudinal direction beneath each abutment using the cross-sections and properties shown in **Appendix F**. The basis for acceptable performance for embankments is specified in AASHTO Article 11.6.2.3 and is summarized as follows:

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

Evaluations were conducted using the computer analytical software *Slope/W 2018*, developed by Geo-Slope International, based on the Modified Bishop method. A grid and radius search technique was used to identify the slip surface with the lowest factor of safety. A 250-psf surcharge load was also included within the limits of the proposed travelway. Slope/W output figures showing the minimum factor of safety for each analysis are presented in **Appendix F**. The plotted contours above the slope indicate relative factors of safety associated with center points of the analyzed circular surfaces. Additional details of the analyses and results are presented below.

Longitudinal Profiles

Static Analysis

The analyzed profiles considered the interpreted typical subsurface conditions along the project baseline. The beneficial reinforcing effect of timber piles supporting the existing abutments and new HP14x89 piles were modeled in Slope/W. The Brom's method was used to calculate the available resistance from the piles, which is based on the stiffness of the embedded pile and the strength of the soil. Seven, HP14x89 piles were modeled beneath each abutment, corresponding to a center-to-center spacing of about 5.7 feet, oriented for strong-axis bending. Brom's method pile resistance calculations are presented in **Appendix F**. The calculated lateral pile resistance was entered in Slope/W using a resistance value per pile and a pile spacing.

The results indicate a minimum factor of safety of 1.5 for the Abutment 1 profile and 1.8 for the Abutment 2 profile. Therefore, the global stability is acceptable in the longitudinal direction.

Seismic Analysis

A pseudostatic analysis was conducted at Abutment 1 including earthquake loading based on the Newmark method by incorporating a horizontal seismic coefficient, k_h , defined as half of the maximum peak ground acceleration at the ground surface (0.114 g). Although the integral abutments will be rigid and generally



non-yielding during an earthquake, the deep nature of the critical stability surface is such that the abutment is not considered likely to prevent deformation of the overall soil mass above the slip surface, allowing use of half of the peak ground acceleration for the pseudostatic seismic coefficient in accordance with AASHTO Article 11.6.5.2.2. For this analysis, the stability is evaluated for the slip surface that is shown to have the lowest safety factor in the static analysis, and the basis of the calculation is that a factor of safety of at least 1 is adequate to protect the embankment from large deformations. The calculated pseudostatic factor of safety against rotational failure is approximately 1.1 for the critical slip surface beneath Abutment 1, indicating that significant slope movement is not likely.

Cross-Sections

Static Analysis

Two cross-sections were selected for analysis, consisting of the highest embankment and/or steepest side slope inclinations in proximity to each abutment. The analyzed geometries and results are summarized in the table below.

SUMMARY OF GLOBAL STABILITY CROSS SECTION EVALUATION			
Abutment / Approach	Proposed Embankment Height	Proposed Slope Inclination	Minimum Factor of Safety (Static)
Abutment 1, Right	28 feet	1.75H:1V	1.0
Abutment 2, Left and Right	16 feet	2H:1V	1.3

Based on the results, global stability is not suitable for the steepest, highest portion of the Abutment 1 right approach, and it is acceptable for the Abutment 2 approaches. The Abutment 2 evaluation is considered to show that stability is acceptable for all planned embankment inclinations of 2H:1V or flatter. The 1.75H:1V slope extends about 40 feet south of the back of Abutment 1. Therefore, this slope area was evaluated for remedial alternatives to improve the global stability.

Potential mitigation alternatives to improve stability include flattening the permanent slope inclination, utilizing lightweight fill in the embankment, constructing a stabilizing toe berm, and driving piles to provide additional shear resistance. Based on cost, constructability and permanent impact considerations, and discussions with Hoyle, Tanner and MaineDOT, pile reinforcement was selected as the preferred alternative. HP14x89 piles oriented for strong-axis bending were selected because they were already planned for use on the project. The results of our analyses indicate that four, HP14x89 piles, installed at a center-to-center spacing of 12 feet, increase the static factor of safety to 1.3 for the 28-foot-high, 1.75H:1V embankment slope. Additional recommendations for the slope reinforcement pile locations are presented in the Recommendations section.

Seismic Analysis

A pseudostatic analysis was conducted using k_h defined as half of the maximum peak ground acceleration at the ground surface (0.114 g), in accordance with AASHTO 11.6.5.2.2, because the slope will be free to move during an earthquake. The calculated pseudostatic factor of safety against rotational failure is approximately 1.0 for the critical slip surface, indicating that large slope deformations are not likely.



Temporary Detour Approaches

Planning-level stability analyses were conducted for an anticipated temporary approach embankment at the south approach. The north approach was not analyzed considering that the geometry will be similar to the permanent embankment. The required embankment height at the south approach will be similar to the permanent embankment discussed above. The results of our analyses indicate that an embankment with a 2H:1V slope will not meet AASHTO global stability criteria. Based on our preliminary analyses, we anticipate that a flattened embankment slope inclination, pile reinforcement or other means to enhance stability will be required to meet AASHTO standards at the south approach. We recommend that the Contractor be made responsible for submitting a design that includes global stability calculations to justify the proposed temporary embankment geometry.

5.3 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments), Clayey Sand, Marine Clay and underlying Sand overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average undrained shear strength in cohesive soils encountered in the borings and the measured shear wave velocities in the CPTs. The average undrained shear strength for encountered cohesive soils is approximately 600 to 700 psf, and the average shear wave velocity ranged from 510 to 630 feet per second. Site Class E is defined as an average undrained shear strength less than 1,000 psf or an average shear wave velocity less than 600 feet per second. Therefore, based on either criterion, the bridge is assigned to Site Class E.

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

5.4 EVALUATION OF FOUNDATIONS

5.4.1 Foundation Type Assessment

GZA conducted preliminary evaluations for foundation support of the integral abutments in support of Hoyle, Tanner's preparation of the Preliminary Design Report. Based on the results of our preliminary evaluations, Hoyle, Tanner selected an integral abutment bridge supported on driven steel H-piles. Design considerations and evaluations are presented below.

5.4.2 Pile Design Considerations

Based on our experience within 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 soil profile will primarily consist of medium stiff marine clay that is sensitive and will lose most of its strength temporarily during pile driving. Therefore, limited geotechnical resistance will be provided by the soil profile during driving, likely on the order of 5 to 20 percent of the required nominal resistance.

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 Hoyle, Tanner 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.

5.4.3 Load and Resistance Factors

Piles should be designed for the 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. Per AASHTO Table 10.5.5.2.3-1, the piles should therefore be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65 if one pile per abutment is tested or 0.75 if all piles in each abutment are tested.

For this project, in consideration of the potential sloping bedrock to limit the achievable geotechnical resistance and to limit the number of piles per abutment, we recommend that all piles be dynamically tested. Therefore, piles will be driven to the lower nominal resistance calculated by dividing the maximum factored pile load (Strength I load case) by a resistance factor of 0.75. Resistance factors for service and extreme limit state design should be taken as 1.0. A load factor of 1.0 is recommended for downdrag loads for strength limit state design.

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 condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per 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.

5.4.4 Pile Type

The abutments are planned to be supported on ASTM A572, Grade 50 ($f_y=50$ kips per square inch [ksi]) steel HP14x89 piles. Each abutment will include seven piles.

5.4.5 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 80 kips will occur on piles at Abutment 2, which is the maximum value for the project because of the thicker clay in this area.

Side friction contributing to downdrag load was estimated using the β -method in accordance with NAVFAC DM 7.2-211, and as recommended by Sanford et al, "Bitumen Coatings Reduce Downdrag on Piles for Route 1 Interchange Bridges." Beta values were assumed to be 0.35 and 0.25 for the Fill and



the Marine Clay, respectively. 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 Hoyle, Tanner.

5.4.6 Pile Loads

Hoyle, Tanner provided a maximum factored axial load of 320 kips per pile for the strength condition, and the factored downdrag load is 80 kips; therefore, piles should be installed to a nominal axial resistance of at least 533 kips, calculated by dividing the maximum factored axial load of 400 kips by a geotechnical resistance factor of 0.75 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted on all piles during construction in accordance with AASHTO requirements to assess nominal geotechnical pile resistance.

At the time of GZA's engineering analyses, the maximum factored axial load was understood to be 330 kips, and that load was the basis of our evaluations. Considering the small decrease in the load, GZA did not modify our evaluations for the revised load, and the basis of pile evaluations was a required nominal axial resistance of 547 kips.

5.4.7 Design-Phase Pile Drivability Analysis

GZA completed wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 547 kips. The goal of the analysis was to evaluate if a typical pile hammer could install the HP 14x89 piles to a nominal resistance of 547 kips without exceeding the allowable driving stress.

The analyzed pile lengths were selected assuming that the piles would be driven to the shallowest and deepest top of rock elevations encountered at the abutments. Estimated side resistance during pile installation was used as an input in wave equation analyses, including a range of 5 to 20 percent contribution of skin friction resistance to the maximum factored pile resistance. Analyses were completed using a Delmag D30-32 diesel hammer with a ram weight of 6.6 pounds and a rated energy of approximately 75,250 foot-pounds (ft-lbs). The results are summarized below.

DESIGN-PHASE WAVE EQUATION ANALYSIS RESULTS					
Pile Location and Type	Embedded Pile Length	Driving System ¹	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
Abutment 1 HP 14x89	26 feet	Delmag D30-32	547	42	11
Abutment 2 HP 14x89	53 feet	Delmag D30-32	547	42	10

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 noted.



5.4.8 Lateral Pile Analysis

GZA conducted lateral pile analyses using L-Pile 2016®. Hoyle, Tanner provide a maximum thermal deflection of 1.5 inches and a factored axial load of 330 kips. Downdrag was not included in the axial load as it will act below the pile head and will not contribute to the maximum pile stress. We assumed a fixed-head condition (zero rotation) and imposed the estimated thermal deflection at the pile head. The ground surface was assumed at the bottom of abutment/top of pile for thermal contraction. The analysis used ASTM A572 Grade 50, HP 14x89 piles aligned for strong-axis bending parallel to the bridge centerline.

GZA developed soil profiles for lateral pile evaluations. Considering the combination of a relatively weak soil profile, shallow bedrock, and strong-axis piles, we concluded that the shortest piles would control the pile response due to fixity considerations. Therefore, lateral pile analyses were conducted for Abutment 1. Pile lengths of 26 and 34 feet below the pile cap were analyzed using the input parameters presented in the tables below.

L-PILE® INPUT PARAMETERS						
ABUTMENT 1, PILE LENGTH = 26 FT (MIN. ANTICIPATED, BB-GLR-101)						
Stratum	Soil Model	Top of Layer Elevation (ft- NAVD 88)	Layer Thickness (ft)	k (pci) / E50	φ' (deg)/ Su (psf)	γ_e (pcf)
Clay Fill	Matlock Clay	83.0	5	0.02	400	115
Clay Fill	Matlock Clay	78.0	4	0.02	400	53
Marine Deposit	Matlock Clay	74.0	9	0.008	900	53
Marine Deposit	Matlock Clay	65.0	8	0.008	800	43

L-PILE® INPUT PARAMETERS						
ABUTMENT 1, PILE LENGTH = 34 FT (ADDITIONAL CLAY THICKNESS)						
Stratum	Soil Model	Top of Layer Elevation (ft- NAVD 88)	Layer Thickness (ft)	k (pci) / E50	φ' (deg)/ Su (psf)	γ_e (pcf)
Clay Fill	Matlock Clay	83.0	5	0.02	400	115
Clay Fill	Matlock Clay	78.0	4	0.02	400	53
Marine Deposit	Matlock Clay	74.0	9	0.008	900	53
Marine Deposit	Matlock Clay	65.0	9	0.008	800	43
Marine Deposit	Matlock Clay	56.0	7	0.008	550	53

The first analysis considered a 26-foot pile where the tip was entirely in clay, neglecting any influence of the tip resistance on rock. The results for this case predict that fixity is not achieved at the pile tip. A 34-foot pile was modeled, adding 8 feet of clay profile at the bottom of the pile, and still without any influence of rock, and this was shown to be the minimum required pile length to achieve fixity at the pile tip in an all-clay profile. We subsequently considered two alternative lateral pile tip resistance cases to account for the pile being driven to bedrock, including:



1. Shear resistance at the pile tip as described in MaineDOT Technical Report ME 01-7, except that the modeled shear resistance was only 10 kips (peak resistance at 0.03 inches deflection, no increased resistance with additional deflection), compared to a maximum theoretical resistance of 70 percent of the applied pile load stated in Technical Report ME 01-7; and
2. Three-inch pile embedment in hard bedrock.

The results of GZA's analyses are summarized in the table below.

L-PILE® RESULTS (ABUTMENT 1, HP 14x89)								
Pile Length (feet)	Axial Load (kips)	Shear Force for Lateral deflection of 1.5 in. (kips)	Moment at Pile Head (kip-ft)	Depth below Pile Head to Fixity (ft)	Total Stress at Pile Head (ksi)	Bending Stress at Pile Head (ksi)	Axial Stress at Pile Head (ksi)	Estimated Deflection of Pile Tip (inches)
26	330	47.9	-379.5	--	47.4	34.8	12.6	-0.232
26*	330	50.3	-393.3	--	48.7	36.0	12.6	-0.059
26**	330	51.3	-399.8	--	49.3	36.6	12.6	-0.003
34	330	51.1	-399.2	33.0	49.2	36.6	12.6	0.002

Notes:

1. The bending stress at pile head is calculated as the maximum moment divided by the pile section modulus.
2. * Indicates Pile was evaluated with a shear tip resistance of 10 kips at a deflection of 0.03" to 5" (plastic response beyond 0.3"). The Lpile model converged at 0.06" deflection for this case.
3. ** Indicates Pile was evaluated with 3" embedment into bedrock.
4. Negative deflection of pile tip indicates opposite direction from pile head deflection, positive indicates same direction.

The results indicate that a very small amount of shear resistance at the pile tip makes the piles behave similarly to a pile that has gained fixity, judged by the similarity of the calculated shear force and moment between the case with 10 kips of shear resistance, 3 inches embedment in rock, and a 34-foot pile. Therefore, a 26-foot-long pile driven to the required axial pile resistance should experience acceptable performance needing only minimal influence of rock to enhance lateral deformation. The results also indicate that the combined stress will remain within the elastic range for piles oriented in the strong axis direction.

Boring BB-GLR-201 was drilled to further evaluate possible Abutment 1 pile lengths. It encountered rock approximately 10 feet deeper than BB-GLR-101, with sand at the bottom of the profile. These results indicate a more favorable profile for lateral resistance than was evaluated herein. For design purposes the average pile length of about 31 feet could be considered for this abutment.

5.4.9 Lateral Earth Pressures

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



Based on the estimated thermal bridge expansion of 1.5 inches and the abutment height of approximately 13 feet, the calculated abutment rotation is 0.0096 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 Coulomb passive earth pressure is appropriate for design. We recommend using a Coulomb Coefficient of Passive Earth Pressure, $K_p = 6.73$ for design of backwalls and wingwalls. Design lateral earth pressure recommendations are provided in **Section 6.3** of this report. AASHTO Commentary C3.10.9.1 specifies that single-span bridges in Seismic Zone 2 are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

5.4.10 Frost Protection

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,300, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 6.4 feet.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN AND REINFORCEMENT CONSIDERATIONS

Typical embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter, except in front of integral abutments where the typical 1.75H:1V slope angle will be used. Slopes should be provided with loam and seed for permanent erosion protection. Where steeper slopes are planned at the Abutment 1 approach, 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 Little River to protect from scour.

We recommend that four, HP14x89 piles be installed, oriented with the strong axis parallel to the downslope direction, for embankment slope stabilization at the south approach. The pile nearest to the abutment should be located with the center of the pile 4 feet behind the back of the abutment, and the remaining four piles should be aligned with the baseline and installed at a 12-foot center-to-center spacing. The piles should be installed at an offset of 21' Right, at the following stations: Sta. 116+85, 116+73, 116+61, and 116+49. The piles may be installed using a vibratory or impact hammer, with final driving to a final penetration resistance of 6 blows per inch for 3 inches using a hammer of similar energy to that needed to drive the foundation piles. The slope stabilization piles should be fitted with cast steel pile tips to limit driving damage. Based on the results of boring BB-GLR-201 and pile top embedment approximately 5 feet below finished grade, an estimated in-place pile length of 50 feet is considered appropriate for the plans.

6.2 SEISMIC DESIGN

The United States Geological Survey Online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class E) for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:



SITE CLASS E SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	2.5
Fa	2.5
Fv	3.5
As (Period = 0.0 sec)	0.227 g
SDs (Period = 0.2 sec)	0.450 g
SD1 (Period = 1.0 sec)	0.160 g

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Coulomb Coefficient of Passive Earth Pressure, $K_p = 6.73$ (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. Alternatively, prefabricated drainage geocomposite material can be placed against the uphill side of abutments, after holes have been created through the backing material at the weep hole locations.

6.4 PILE DESIGN

- 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 the bedrock surface.
- To promote penetration into rock and limit driving damage, the Abutment 1 steel H-piles should be fitted with HP Rock Injector pile tips, which have a chisel-like point intended to limit the potential for the pile to slide on a sloping rock surface. The Abutment 2 steel H-piles should be fitted with traditional cast steel pile tips to limit driving damage.



- 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 533 kips, calculated by dividing the maximum factored pile load of 400 kips (320 kips Strength 1 plus 80 kips downdrag) by a resistance factor of 0.75.
- Preliminary wave equation analyses indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 75,250 ft-lbs for the anticipated 26- to 53-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9F_y for 50 ksi steel), and with a final penetration resistance of 10 to 11 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. 46 to El. 56 at Abutment 1, and approximately El. 18 to El. 30 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 all of the piles at each abutment 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 two piles at each abutment (four total), to be selected by the Geotechnical Engineer based on observations during initial driving. In the event that the initial driving results are favorable and show sufficient excess resistance to allow for some relaxation, one or both restrike tests may be waived.
- Piles may be installed through the Marine Clay stratum by vibratory methods. Vibratory installation should be terminated when an increase in resistance is detected, and the pile should be monitored dynamically for throughout the remainder of the driving.
- Piles shall be spliced in accordance with MaineDOT Section 501.047.
- Integral abutment should 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 $\phi_{cc}=0.7$ 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).
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankment to the bridge. The slabs should be positively connected to the backwalls.

7.0 CONSTRUCTION CONSIDERATIONS

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



7.1 ABUTMENT PILE INSTALLATION CONTROL

We recommend that the 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, piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.75. Therefore, all piles should be dynamically tested during initial drive, and restrike tests should be completed on two piles per abutment, unless waived by the Geotechnical Engineer.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

We anticipate that temporary support of excavation may be necessary to maintain portions of the existing or temporary relocated roadway. It is anticipated that sheet pile walls will be required in the vicinity of the bridge in areas where grade separations exceed approximately 4 feet.

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

7.3 PILE OBSTRUCTIONS

Although no evidence of cobbles and boulders was seen in the Fill in the test borings, it is possible that pre-drilling, pre-excavation or spudding may be necessary to bypass potential obstructions, such as boulders, rock fill, timber cribbing or existing foundations.

7.4 REUSE OF ON-SITE MATERIALS

Based on the test boring results, only one of the three upper fill samples tested had less than 10 percent passing the No. 200 sieve, and the deeper fills contain considerable silt and clay, indicating the fill generally will not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and would not be suitable for use as structural backfill. The material is considered suitable for use as Common Borrow.

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

7.5 TEMPORARY BRIDGE AND APPROACH EMBANKMENTS

The Contractor's submitted design computations and plans shall demonstrate that the temporary structure and approaches achieve acceptable minimum factors of safety for slope stability. Minimum factor of safety shall be 1.3 for approach embankment slopes and 1.5 for embankment slopes that contain or support an abutment. The design computations shall be conducted by a geotechnical consulting firm from the MaineDOT Prequalified List (Category 804.10) retained by the Contractor. The



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computations shall be stamped by a professional engineer licensed in the State of Maine. Construction shall not begin on the temporary approach embankments prior to review and acceptance of the design computations by MaineDOT.

As noted in **Section 5.2.2**, approximately 5 inches and 2 inches of settlement is predicted at the temporary approach fills adjacent to Abutments 1 and 2, respectively. Pavement shimming may be needed to maintain serviceability of the approaches to the temporary bridge if the bridge is pile supported.

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TABLES

TABLE 1
SUMMARY OF SUBSURFACE EXPLORATIONS
Little River Bridge #3112, Route 237 over Little River
Gorham, ME

	GZA TEST BORINGS AND CPTS				PREVIOUS BORINGS		
	BB-GLR-101	BB-GLR-201	CPT-GLR-101	CPT/BB-GLR-102	B-2	B-3	B-4
Nearest Abutment	1	1	2	2	2	2	1
<u>Depth (ft) to:</u>							
Fill (Sand)	1	1.1	NE	1	1	ND	0
Fill (Clay)	8.5	9.1	1	4	5	ND	NE
Clayey Sand	NE	NE	11	13.5	NE	ND	NE
Marine Clay	18.5	20	15	23.5	11	13	4
Sand	NE	37.5	46 **	NE	36	31	28
Bedrock	35.7	45.1	ND	57.7	39	48.3	64.7
Groundwater	7.5	20.4	5.7	14.4			
Bottom of Exploration	48	55.1	46	62.5	46.4	55.8	65.2
<u>Thickness (ft) of:</u>							
Fill (Sand)	7.5	8	0	3	4	ND	4
Fill (Clay)	10	10.9	10	9.5	6	ND	NE
Clayey Sand	NE	NE	4	10	NE	ND	NE
Marine Clay	17.2	25.1	31	34.2	25	18	24
Sand	NE	7.6	ND	NE	3	17.3	36.7
<u>Approximate Elevations (ft):</u>							
Ground Surface ²	91.5	91.5	76	87.5	75.1	74	74
Top of Fill (Sand)	90.5	90.4	NE	86.5	74.1	ND	74
Top of Fill (Clay)	83	82.4	75	83.5	70.1	ND	NE
Top of Clayey Sand	NE	NE	65	74	NE	ND	NE
Top of Marine Clay	73	71.5	61	64	64.1	61	70
Top of Sand	NE	54	30 **	NE	39.1	43	46
Top of Bedrock ³	55.8	46.4	ND	29.8	36.1	25.7	9.3
Groundwater	84	71.1	70.3	73.1	--	--	--
Bottom of Exploration	43.5	36.4	30	25	28.7	18.2	8.8

NE - Not Encountered

ND - Not Determined (either due to early refusal or non-interpretation of stratum in previous boring)

** - Nature of CPT Refusal not confirmed (Sand or Bedrock)

Notes:

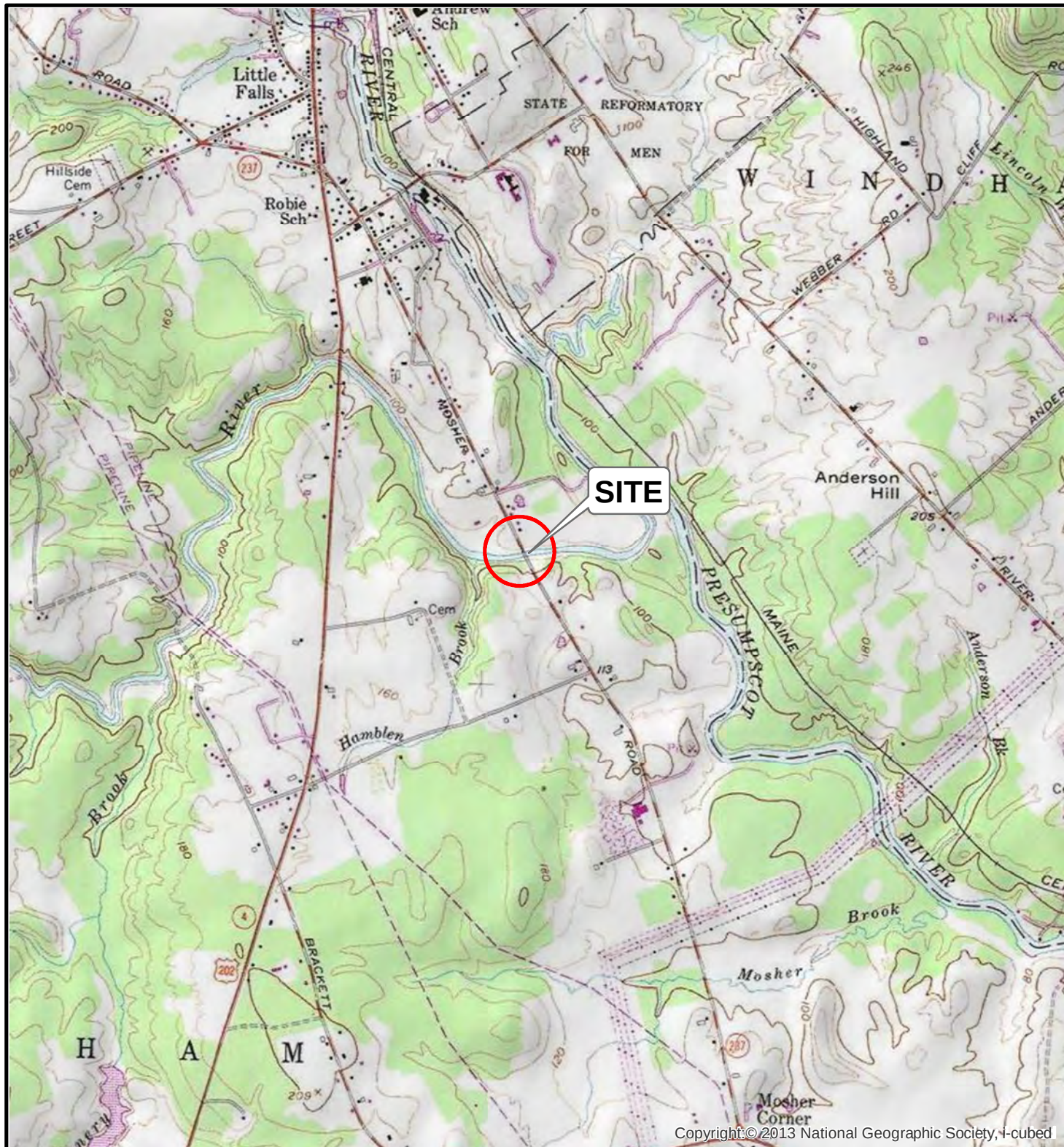
1. Ground surface elevations at the BB-GLR-100 borings and CPT-GLR CPTs were surveyed by MaineDOT and provided to GZA. Ground surface elevation at BB-GLR-201 was estimated using existing elevation contour data. Ground surface elevations at previous borings test borings were recorded in the 1950 geotechnical report.
2. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.
3. Groundwater level reported at CPT/BB-GLR-102 was measured in CPT-GLR-102.
4. See Appendix B for previous boring logs, Appendix C for BB-GLR series current boring logs, and Appendix D for the CPT report.



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

FIGURES



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

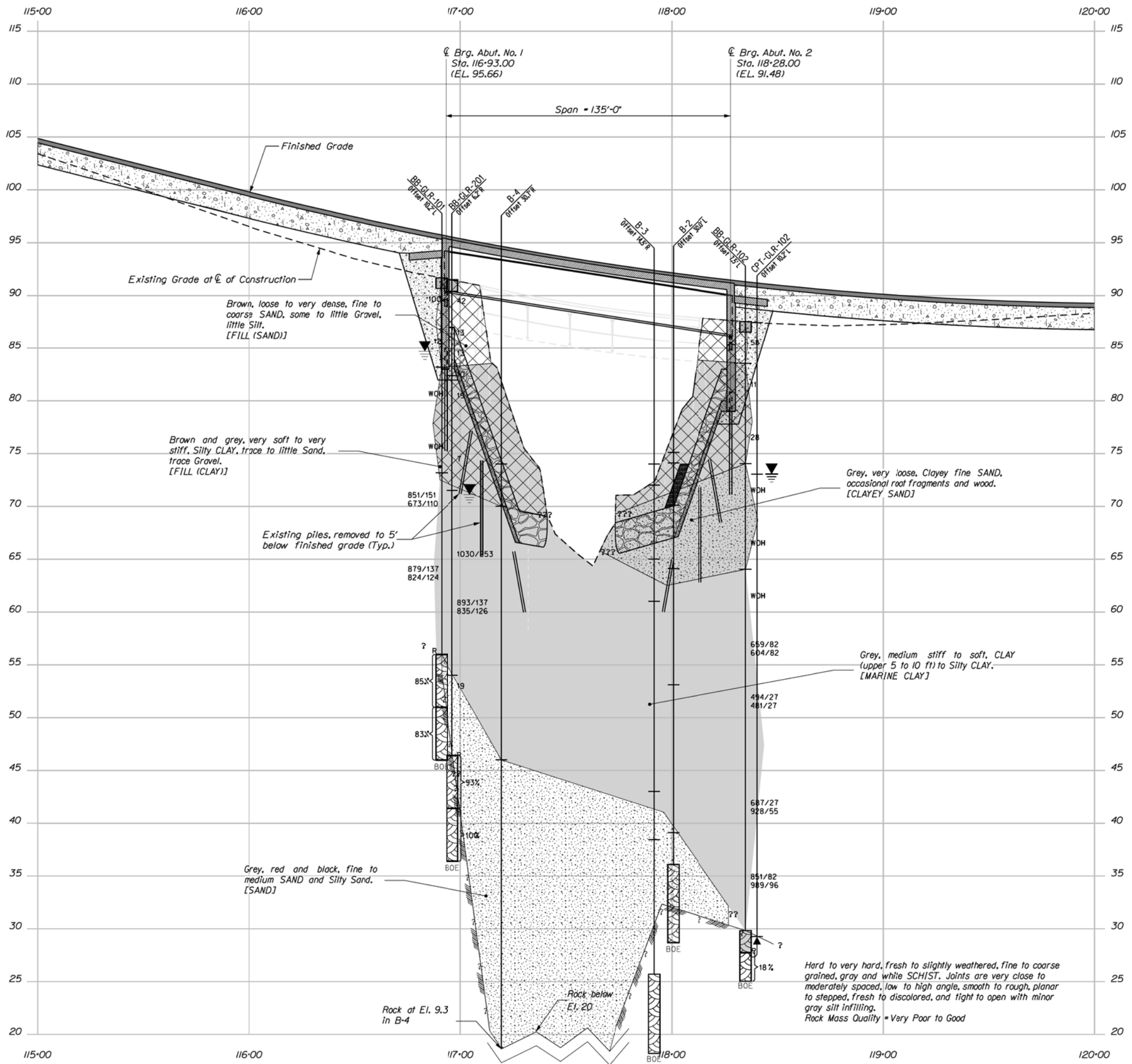


PROJ. MGR.: EDF
DESIGNED BY: EDF
REVIEWED BY: ARB
OPERATOR: EDF
DATE: 11-16-2018

LOCUS PLAN

LITTLE RIVER BRIDGE ROUTE 237
GORHAM, MAINE

JOB NO.
09.0025988.00
FIGURE NO.
1



NOTES

1) Base map developed from electronic files provided by Hoyle, Tanner & Associates, Inc. on September 4, 2019 (Files included Profiles.dgn and 003_Profile.dgn)

2) The Lower Sand stratum was not encountered in several GZA test borings. Therefore, interpreted strata boundaries and generalized description are based only on 1950 B-series boring logs and should be considered approximate.

3) 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

LEGEND

Strata Interface

Boring No. 10957 Offset 15' shown

Pavement thickness if applicable

Energy-Corrected SPT N60 Value (blows/foot)

WOH Indicates weight of hammer

85/151 Indicates in-situ field vane strength in pounds per square foot.

R Split Spoon Refusal (>50 blows for 1' penetration)

Advancement of roller cone without coring

Interpreted Top of Bedrock (See Note 3)

RQD = Rock Quality Designation for Rock Core Sample

BOE Bottom of Exploration

Encountered Groundwater Level in Boring

Bottom of Exploration, Refusal

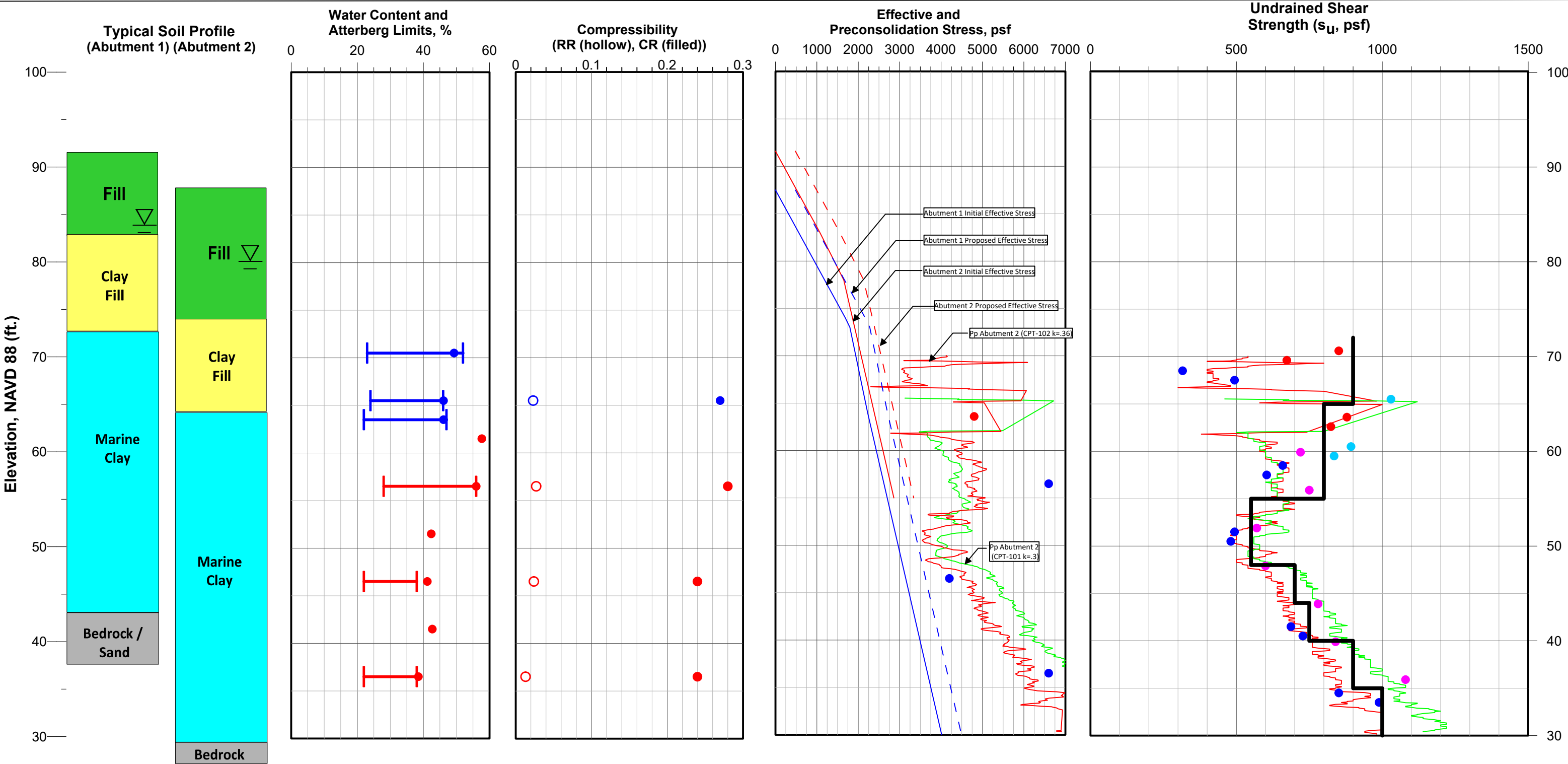
Scale of Feet

Horiz. 25 0 25 50

Vert. 5 0 5 10

LITTLE RIVER BRIDGE OVER LITTLE RIVER BRIDGE NO. 5830, MAINEDOT WIN 023909.00 GORHAM, ME			
INTERPRETIVE SUBSURFACE PROFILE			
PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: HOYLE, TANNER & ASSOCIATES, INC.	
PROJ MGR: NWV	REVIEWED BY: ARB	CHECKED BY: CLS	FIG
DESIGNED BY: NWV	DRAWN BY: NWV	SCALE: AS SHOWN	3
DATE: 10/1/2019	PROJECT NO. 09.0025988.01	REVISION NO. 0	

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	023909.00	WIN 23909.00	BRIDGE NO. 3112 BRIDGE PLANS
DATE: 9/20/2019	SIGNATURE: [Signature]	P.E. NUMBER: 10957	DATE: OCT 1, 2019
BY: NWV	CLS		
PROJ. MGR: NWV	CHECKED-REVIEWED: ARB	DESIGN-DETAILS: [Blank]	REVISIONS: 1
			REVISIONS: 2
			REVISIONS: 3
			REVISIONS: 4
FIELD CHANGES			
LITTLE RIVER BRIDGE LITTLE RIVER CUMBERLAND COUNTY GORHAM			
INTERPRETIVE SUBSURFACE PROFILE			
SHEET NUMBER 8 OF 42			



NOTES:

1. DATA BASED ON TEST BORINGS (BB-GLR-101, AND -102) PERFORMED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE BETWEEN AUGUST 16 AND 20, 2018 AND TEST BORING CPTB-GLR-101 PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE ON AUGUST 16, 2018. BORINGS OBSERVED AND LOGGED BY GZA PERSONNEL.
2. CPT EXPLORATIONS (CPT-GLR-101 AND -102) PERFORMED BY SUMMIT GEOENGINEERING OF ROCKLAND, MAINE ON AUGUST 16, 2018.
3. TYPICAL SOIL PROFILE BASED ON BORING BB-GLR-101, -102 and -201.
4. WATER CONTENTS BASED ON LABORATORY TESTS PERFORMED ON SAMPLES TAKEN FROM RECENT BORINGS.
5. PRECONSOLIDATION PRESSURE CALCULATED FROM CONSOLIDATION TESTS USING THE CASAGRANDE METHOD.
6. CORRELATED UNDRAINED SHEAR STRENGTH FROM CPT DATA IS BASED ON $N_{kt} = 22$ (CPT-GLR-101) AND $N_{kt}=20$ (CPT-GLR-102).
7. CORRELATED MAXIMUM PAST PRESSURE FROM CPT DATA IS BASED ON $k=0.3$ (CPT-GLR-101) AND $k=0.36$ (CPT-GLR-102).
8. IN LEGEND, FV=UNDRAINED SHEAR STRENGTH FROM IN-SITU FIELD VANE, CONSOL=LAB DATA FROM CONSOLIDATION TEST.

LEGEND

●

BB-GLR-101 (FV, Consol)

●

CPTB-GLR-101 (FV, Consol)

●

BB-GLR-102 (FV)

●

BB-GLR-201 (FV)

—

CPT-GLR-101 (Nkt 22, k=0.3)

—

CPT-GLR-102 (Nkt 20, k=0.36)

—

DESIGN Su PROFILE

Plastic Limit, PL

Water Content, Wn

Liquid Limit, LL

UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION DEPICTED ON THIS DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

LITTLE RIVER BRIDGE REPLACEMENT
GORHAM, ME

MARINE CLAY PROPERTIES

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers & Scientists
www.gza.com

PREPARED FOR:
MAINE DEPARTMENT OF TRANSPORTATION

PROJ MGR: NWV

DESIGNED BY: ARB

DATE: 9/18/2019

REVIEWED BY: ARB

DRAWN BY: BMC

PROJECT NUMBER: 09.0025988.01

CHECKED BY: CLS

SCALE: N/A

REVISION NUMBER: 0

FIGURE
4



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

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

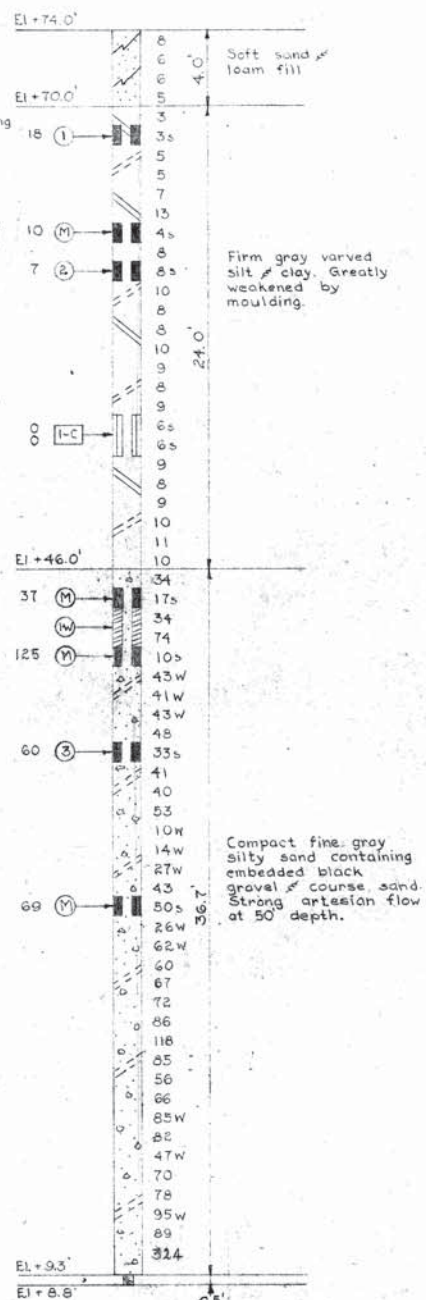
12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; 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.



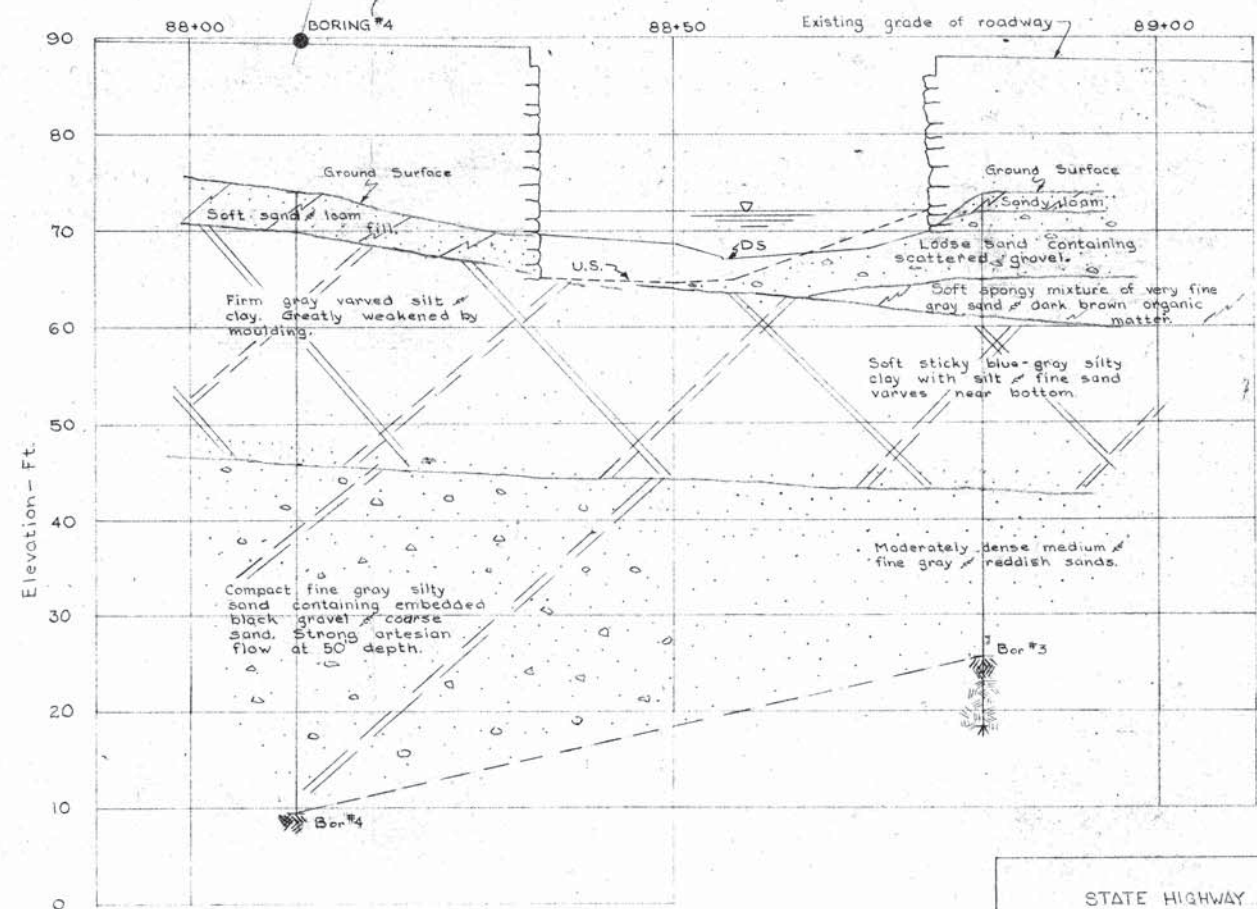
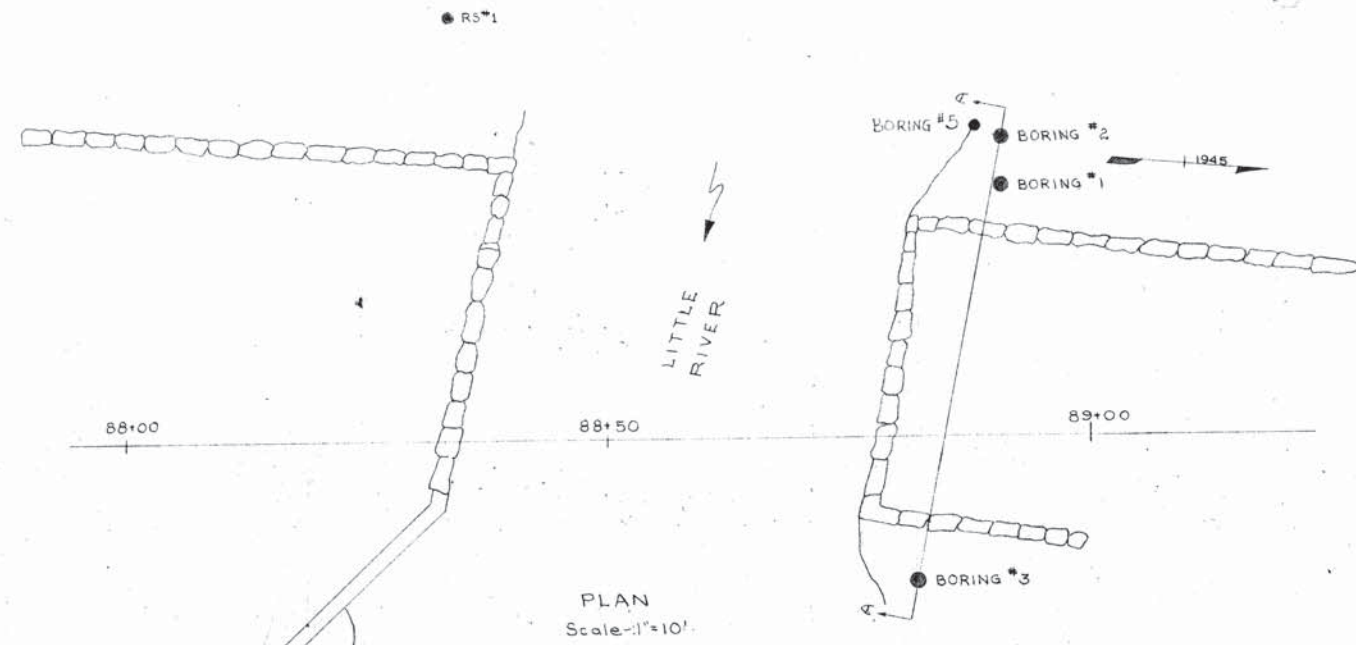
10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX B – PREVIOUS 1950 TEST BORING LOG SHEET



- NOTES -
1. Scale: 1"=5'
 2. Location ↗ designation of dry samples indicated thus ①
 3. Location ↗ designation of wash samples indicated thus ②
 4. Location ↗ designation of continuous samples indicated thus 11-1
 5. Unsuccessful attempts to secure sample indicated by M
 6. Dry samples taken in S&H sampler Catalogue No. 1290S
 7. Continuous samples taken in 2" O.D. 18 gauge seamless tubing.
 8. Figures to left of sample markers indicate number of drops of 300lb. hammer required to drive sampling spoon or seamless tubing one foot.
 9. Number of 20" drops of 300lb. hammer required to drive 2 1/2" extra heavy casing one foot indicated thus: 1/4 42
 10. "S" indicates hole was sampled ahead of casing; "W" hole was washed ahead of casing
 11. Natural water contents, given as percent of dry weight, are shown to left of sample marker.
 12. Recovery of rock core by size E diamond bit indicated thus 1/2 39%
 13. B¹ denotes refusal of drill rods or casing.



PROFILE
Scale-: 1"=10'

STATE HIGHWAY COMMISSION
BRIDGE DIVISION
LOGS OF EXPLORATORY WASH BORINGS
PLAN & STRATIGRAPHICAL PROFILE
LITTLE RIVER BRIDGE
GORHAM, MAINE
APRIL 1950
SOILS ENGINEER H. GRAY



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX C – RECENT TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM			
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES				
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Descriptive Term		Portion of Total (%)	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10		
					little	11 - 20		
					some	21 - 35		
				adjective (e.g. sandy, clayey)	36 - 50			
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY			
			GC	Clayey gravels, gravel-sand-clay mixtures.				
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).			
(little or no fines)			SP	Poorly-graded sands, gravelly sand, little or no fines.				
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures	Density of Cohesionless Soils		Standard Penetration Resistance N-Value (blows per foot)	
			SC	Clayey sands, sand-clay mixtures.	Very loose	0 - 4		
					Loose	5 - 10		
					Medium Dense	11 - 30		
	SILTS AND CLAYS (liquid limit greater than 50)				Dense	31 - 50		
					Very Dense	> 50		
					Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.			
HIGHLY ORGANIC SOILS				Approximate Undrained Shear Strength (psf)		Field Guidelines		
				Consistency of Cohesive soils	SPT N-Value (blows per foot)			
				Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates	
				Soft	2 - 4	250 - 500	Thumb easily penetrates	
			Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort		
			Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort		
			Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail		
			Hard	>30	over 4000	Indented by thumbnail with difficulty		
Desired Soil Observations (in this order, if applicable):					Rock Quality Designation (RQD):			
Color (Munsell color chart)					RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^*}{\text{length of core advance}}$			
Moisture (dry, damp, moist, wet)					*Minimum NQ rock core (1.88 in. OD of core)			
Density/Consistency (from above right hand side)					Correlation of RQD to Rock Mass Quality			
Texture (fine, medium, coarse, etc.)					Rock Mass Quality			
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)					Very Poor			
Gradation (well-graded, poorly-graded, uniform, etc.)					Poor			
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)					Fair			
Structure (layering, fractures, cracks, etc.)					Good			
Bonding (well, moderately, loosely, etc.,)					Excellent			
Cementation (weak, moderate, or strong)					Desired Rock Observations (in this order, if applicable):			
Geologic Origin (till, marine clay, alluvium, etc.)					Color (Munsell color chart)			
Groundwater level					Texture (aphanitic, fine-grained, etc.)			
					Rock Type (granite, schist, sandstone, etc.)			
					Hardness (very hard, hard, mod. hard, etc.)			
					Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)			
					Geologic discontinuities/jointing:			
					-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)			
					-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)			
					-tightness (tight, open, or healed)			
					-infilling (grain size, color, etc.)			
					Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)			
					RQD and correlation to rock mass quality (very poor, poor, etc.)			
					ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A			
					Recovery (inch/inch and percentage)			
					Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements:			
					WIN			
					Blow Counts			
					Bridge Name / Town			
					Sample Recovery			
					Boring Number			
					Date			
					Sample Number			
					Personnel Initials			
					Sample Depth			

[illegible]

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS										Project: Little River Bridge #3112 Route 237 Over Little River Location: Gorham, Maine			Boring No.: BB-GLR-102 PIN: 23909.00											
Driller: New England Boring Contractors					Elevation (ft.) 87.5					Auger ID/OD: 2.5" SSA														
Operator: T. Schafer					Datum: NAVD88					Sampler: Split Spoon														
Logged By: E. Friede					Rig Type: Mobile B-53					Hammer Wt./Fall: 140/30														
Date Start/Finish: 8-20-18					Drilling Method: Drive/Spin & Wash					Core Barrel: NQ														
Boring Location: N 320904.9, E 973923.6					Casing ID/OD: 4"/4.5", 3"/3.5"					Water Level*: 7.5'														
Hammer Efficiency Factor: 0.937					Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																			
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt										R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person					S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected					S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test				
Sample Information																								
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.													
0							SSA	86.5		-ASPHALT-														
	1D	24/18	1.0 - 3.0	13-14-23-14	37	58				Brown, dry, very dense, fine to coarse SAND, little Gravel, little Silt, (Fill).	G# S-7 A-1-b, SP-SM WC=4.4													
5	2D	24/20	5.0 - 7.0	5-3-4-3	7	11	25			Brown and grey, moist, stiff, fine to medium Sandy CLAY, trace Gravel, with sand seams, (Fill).	G# S-8 A-6, CL WC=18.6													
							35																	
							44																	
							39																	
							56																	
10	3D	24/24	10.0 - 12.0	5-8-10-12	18	28	40			Brown and grey mottled, moist, very stiff, Silty CLAY, trace fine Sand, trace Gravel, (Fill).	WC=28.3													
							39																	
							45																	
							33																	
							44																	
15	4D	24/24	15.0 - 17.0	WOH-WOH-WOH-1	--		RC			Grey, wet, very loose, Clayey fine SAND, occasional wood fragments and roots, (Clayey Sand).	G# S-10 A-6, SC WC=28.8													
	5D	24/11	18.0 - 20.0	Push thru vane			23			Grey, wet, fine to medium Clayey SAND, (Clayey Sand). 65x130 mm vane tests conducted, sample was granular, results not recorded.														
	MV	5/5	18.6 - 19.0				20																	
20	6D	24/24	20.0 - 22.0	WOR-WOH-WOH-1	--		23			Grey, wet, very loose, Clayey fine SAND, occasional roots, (Clayey Sand).	G# S-11 A-6, SC WC=25.1													
							22																	
							23																	
							31																	
							39			Top of Marine Deposit.														
25								64.0																
Remarks: 1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT standard based percentages passing specific grain sizes. 2. Automatic hammer NEBC D-24 Energy Transfer Ratio = 0.937 3. Advance SSA to 5.0'; switch to Drive & Wash, insert 4" casing. 4. Advance 4" casing to 58.5'; telescope 3" casing and spin to 60.0'. 5. Water level measurements were taken immediately after removal of the casing.																								
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 3 Boring No.: BB-GLR-102														
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.																								

Maine Department of Transportation										Project: Little River Bridge #3112		Boring No.: BB-GLR-102	
Soil/Rock Exploration Log										Route 237 Over Little River			
US CUSTOMARY UNITS										Location: Gorham, Maine		PIN: 23909.00	
Driller: New England Boring Contractors					Elevation (ft.) 87.5					Auger ID/OD: 2.5" SSA			
Operator: T. Schafer					Datum: NAVD88					Sampler: Split Spoon			
Logged By: E. Friede					Rig Type: Mobile B-53					Hammer Wt./Fall: 140/30			
Date Start/Finish: 8-20-18					Drilling Method: Drive/Spin & Wash					Core Barrel: NQ			
Boring Location: N 320904.9, E 973923.6					Casing ID/OD: 4"/4.5", 3"/3.5"					Water Level*: 7.5'			
Hammer Efficiency Factor: 0.937					Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: R = Rock Core Sample S _u = Insitu Field Vane Shear Strength (psf) S _u (lab) = Lab Vane Shear Strength (psf)													
D = Split Spoon Sample SSA = Solid Stem Auger T _v = Pocket Torvane Shear Strength (psf) W _C = water content, percent													
MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q _p = Unconfined Compressive Strength (ksf) LL = Liquid Limit													
U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit													
MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index													
V = Insitu Vane Shear Test WOR = weight of rods N ₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis													
MV = Unsuccessful Insitu Vane Shear Test attempt WQ1P = Weight of one person N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test													
Depth (ft.)	Sample Information									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	Elevation (ft.)					
25	7D	24/24	25.0 - 27.0	WOH-WOH-WOH-WOH	-		PUSH			Grey, wet, soft, CLAY, gravel and sand in tip of spoon, (Marine Deposit).	WC=57.7		
	V1	5/5	28.6 - 29.0	S _u =659/82 psf							65x130 mm vane raw torque readings:		
	V2	5/5	29.6 - 30.0	S _u =604/82 psf							V1: 24/3 ft-lbs.		
30	1U	24/20	30.0 - 32.0								V2: 22/3 ft-lbs.	C# CRC-2	
											1 U: Grey, wet, CLAY, (Marine Deposit).	A-7-6, CH	
												LL=56	
												PL=28	
												PI=28	
											WC=56.0		
35	8D	24/24	35.0 - 37.0	Push thru vane						Grey, wet, soft, Silty CLAY, (Fill).	WC=42.4		
	V3	5/5	35.6 - 36.0	S _u =494/27 psf							65x130 mm vane raw torque readings:		
	V4	5/5	36.6 - 37.0	S _u =481/27 psf							V3: 10/1 ft-lbs.		
											V4: 17.5/1 ft-lbs.		
40	2U	24/24	40.0 - 42.0							2U: Grey, wet, Silty CLAY, (Marine Deposit).	C# IP-2		
											A-7-6, CL		
											LL=38		
											PL=22		
											PI=16		
											WC=41.2		
45	9D	24/24	45.0 - 47.0	Push thru vane						Grey, wet, medium stiff, Silty CLAY, (Marine Deposit).	WC=42.7		
	V5	5/5	45.6 - 46.0	S _u =687/27 psf							65x130 mm vane raw torque readings:		
	V6	5/5	46.6 - 47.0	S _u =928/55 psf							V5: 25/1 ft-lbs.		
											V6: 26.5/2 ft-lbs.		
50													
Remarks:													
1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT standard based percentages passing specific grain sizes.													
2. Automatic hammer NEBC D-24 Energy Transfer Ratio = 0.937													
3. Advance SSA to 5.0'; switch to Drive & Wash, insert 4" casing.													
4. Advance 4" casing to 58.5'; telescope 3" casing and spin to 60.0'.													
5. Water level measurements were taken immediately after removal of the casing.													
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3			
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GLR-102			

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Little River Bridge #3112 Route 237 Over Little River Location: Gorham, Maine		Boring No.: BB-GLR-201 PIN: 23909.00																																																																																																																																																																																																																																																																																																				
Driller: New England Boring Contractors			Elevation (ft.) 91.5 (Est.)		Auger ID/OD: 4.25" OD																																																																																																																																																																																																																																																																																																					
Operator: Brad Enos			Datum: NAVD88		Sampler: Standard																																																																																																																																																																																																																																																																																																					
Logged By: B. Cardali			Rig Type: ATV Mobile B-53		Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																																																																					
Date Start/Finish: 8/29/19-8/29/19			Drilling Method: Drive & Wash		Core Barrel: NX2																																																																																																																																																																																																																																																																																																					
Boring Location: N320786, E974001			Casing ID/OD: 4"/4.5", 3"/3.5"		Water Level*: 20.4'																																																																																																																																																																																																																																																																																																					
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Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt			R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person		S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected		S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																																																																																																																																			
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Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>90.4</td><td></td><td>-ASPHALT-</td><td></td></tr><tr><td rowspan="5">5</td><td>1D</td><td>24/13</td><td>1.0 - 3.0</td><td>22-16-12-13</td><td>28</td><td>42</td><td></td><td></td><td></td><td>Brown and grey, dry, dense, fine to coarse SAND, some gravel, trace silt, (Fill).</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></tr><tr><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>2D</td><td>24/10</td><td>4.0 - 6.0</td><td>5-5-4-4</td><td>9</td><td>13</td><td></td><td></td><td></td><td>Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).</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></tr><tr><td rowspan="5">10</td><td>3D</td><td>24/0</td><td>6.0 - 8.0</td><td>5-5-4-4</td><td>9</td><td>13</td><td></td><td></td><td></td><td>No recovery.</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></tr><tr><td>4D</td><td>24/21</td><td>8.0 - 10.0</td><td>3-4-3-5</td><td>7</td><td>10</td><td></td><td></td><td></td><td>Top 13": Brown, moist, loose, fine to coarse SAND, some silt, (Fill).</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></tr><tr><td>5D</td><td>24/24</td><td>10.0 - 12.0</td><td>5-6-7-7</td><td>13</td><td>19</td><td>PUSH</td><td></td><td>82.4</td><td>Bottom 8": Brown and grey, moist, Sandy Silty CLAY, (Fill). Grey-brown, very stiff, Silty CLAY, trace fine sand, trace roots, (Fill).</td><td></td></tr><tr><td rowspan="5">15</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>MV 6D</td><td>24/24</td><td>12.0 - 12.1 12.0 - 14.0</td><td>WOH-2-2-3</td><td>4</td><td>6</td><td></td><td></td><td></td><td>Top 14": Grey-brown, medium stiff, Silty CLAY, trace fine sand, trace roots, (Fill). Bottom 10": Grey, wet, medium stiff, Silty CLAY, (Fill).</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></tr><tr><td>7D MV</td><td>24/24</td><td>14.0 - 16.0 14.0 - 14.1</td><td>WOH-WOH-WOH-WOH</td><td>--</td><td></td><td></td><td></td><td></td><td>Grey, wet, very soft, Sandy Silty CLAY, (Fill).</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></tr><tr><td rowspan="5">20</td><td>8D</td><td>24/0</td><td>16.0 - 18.0</td><td>1-2-3-4</td><td>5</td><td>7</td><td></td><td></td><td></td><td>No recovery in 2" splitspoon.</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></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Grey, wet, medium stiff, Silty CLAY, some fine to medium sand, (Fill).</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></tr><tr><td>MV 9D</td><td>24/24</td><td>20.0 - 20.1 20.0 - 22.0</td><td>WOH-1-2-3</td><td>3</td><td>4</td><td>RC</td><td></td><td>71.5</td><td>Grey-brown, wet, soft, Silty CLAY, trace fine Sand, (Marine Deposit).</td><td></td></tr><tr><td rowspan="3">25</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></tr><tr><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	0							SSA	90.4		-ASPHALT-		5	1D	24/13	1.0 - 3.0	22-16-12-13	28	42				Brown and grey, dry, dense, fine to coarse SAND, some gravel, trace silt, (Fill).																								2D	24/10	4.0 - 6.0	5-5-4-4	9	13				Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).													10	3D	24/0	6.0 - 8.0	5-5-4-4	9	13				No recovery.													4D	24/21	8.0 - 10.0	3-4-3-5	7	10				Top 13": Brown, moist, loose, fine to coarse SAND, some silt, (Fill).													5D	24/24	10.0 - 12.0	5-6-7-7	13	19	PUSH		82.4	Bottom 8": Brown and grey, moist, Sandy Silty CLAY, (Fill). 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Maine Department of Transportation						Project: Little River Bridge #3112 Route 237 Over Little River				Boring No.: BB-GLR-201					
Soil/Rock Exploration Log US CUSTOMARY UNITS						Location: Gorham, Maine				PIN: 23909.00					
Driller: New England Boring Contractors						Elevation (ft.): 91.5 (Est.)				Auger ID/OD: 4.25" OD					
Operator: Brad Enos						Datum: NAVD88				Sampler: Standard					
Logged By: B. Cardali						Rig Type: ATV Mobile B-53				Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 8/29/19-8/29/19						Drilling Method: Drive & Wash				Core Barrel: NX2					
Boring Location: N320786, E974001						Casing ID/OD: 4"/4.5", 3"/3.5"				Water Level*: 20.4'					
Hammer Efficiency Factor: 0.895						Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt						R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) N-uncorrected = Raw field SPT N-value q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected					
S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test															
Sample Information															
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows /6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.				
25	10D V1	24/24	25.0 - 27.0 25.6 - 26.0	Push thru vane S _u =1030/253 psf	--					Grey, wet, stiff, Silty CLAY, trace sand, (Marine Deposit). 65x130 mm vane raw torque readings: V1: 450/110 in-lbs Could not advance vane beyond 26.1'.					
	MV		26.0 - 26.1												
30	11D V2	24/24	30.0 - 32.0 30.6 - 31.0	Push thru vane S _u =893/137 psf	--					Grey, wet, medium stiff, silty CLAY, (Marine Deposit). 65x130 mm vane raw torque readings: V2: 390/60 in-lbs V3: 365/55 in-lbs					
	V3		31.6 - 32.0	S _u =835/126 psf											
35															
	12D	24/9	37.5 - 39.5	7-5-8-11	13	19		54.0		Grey, wet, medium dense, fine to coarse SAND, some gravel, little silt, (Sand).					
40															
45	13D R1	1/1 60/56	45.0 - 45.1 45.1 - 50.1	75/1" RQD=93%	--		NX2	46.4		Grey, wet, Gravelly fine to coarse SAND, trace silt, (Sand). Splitspoon refusal at 45.1' bgs. Advanced roller cone to 45.1' and set up to core. R1: Very hard, fresh, fine to coarse grained, grey and white banded, SCHIST. Joints are moderately spaced, moderately dipping to high angle, planar and stepped, rough, fresh, tight to partially open. -VASSALBORO FORMATION- Rock Mass Quality = Excellent					
50															
Remarks:															
1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MaineDOT Standard based percentages passing specific grain sizes. 2. Automatic hammer NEBC D-19 Energy Transfer Ratio = 0.895 3. Description of 8D is from recovery of 3" OD Spoon taken after no recovery in standard splitspoon. 4. Advance 4" casing to 20' bgs, then 3" casing to 45.1'. 5. Water level was measured after the drill casing was removed.															
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 3 Boring No.: BB-GLR-201					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS						Project: Little River Bridge #3112 Route 237 Over Little River Location: Gorham, Maine				Boring No.: BB-GLR-201 PIN: 23909.00			
Driller: New England Boring Contractors						Elevation (ft.): 91.5 (Est.)				Auger ID/OD: 4.25" OD			
Operator: Brad Enos						Datum: NAVD88				Sampler: Standard			
Logged By: B. Cardali						Rig Type: ATV Mobile B-53				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 8/29/19-8/29/19						Drilling Method: Drive & Wash				Core Barrel: NX2			
Boring Location: N320786, E974001						Casing ID/OD: 4"/4.5", 3"/3.5"				Water Level*: 20.4'			
Hammer Efficiency Factor: 0.895						Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test													
Sample Information													
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows /6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.		
50	R2	60/60	50.1 - 55.1	RQD = 100%						Recovery = 93% Rock Core Times (min:sec): 45.1-46.1' (3:00), 46.1-47.1' (2:10), 47.1-48.1' (2:05), 48.1-49.1' (1:30), 49.1-50.1' (2:50) R2: Very hard, fresh, fine to coarse grained, grey and white banded, SCHIST. No joints in core run. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 50.1-51.1' (1:05), 51.1-52.1' (1:10), 52.1-53.1' (1:40), 53.1-54.1' (2:10), 54.1-55.1' (2:30)			
55								36.4		Bottom of Exploration at 55.10 feet below ground surface.			
60													
65													
70													
75													
Remarks: 1. Fine-Grained Soil Descriptions on this log are based on plasticity estimated using visual-manual classification techniques or laboratory Atterburg Limit tests if available, rather than the MainesDOT Standard based percentages passing specific grain sizes. 2. Automatic hammer NEBC D-19 Energy Transfer Ratio = 0.895 3. Description of 8D is from recovery of 3" OD Spoon taken after no recovery in standard splitspoon. 4. Advance 4" casing to 20' bgs, then 3" casing to 45.1'. 5. Water level was measured after the drill casing was removed.													
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 3 of 3 Boring No.: BB-GLR-201			



10/01/2019

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LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX D – CONE PENETRATION TEST REPORT



10/01/2019

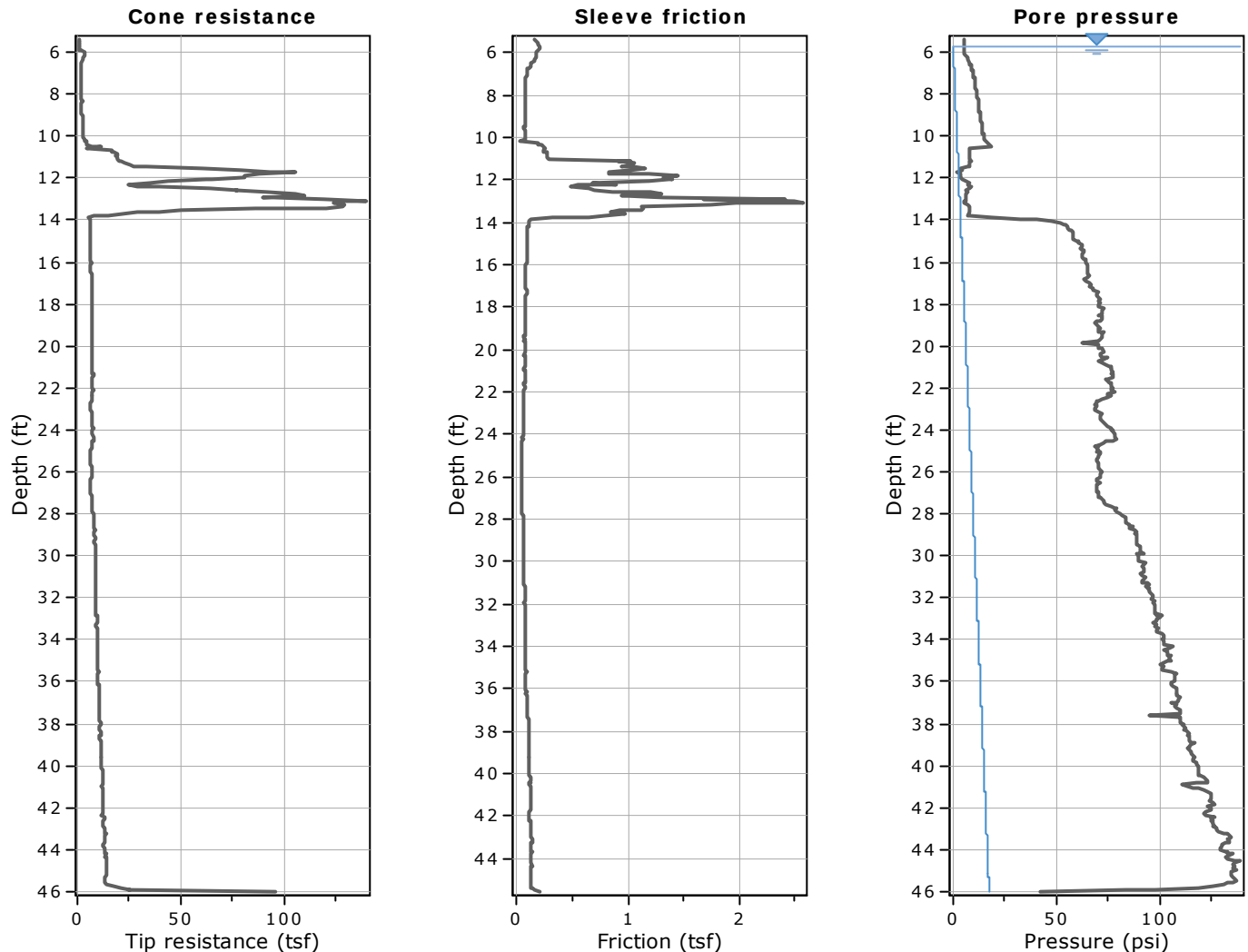
GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX D.1 – CPT INTERPRETATION REPORT BY GZA

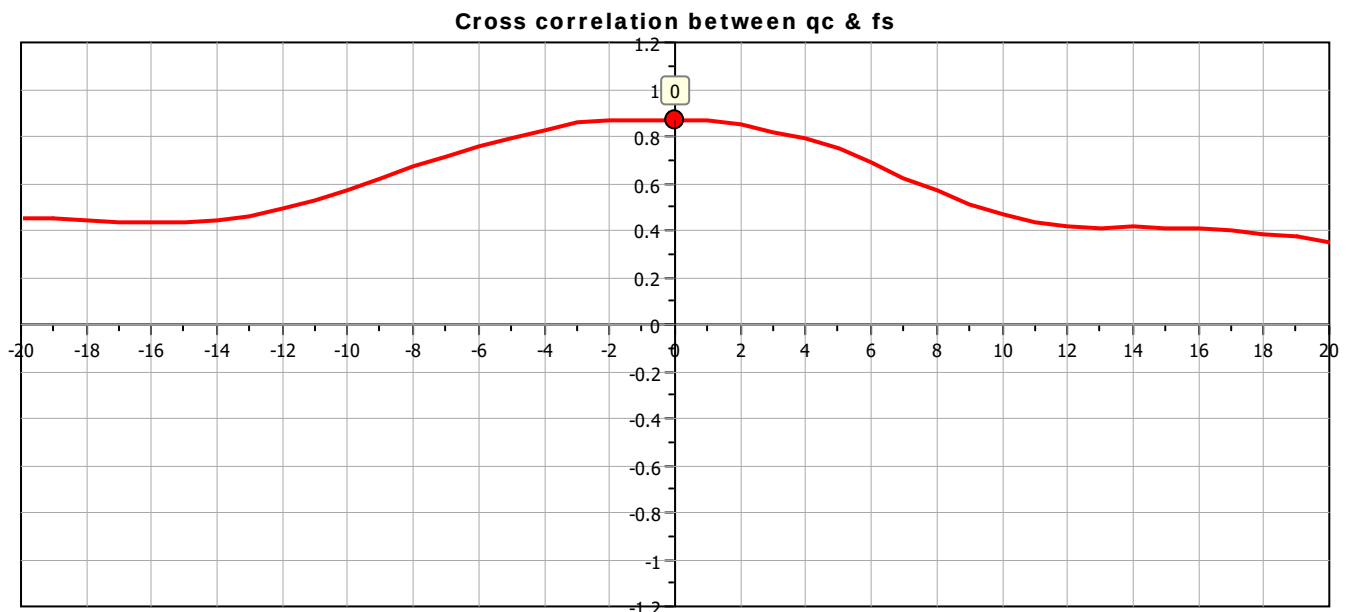


Project: Little River Bridge

Location: Gorham, ME



The plot below presents the cross correlation coefficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two successive CPT measurements).

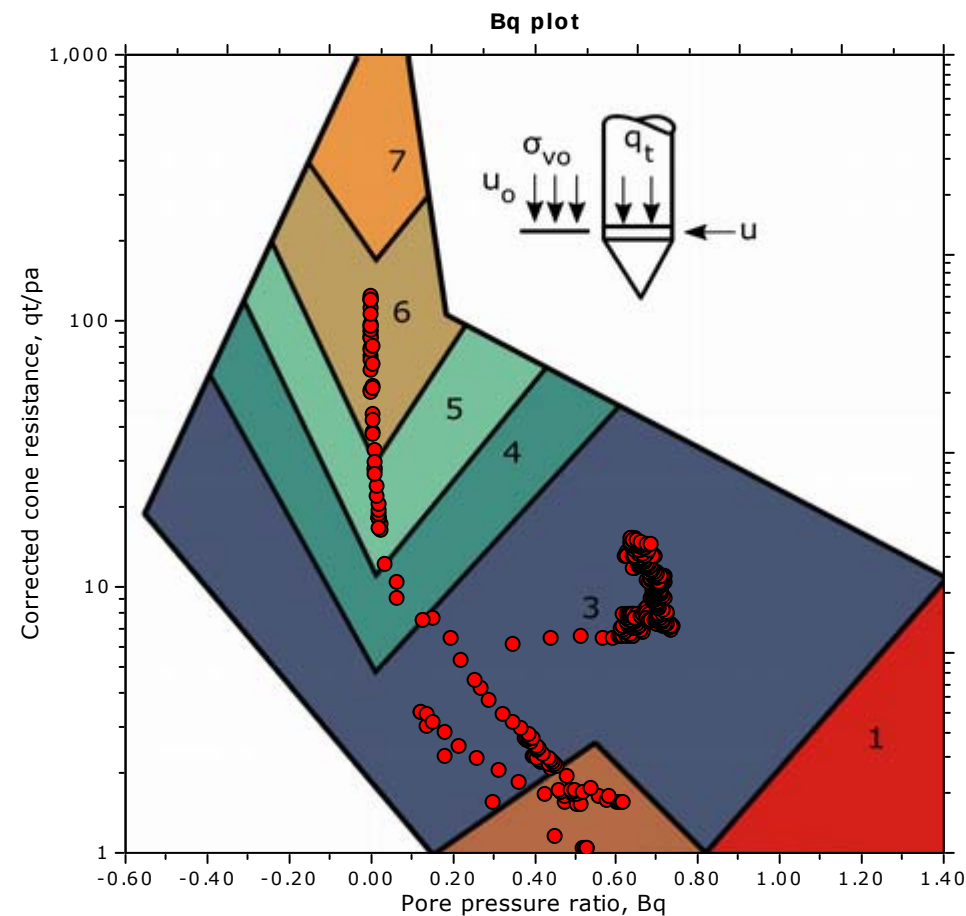
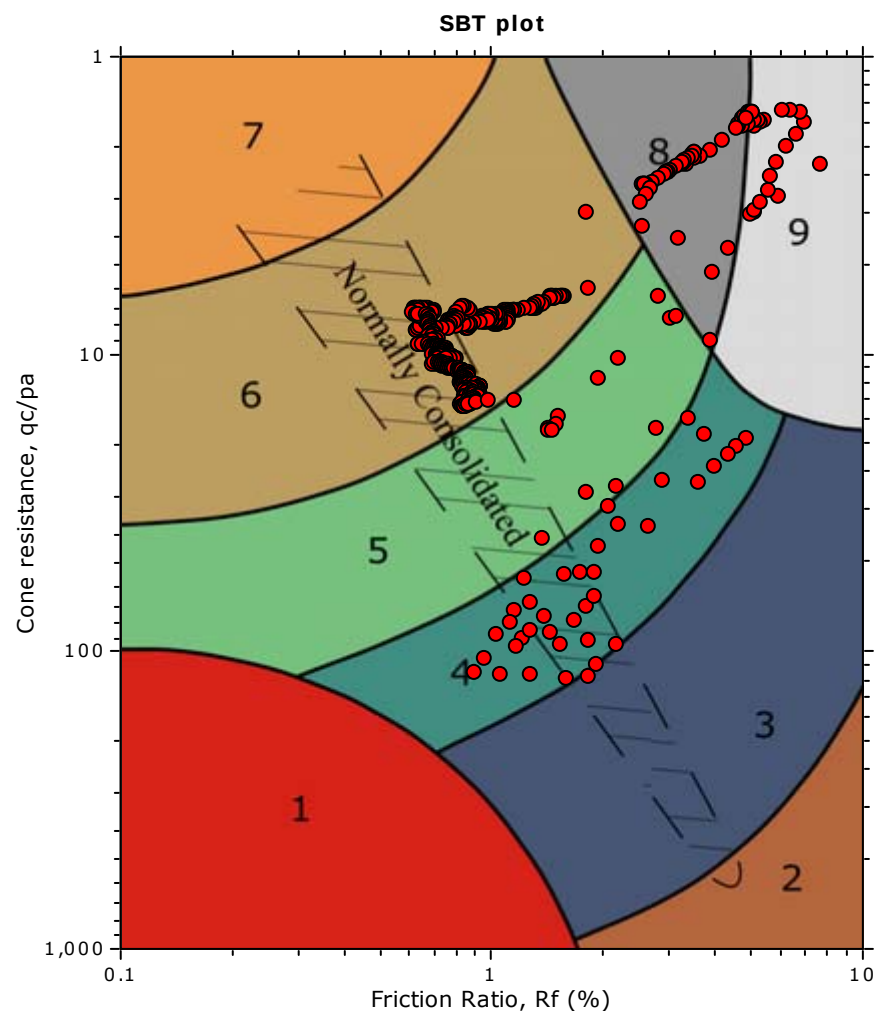




Project: Little River Bridge

Location: Gorham, ME

SBT - Bq plots



SBT legend

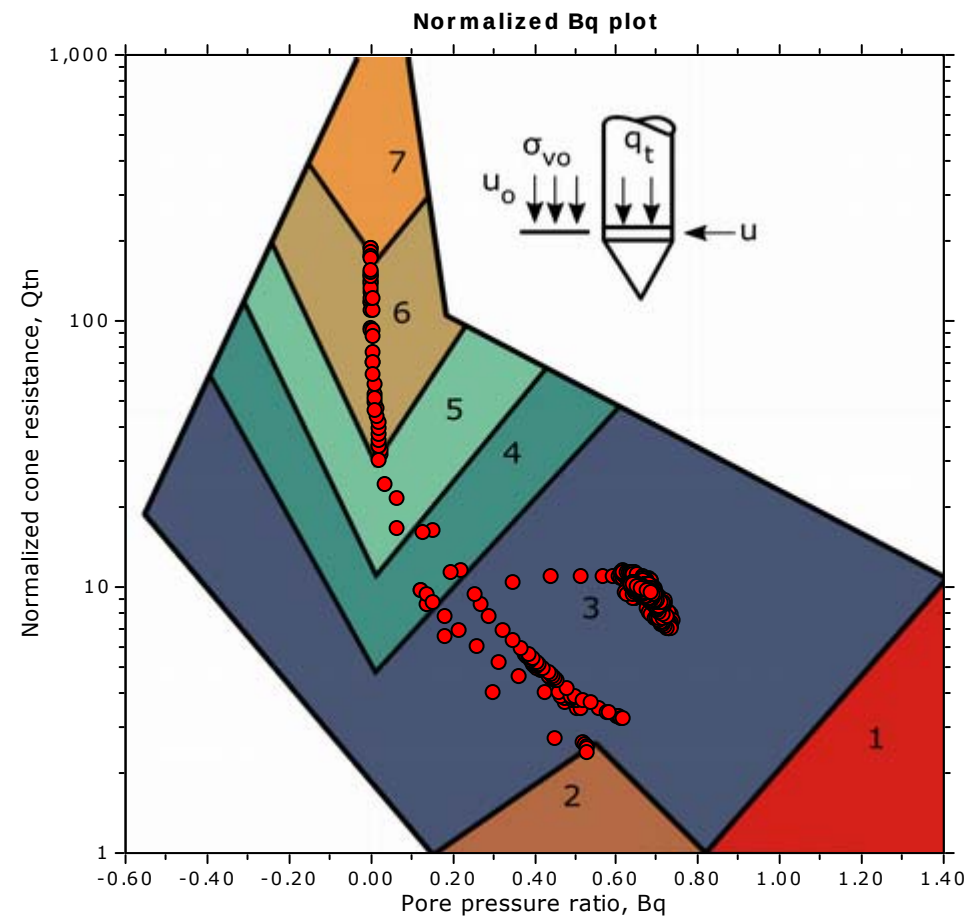
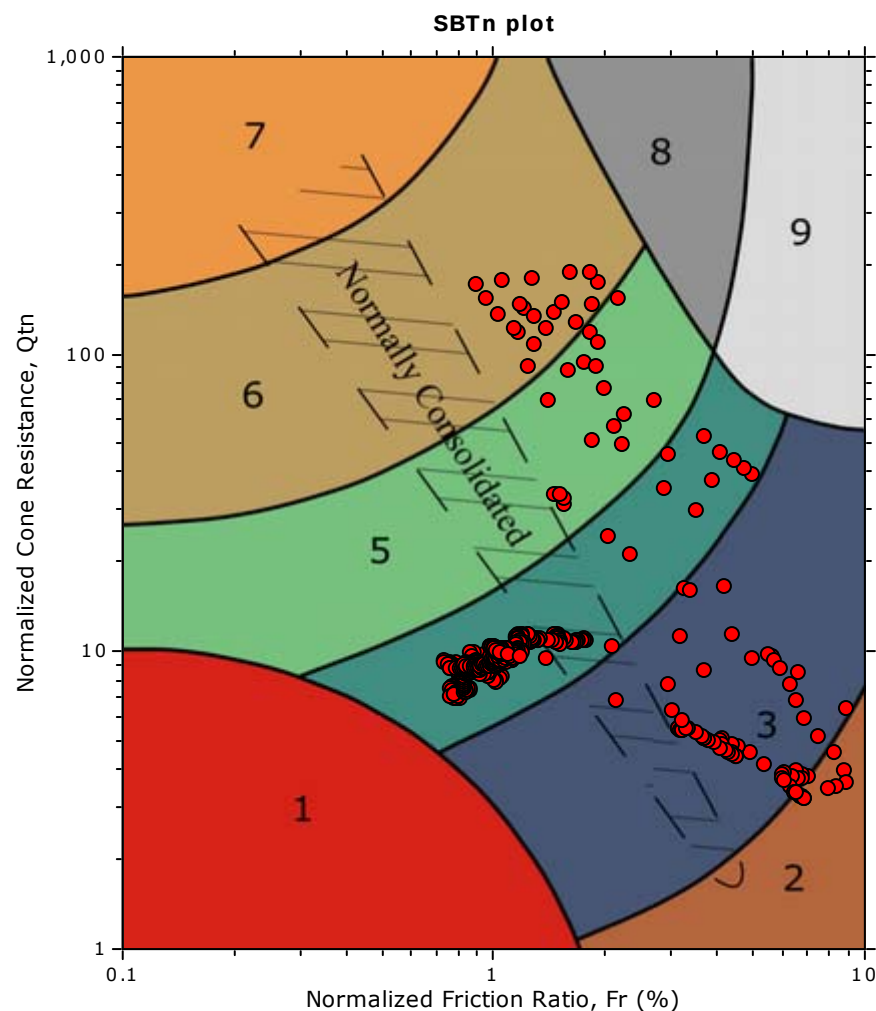
- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravelly sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty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



Project: Little River Bridge

Location: Gorham, ME

SBT - Bq plots (normalized)



SBTn legend

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



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

CPT: CPT-GLR-101

Total depth: 46.01 ft, Date: 8/15/2018

Surface Elevation: 75.90 ft

Coords: X:973977.60, Y:320922.10

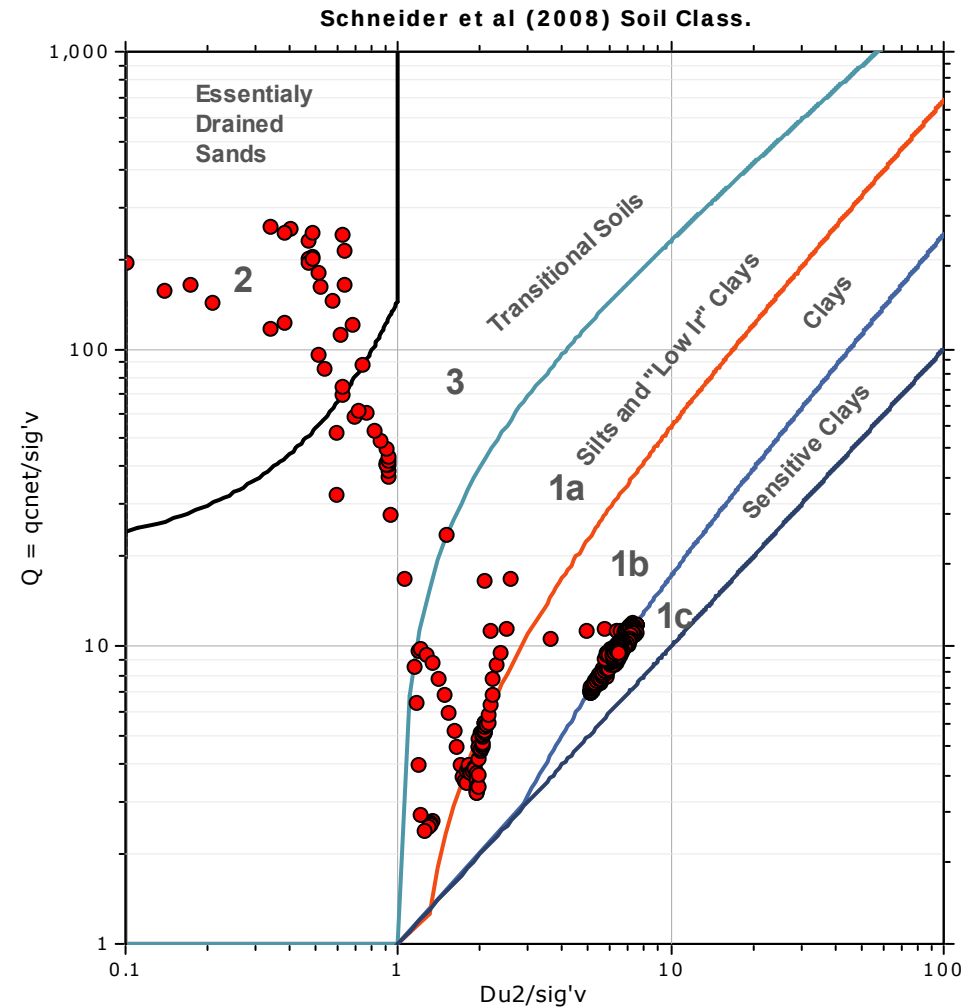
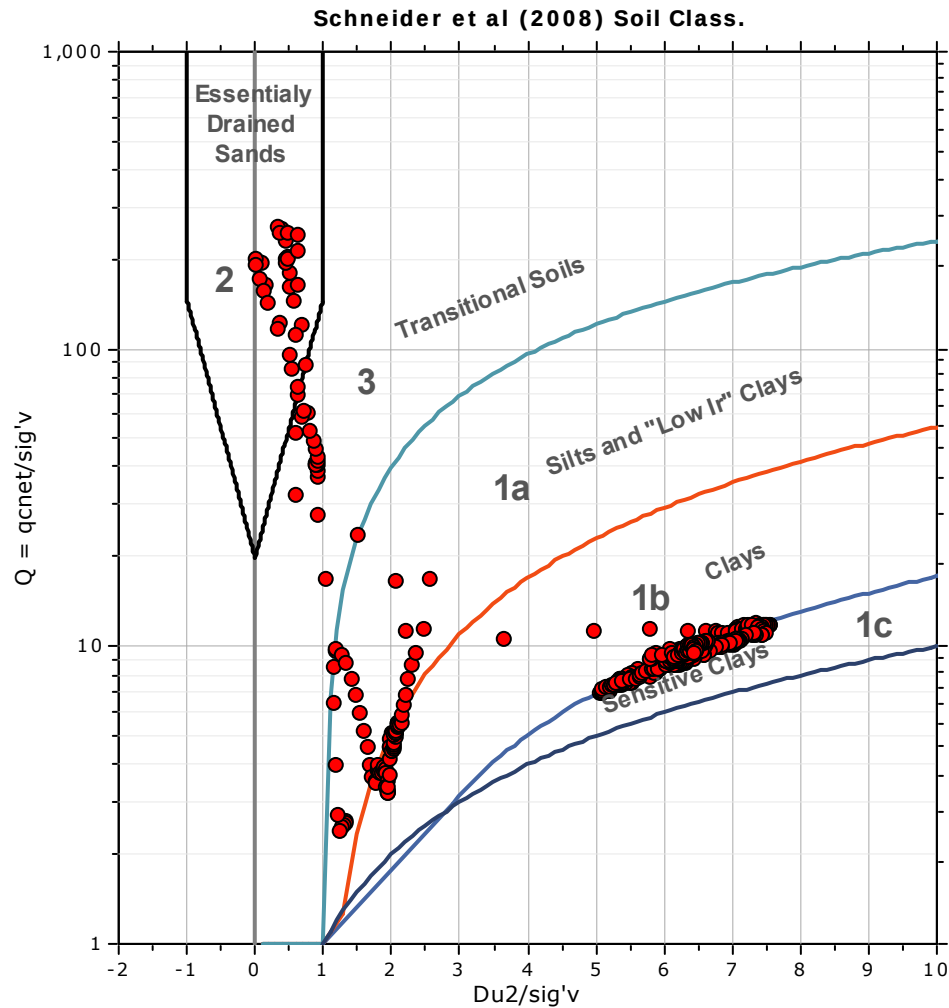
Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME

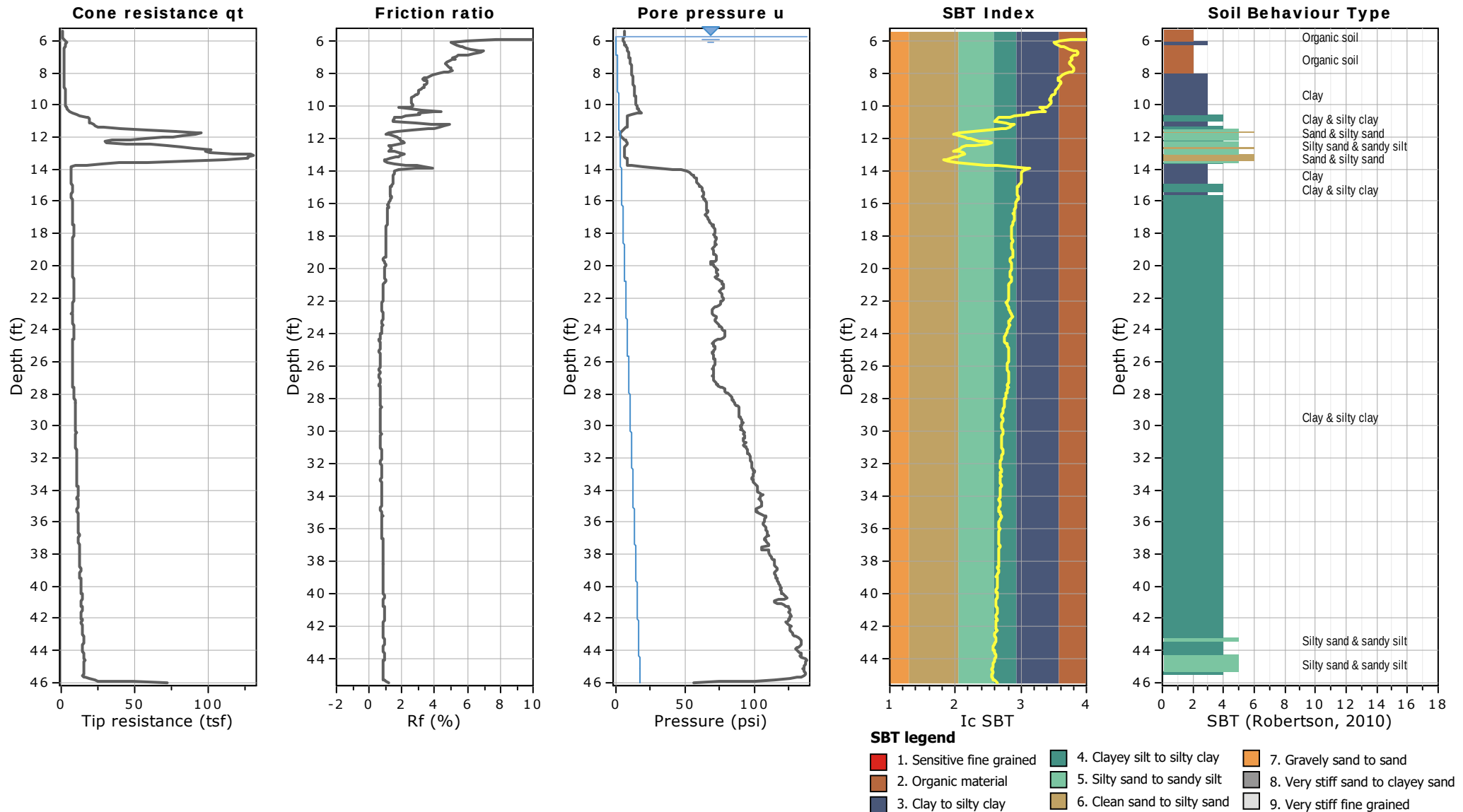
Bq plots (Schneider)





Project: Little River Bridge

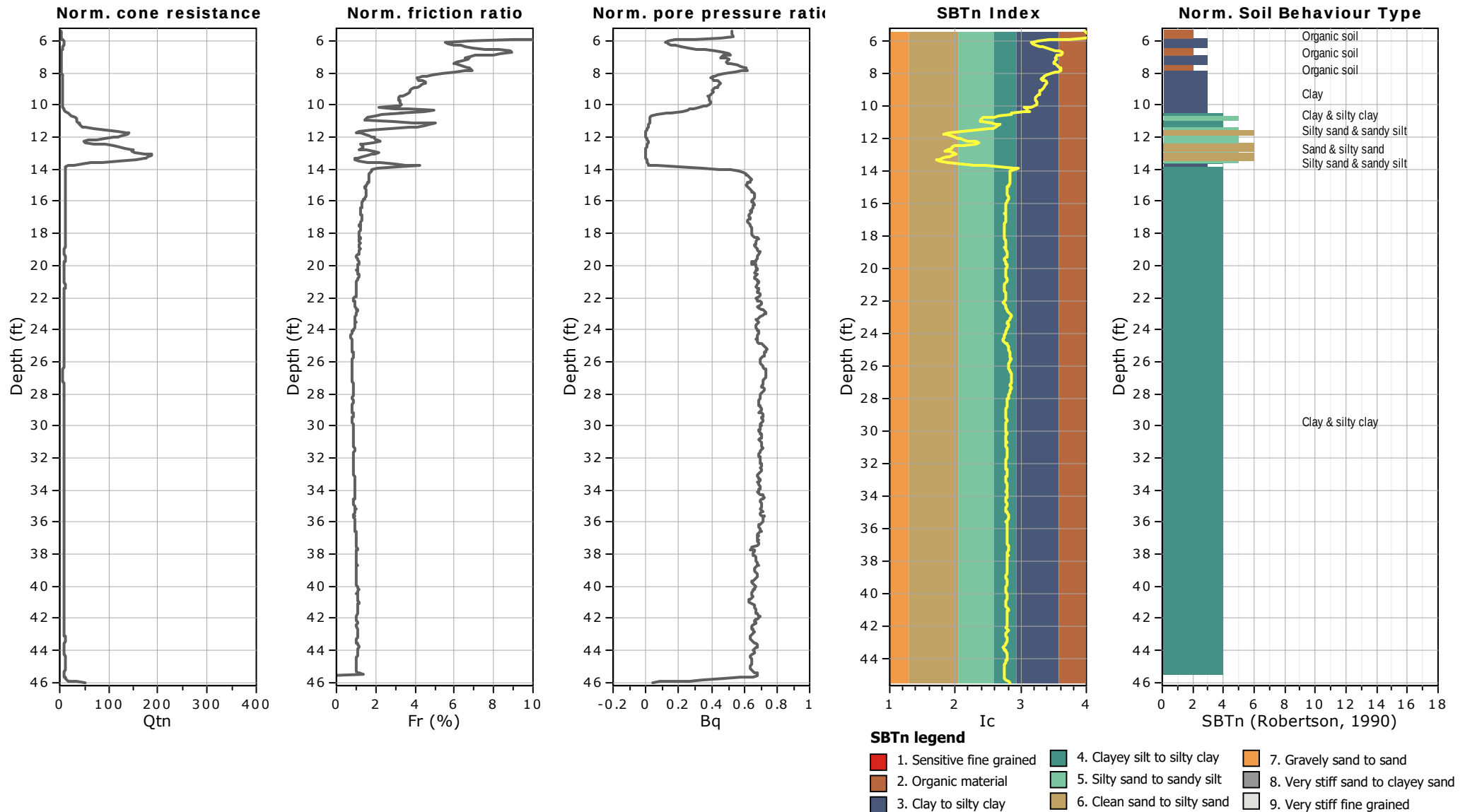
Location: Gorham, ME





Project: Little River Bridge

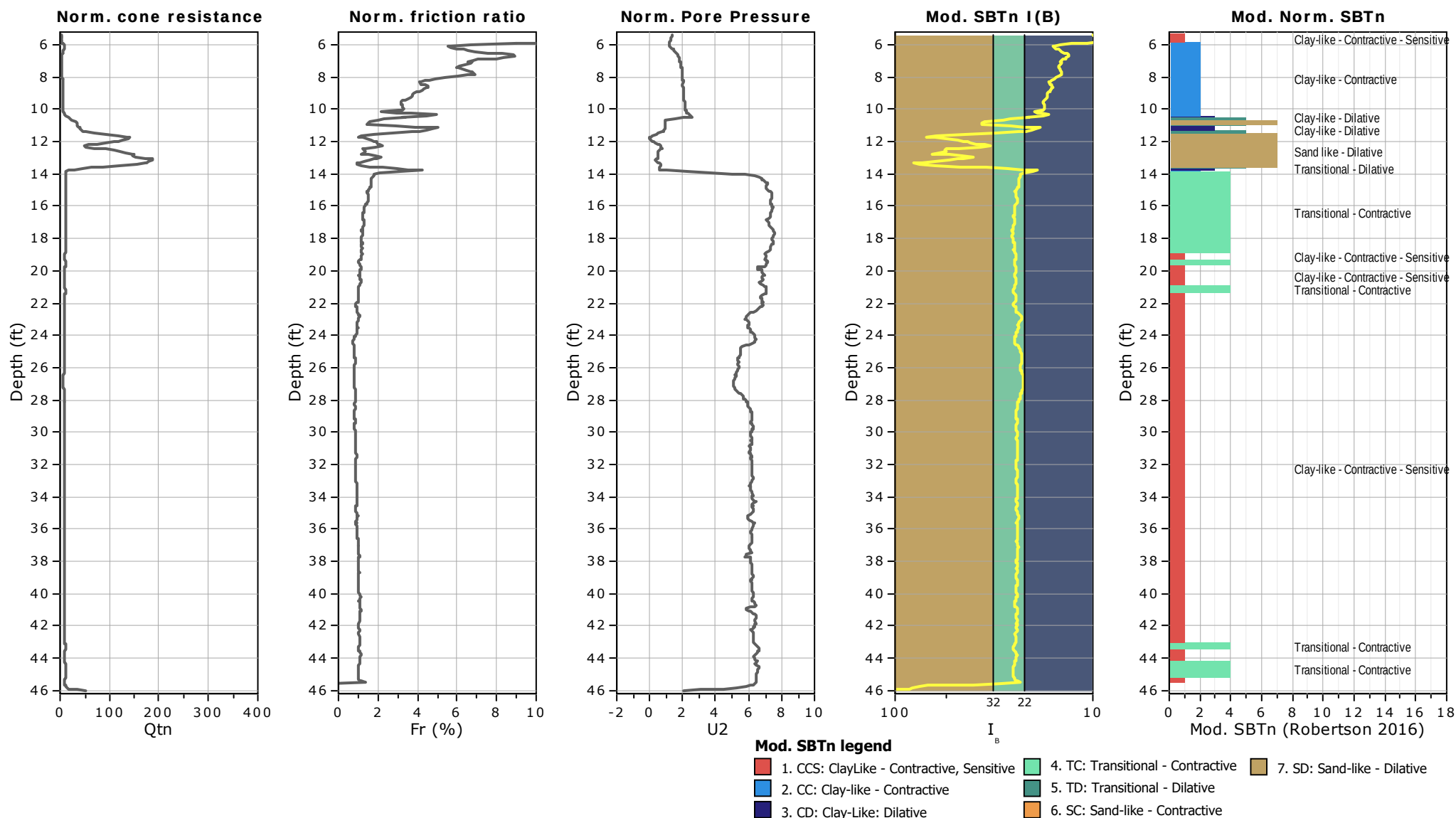
Location: Gorham, ME





Project: Little River Bridge

Location: Gorham, ME

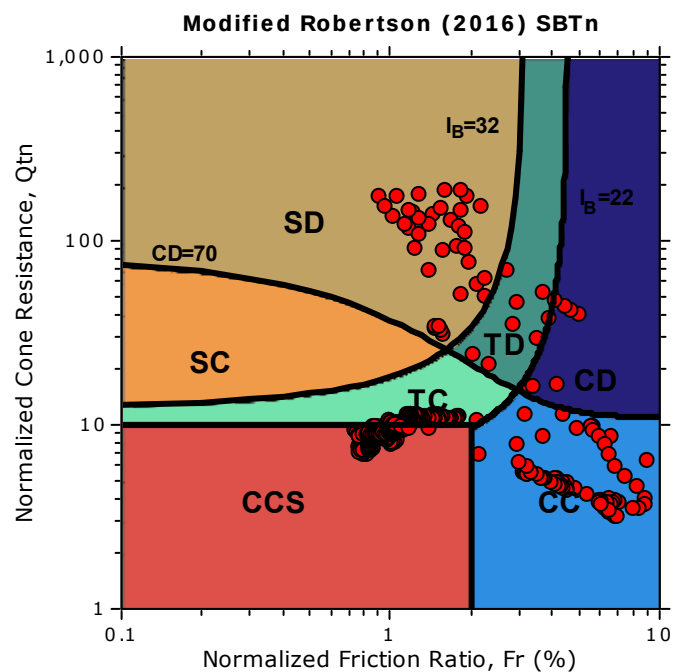




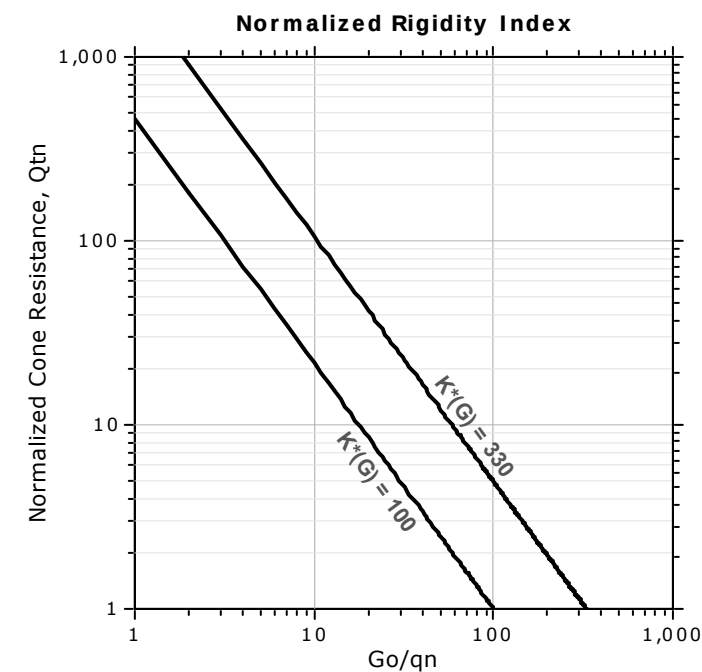
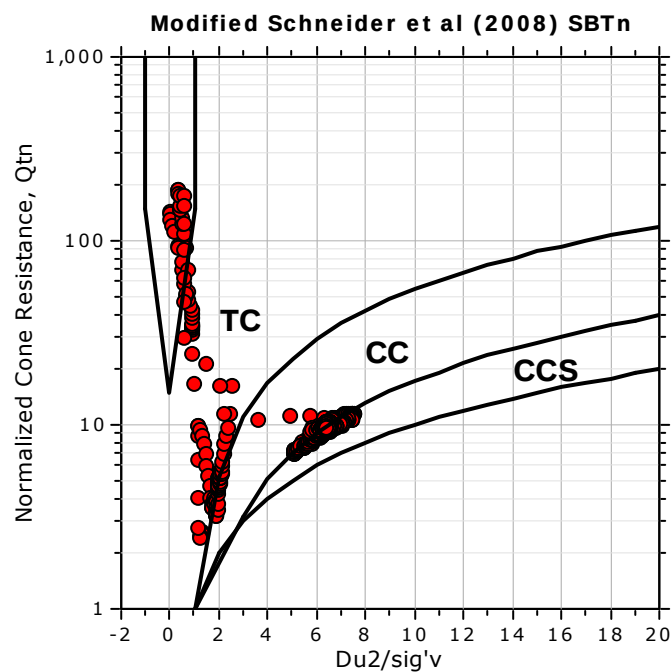
Project: Little River Bridge

Location: Gorham, ME

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative



$K(G) > 330$: Soils with significant microstructure
(e.g. age/cementation)



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

CPT: CPT-GLR-101

Total depth: 46.01 ft, Date: 8/15/2018

Surface Elevation: 75.90 ft

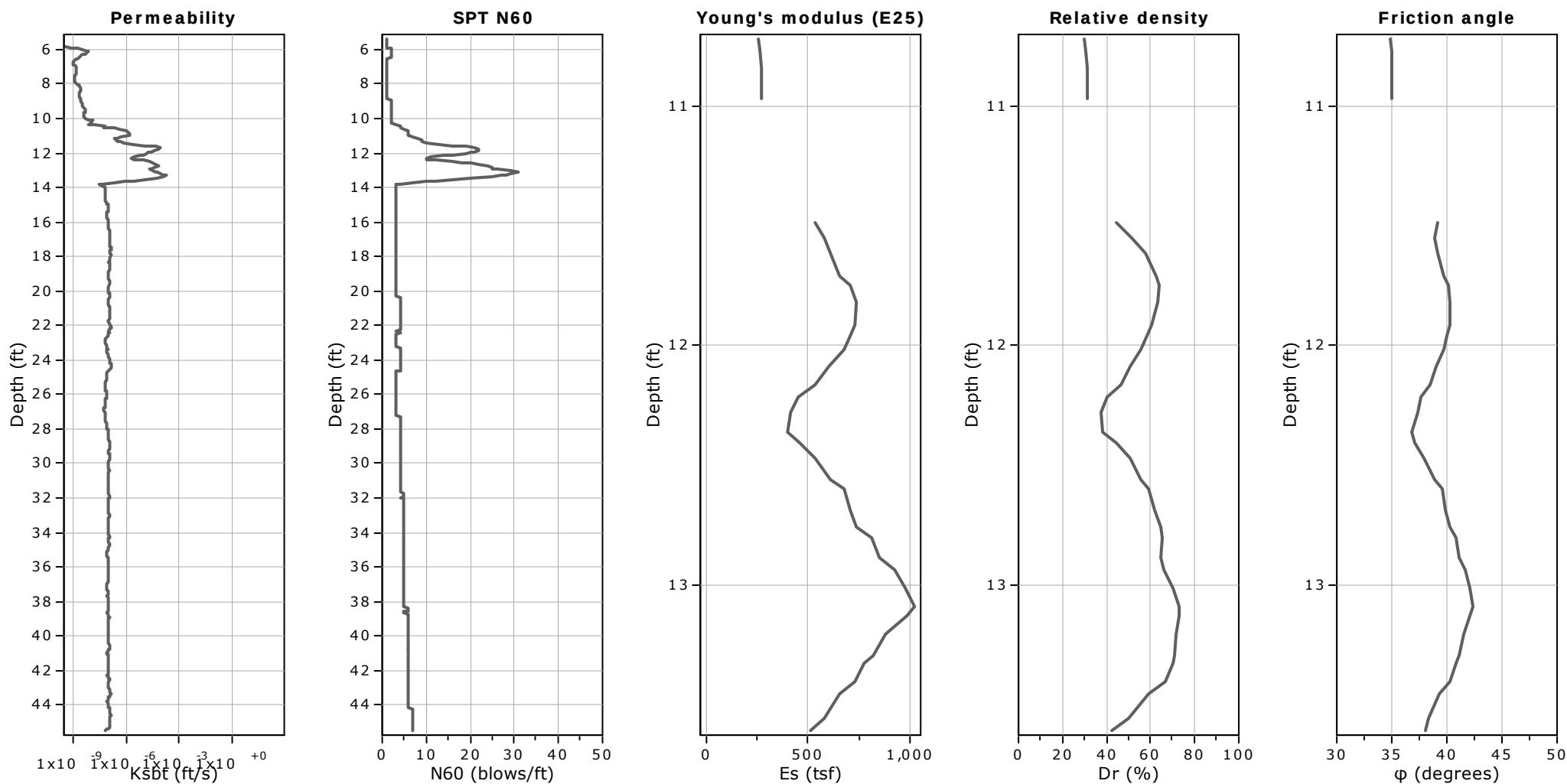
Coords: X:973977.60, Y:320922.10

Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60} : Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr} : 350.0

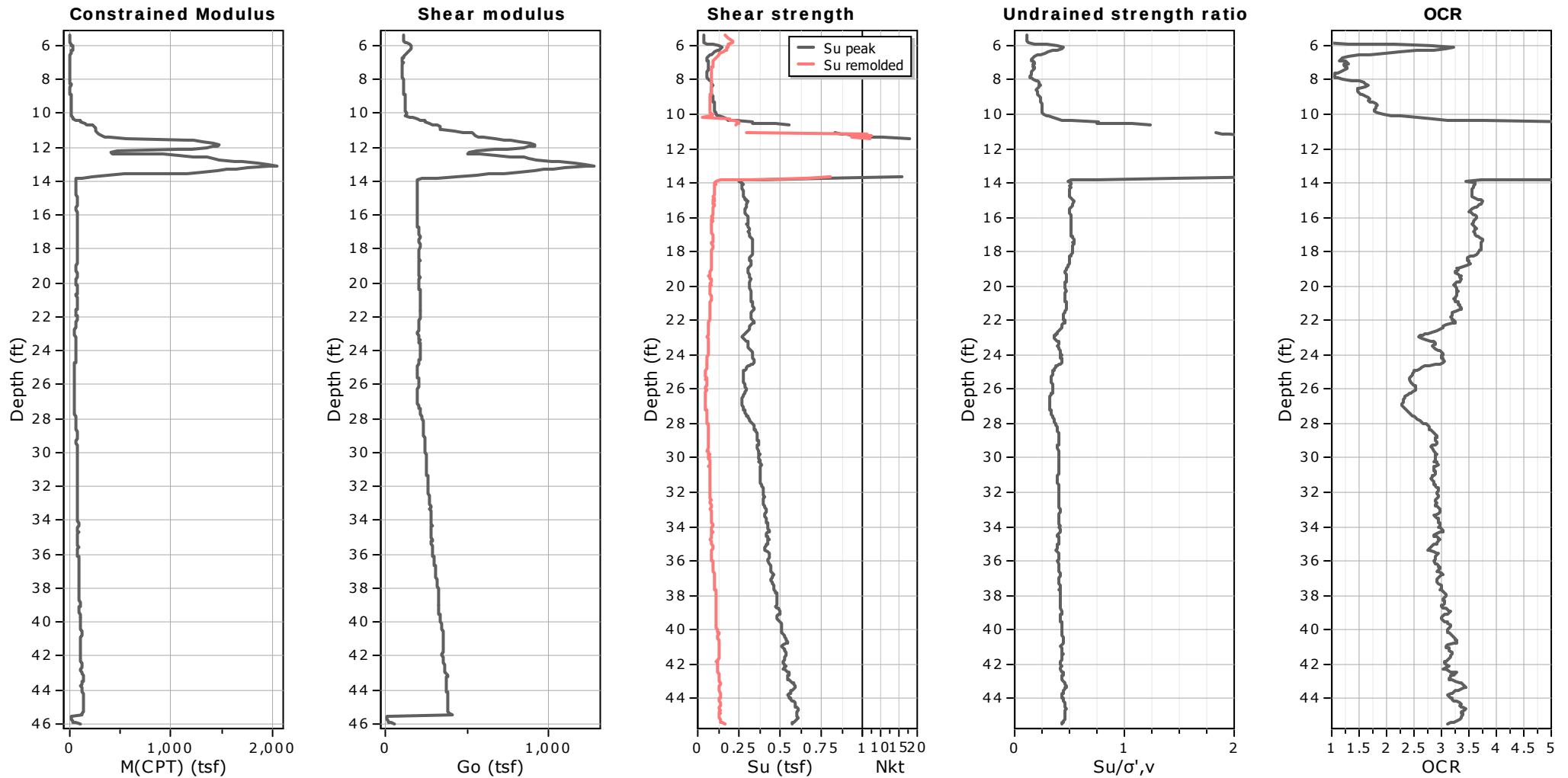
Phi: Based on Kulhawy & Mayne (1990)

—●— User defined estimation data



Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

Constrained modulus: Based on variable α using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable α using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt} : 22

OCR factor for clays, N_{kt} : 0.33

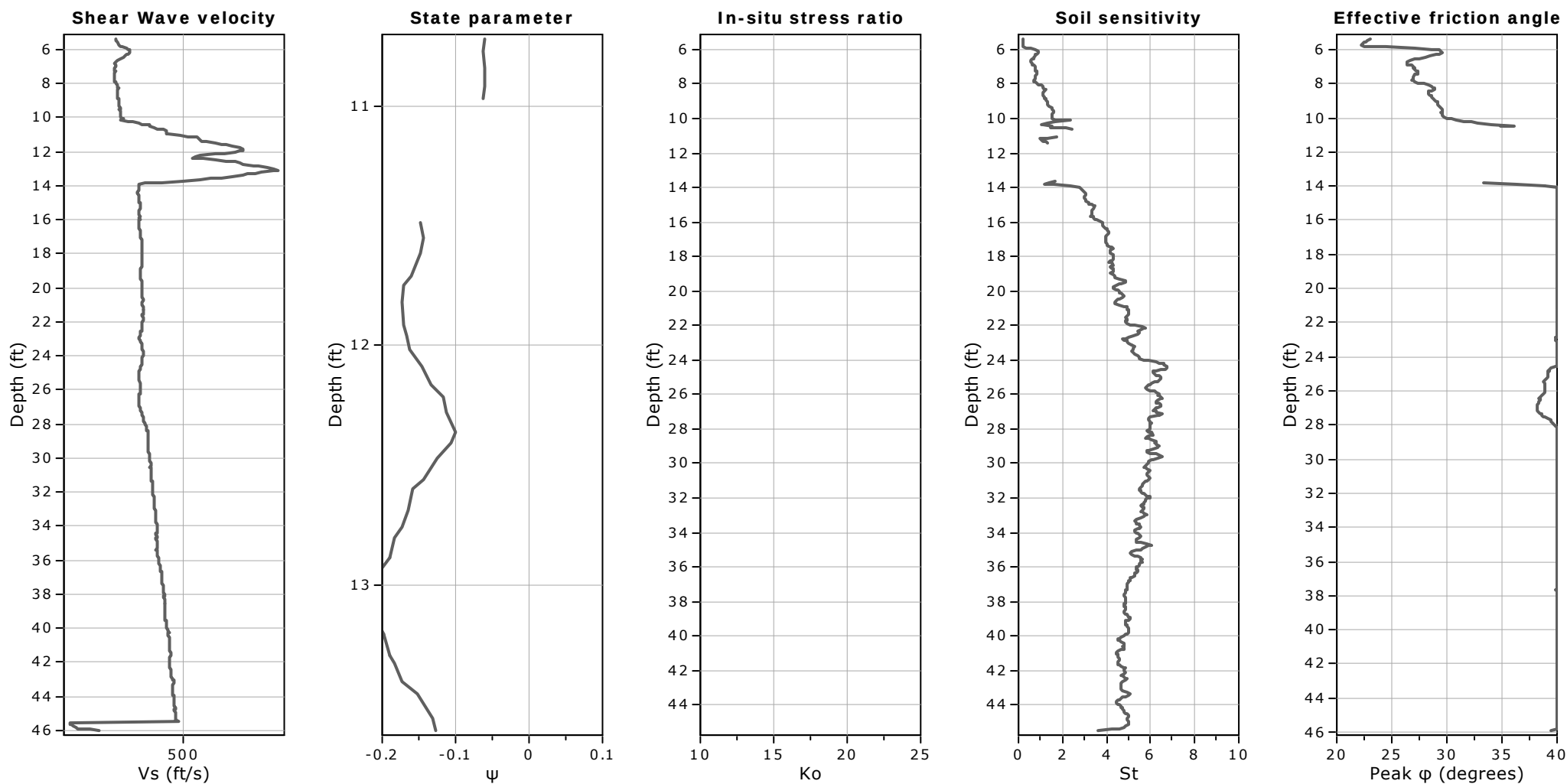
● User defined estimation data

● Flat Dilatometer Test data



Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

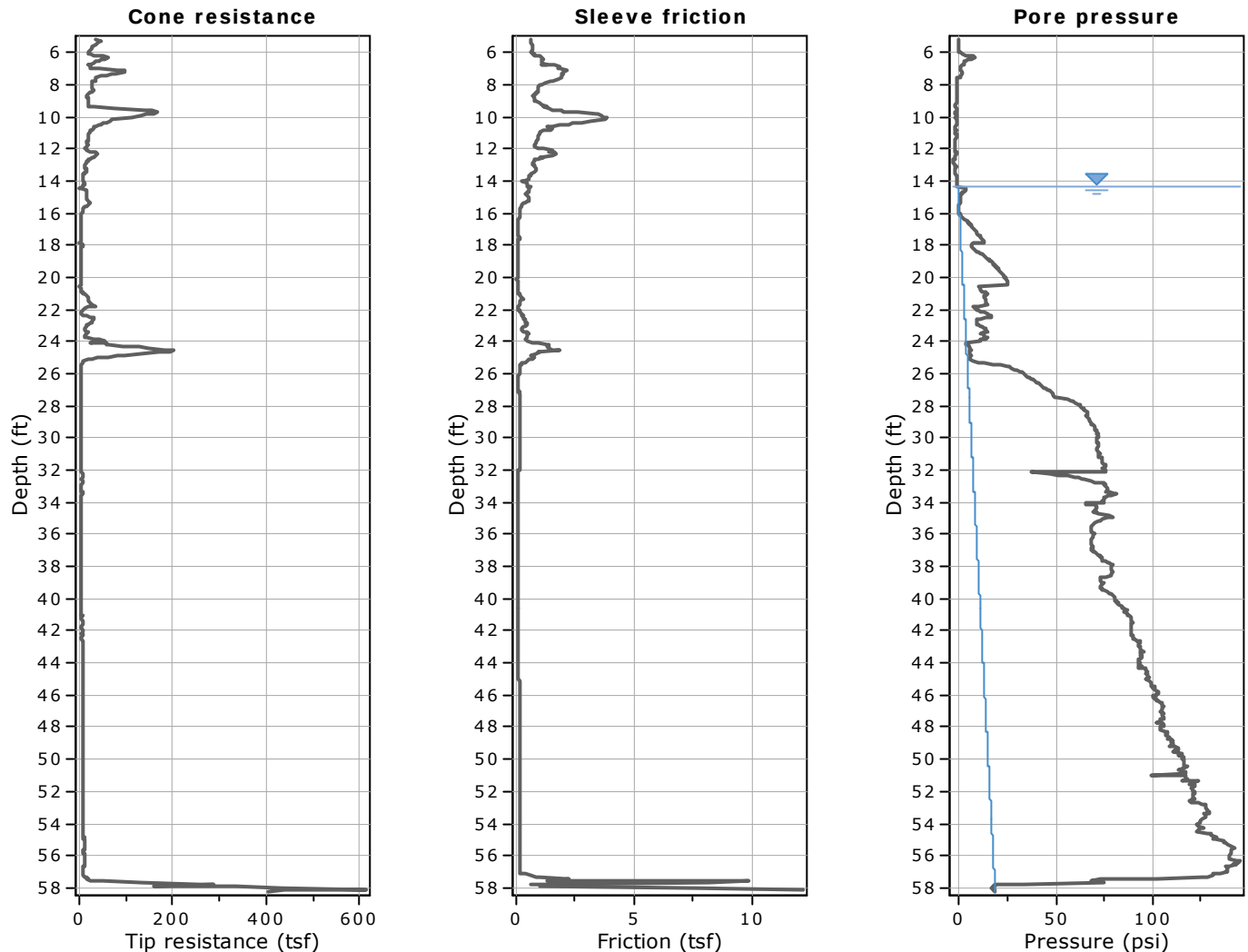
Soil Sensitivity factor, N_s : 5.00

—●— User defined estimation data

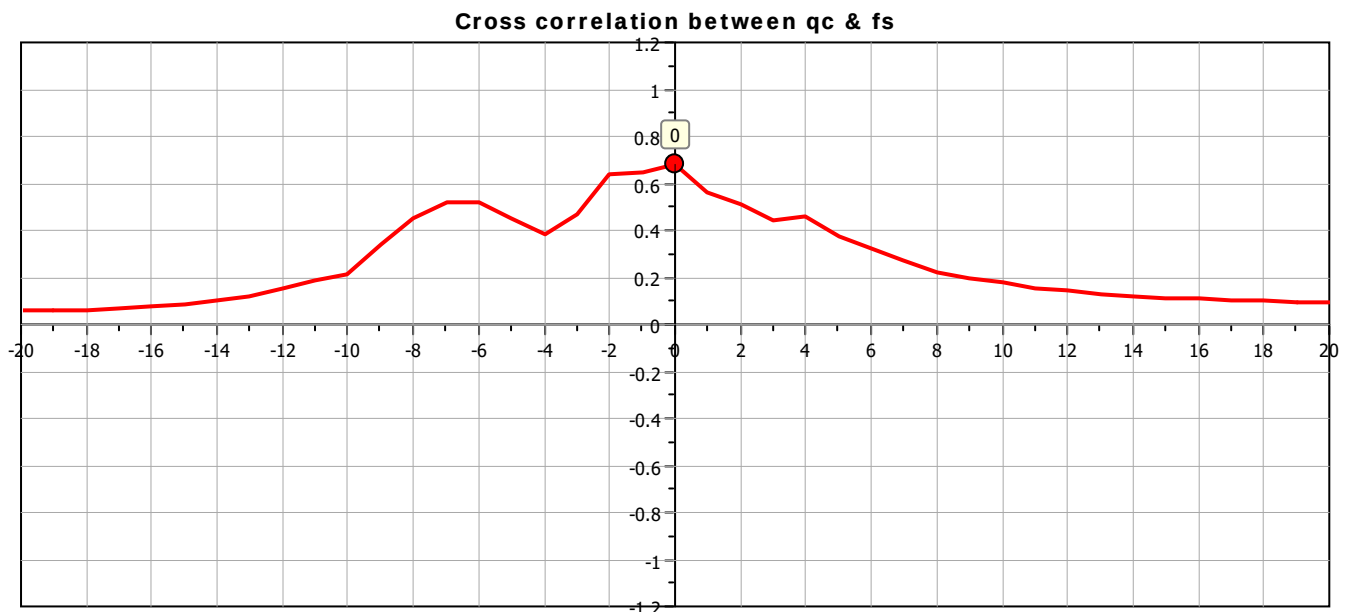


Project: Little River Bridge

Location: Gorham, ME



The plot below presents the cross correlation coefficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two successive CPT measurements).

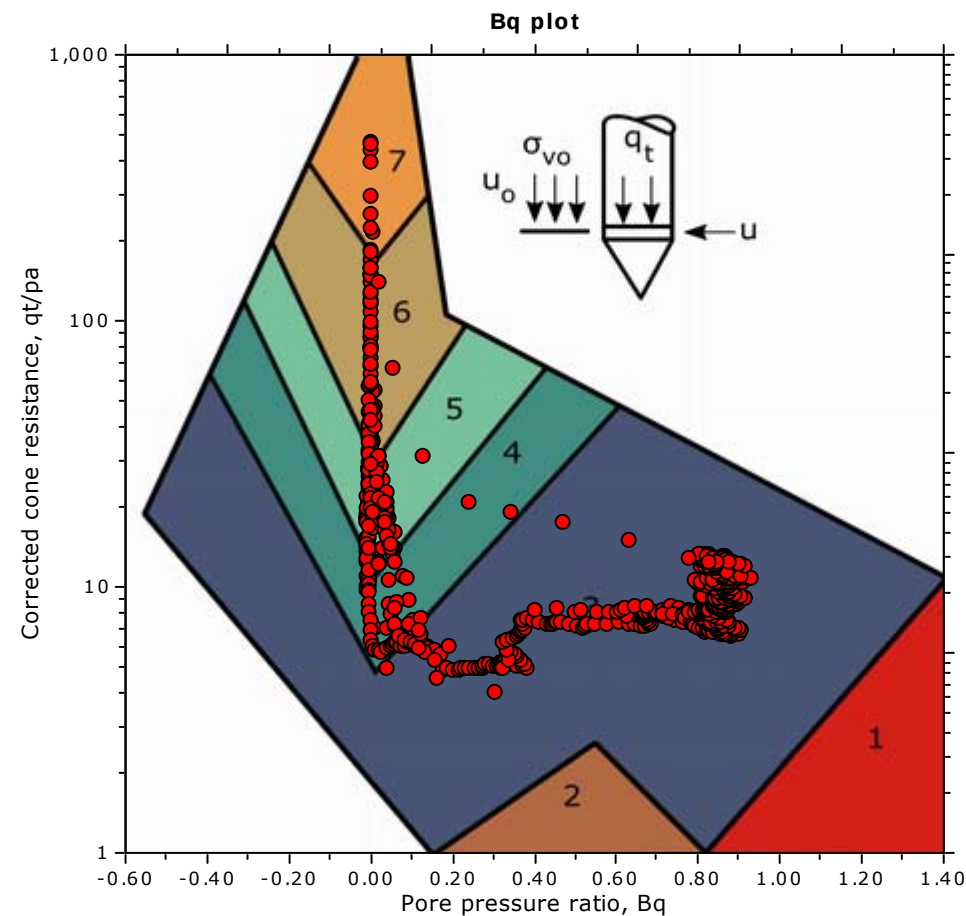
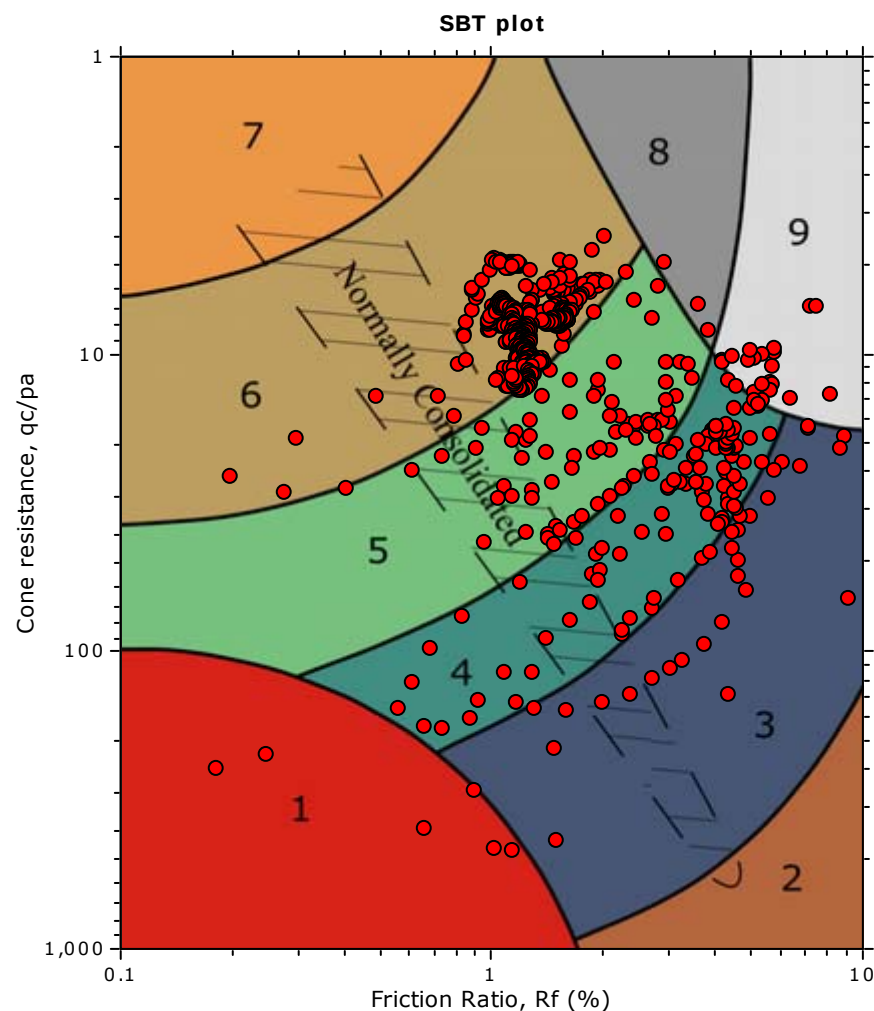




Project: Little River Bridge

Location: Gorham, ME

SBT - Bq plots



SBT legend

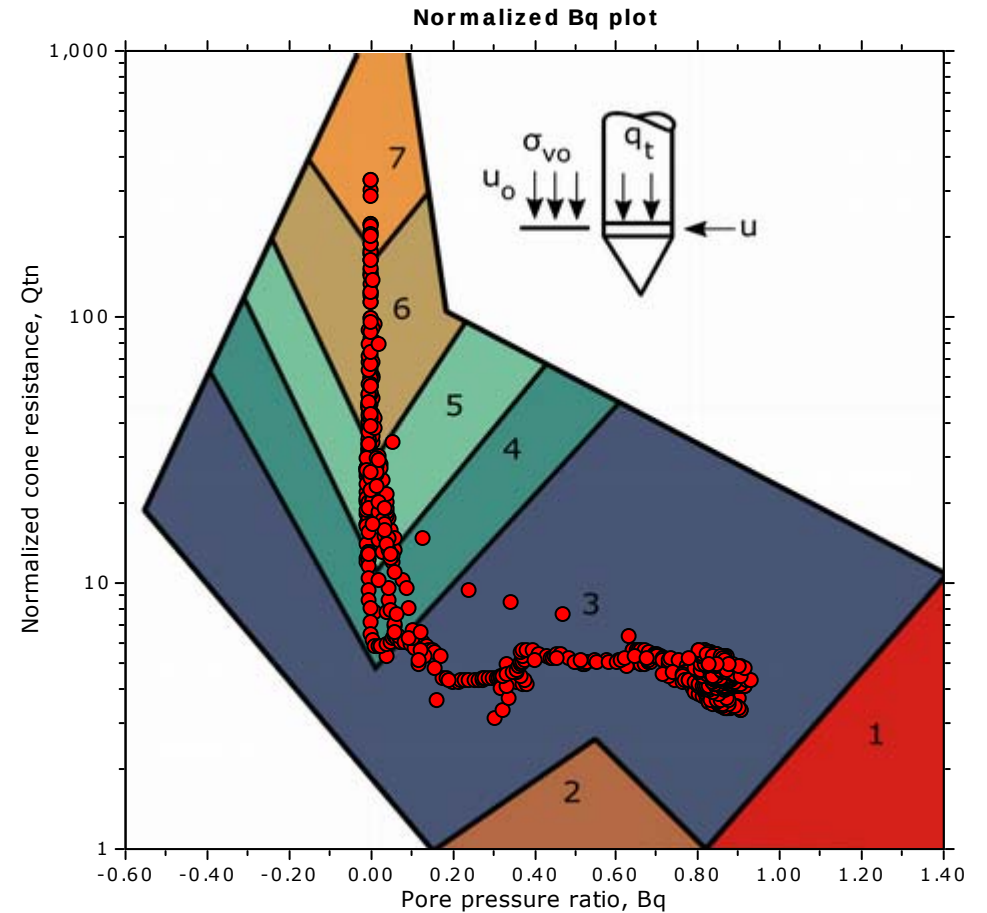
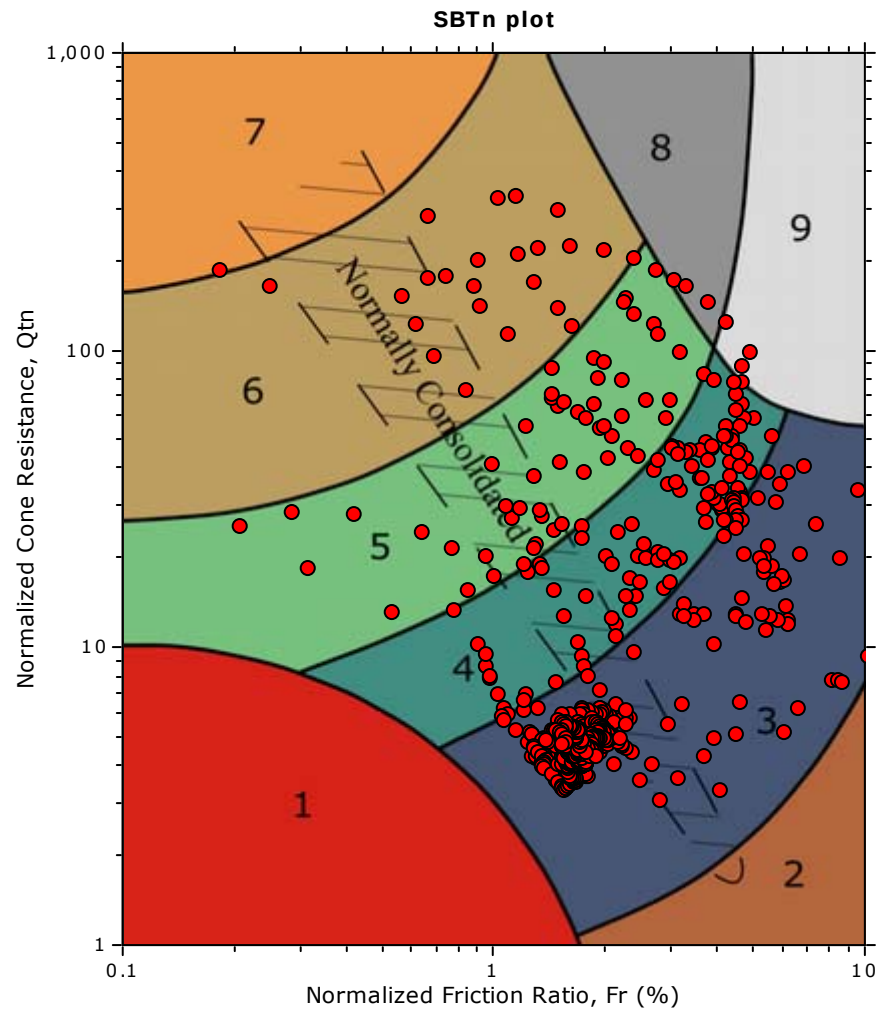
- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravelly sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty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



Project: Little River Bridge

Location: Gorham, ME

SBT - Bq plots (normalized)



SBTn legend

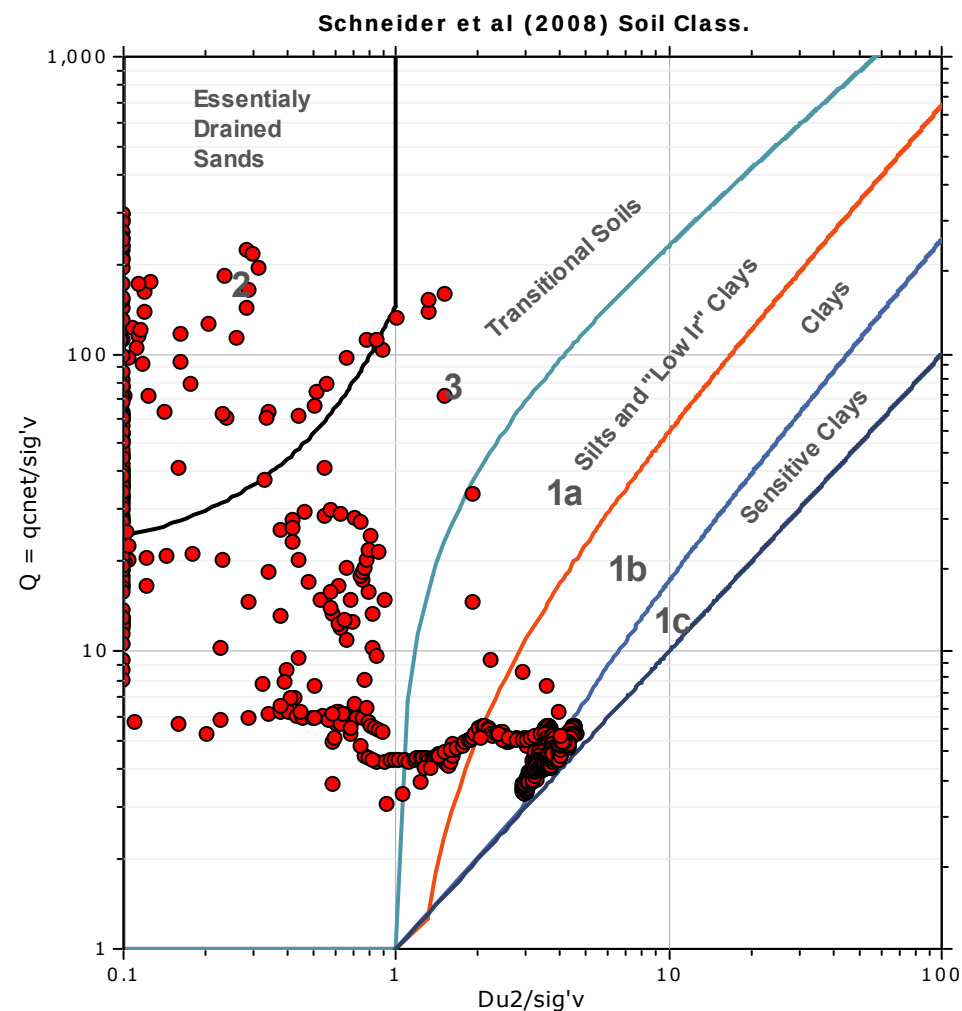
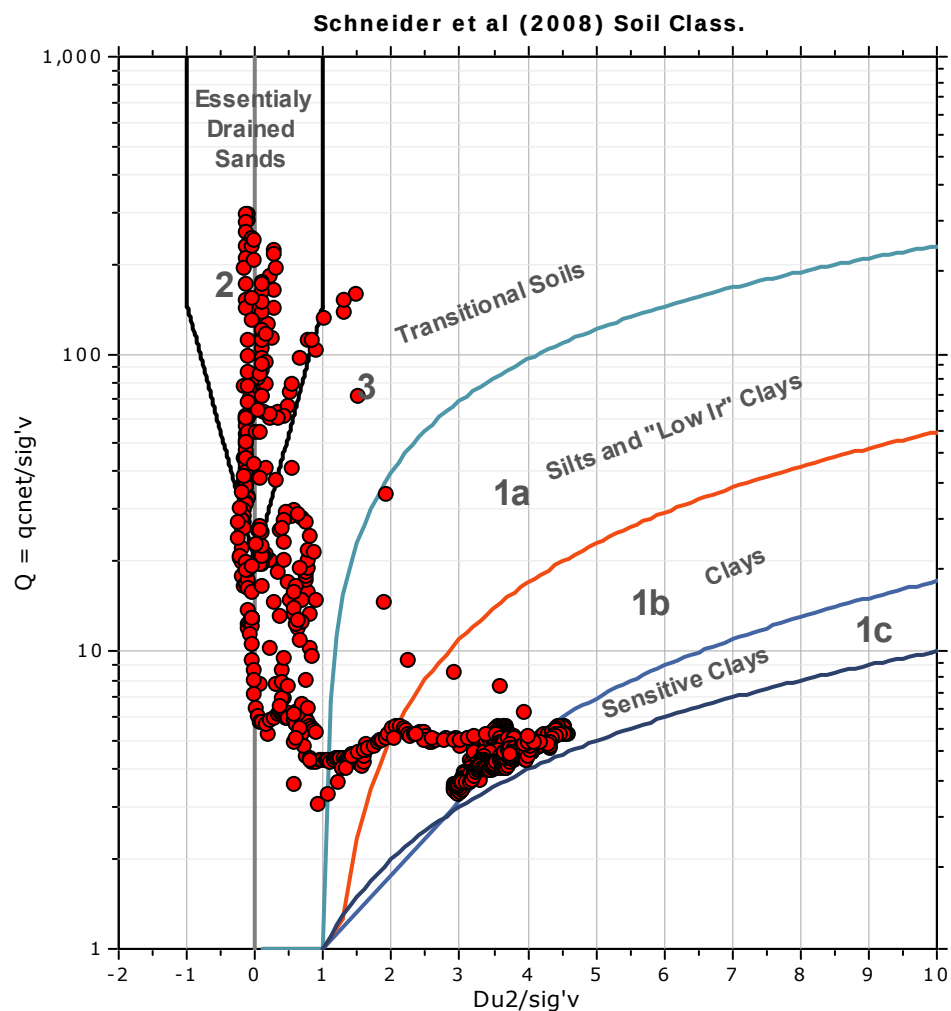
- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravelly sand to sand |
| 2. Organic material | 5. Silty sand to sandy silt | 8. Very stiff sand to clayey sand |
| 3. Clay to silty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



Project: Little River Bridge

Location: Gorham, ME

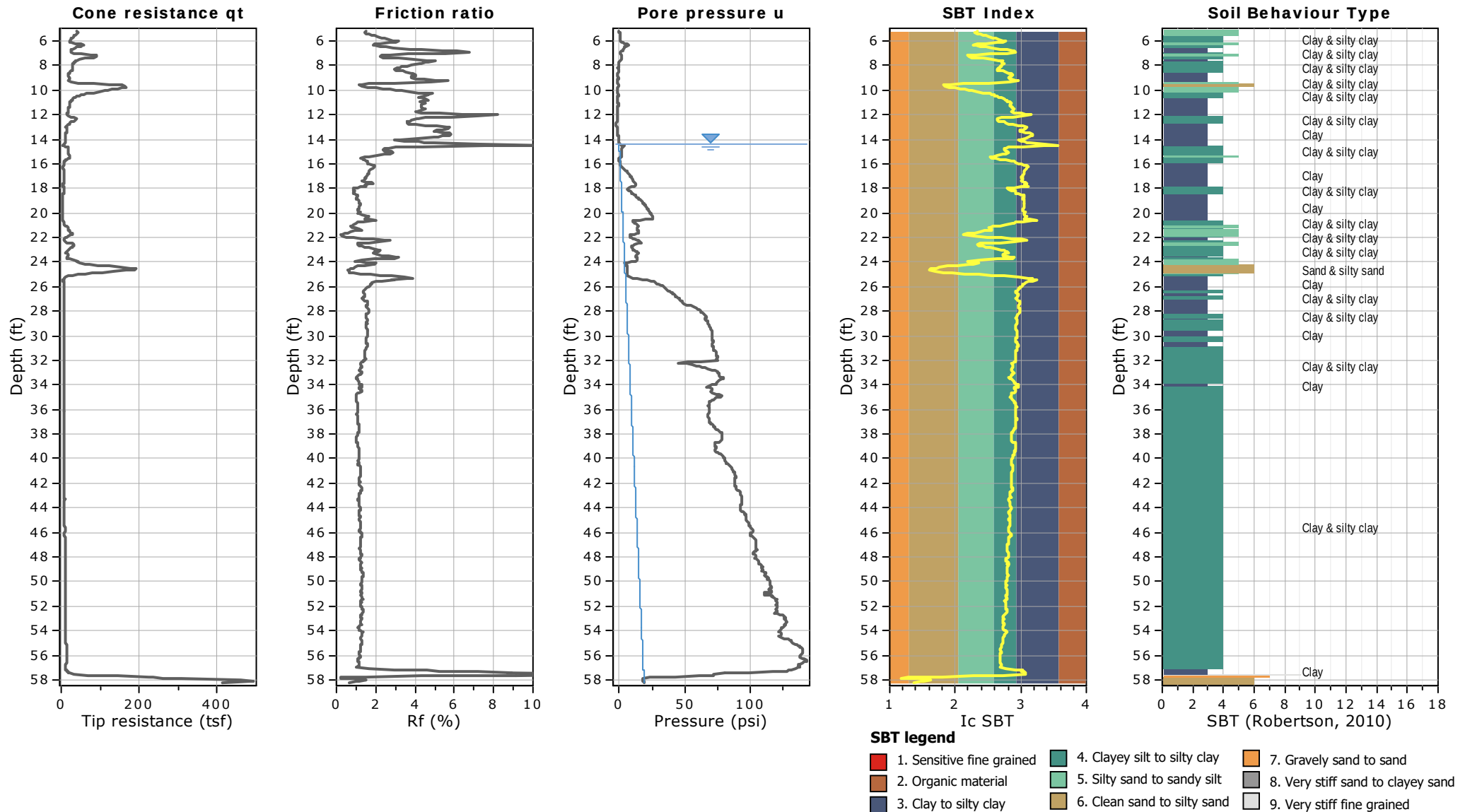
Bq plots (Schneider)





Project: Little River Bridge

Location: Gorham, ME





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

CPT: CPT-GLR-102

Total depth: 58.24 ft, Date: 8/26/2018

Surface Elevation: 87.50 ft

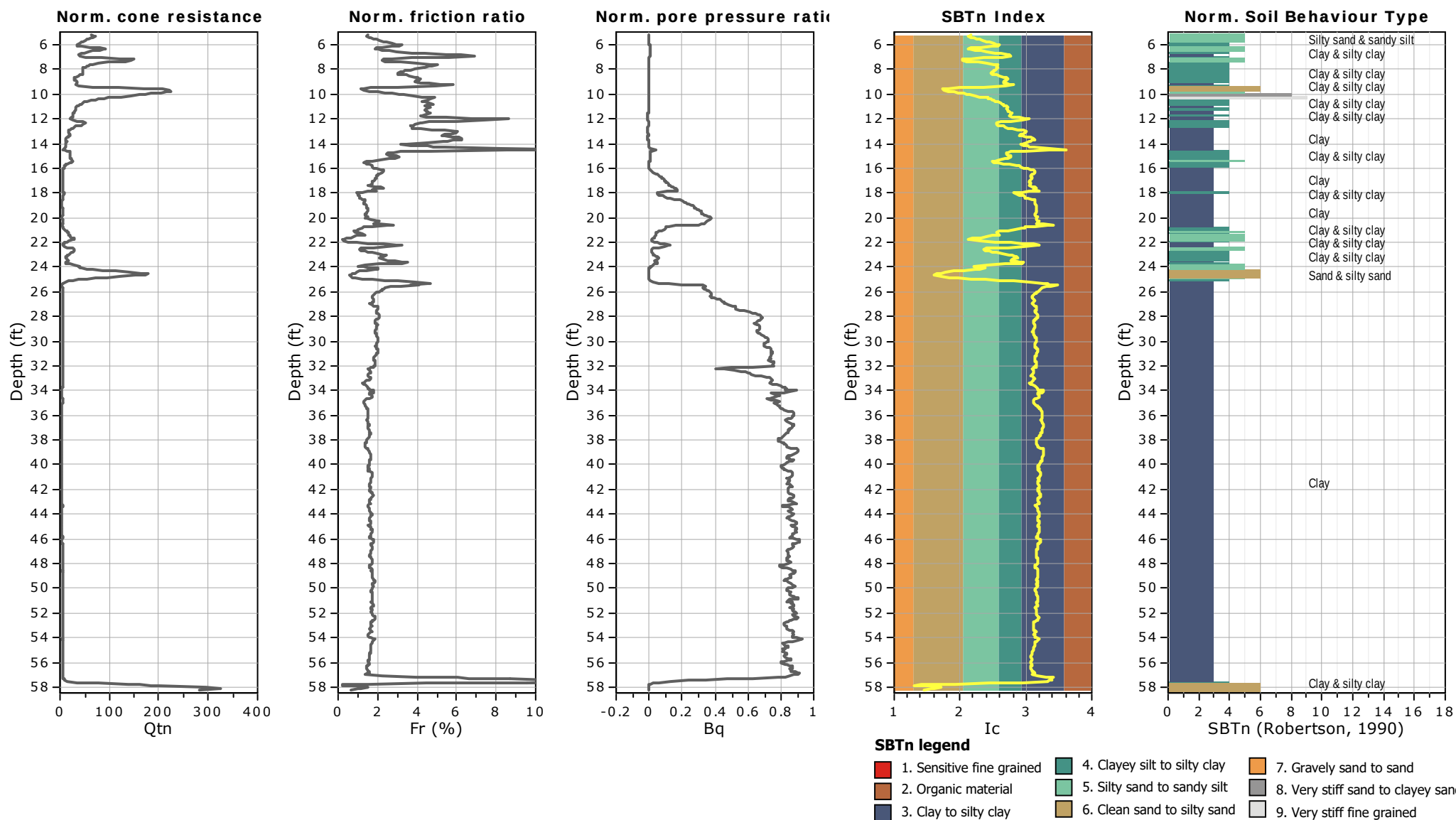
Coords: X:973919.80, Y:320906.30

Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME





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

CPT: CPT-GLR-102

Total depth: 58.24 ft, Date: 8/26/2018

Surface Elevation: 87.50 ft

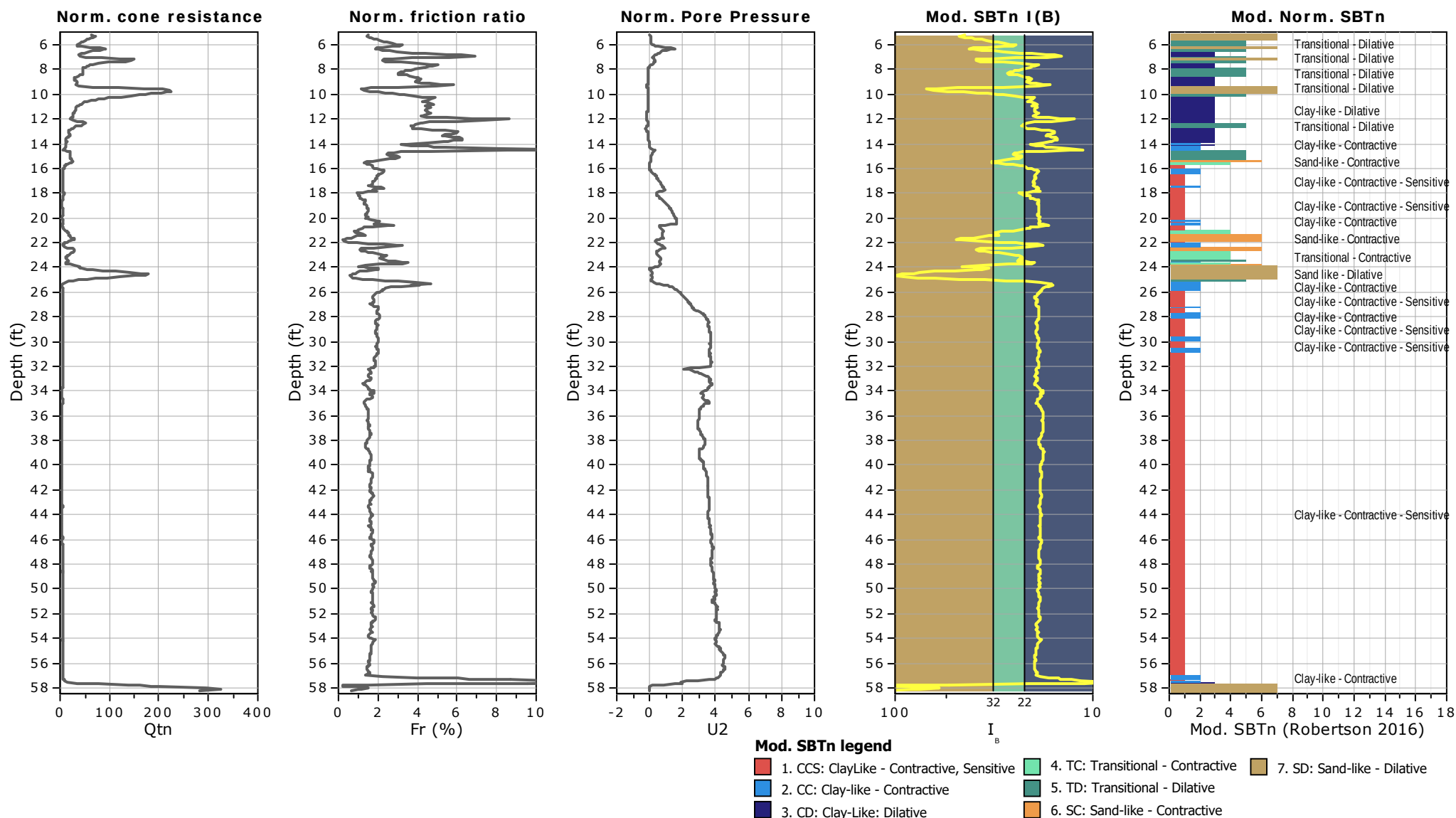
Coords: X:973919.80, Y:320906.30

Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME

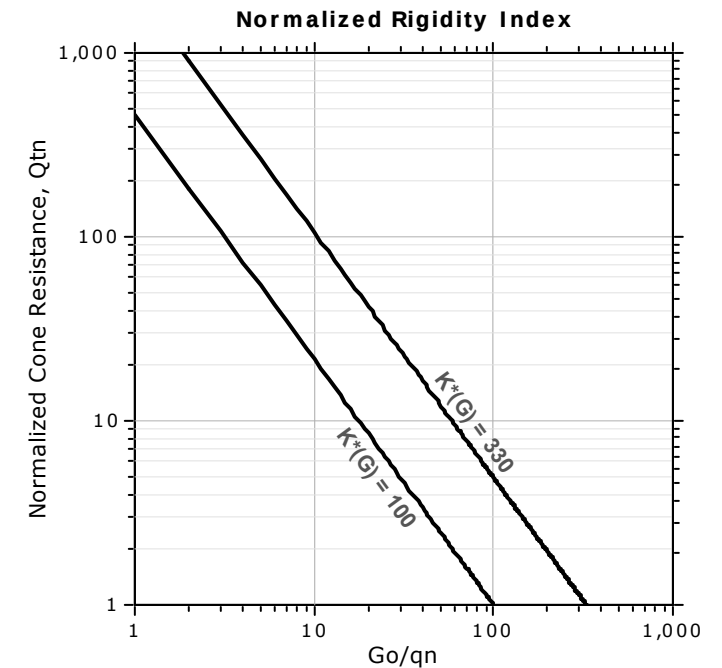
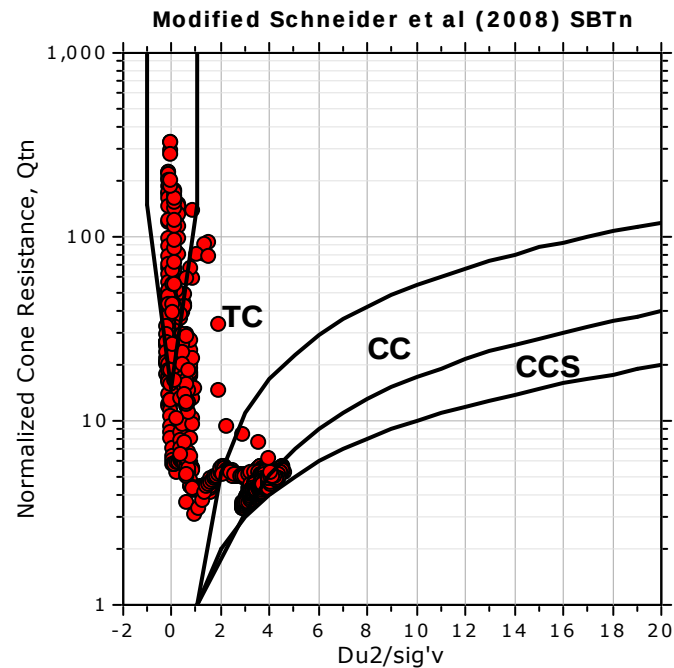
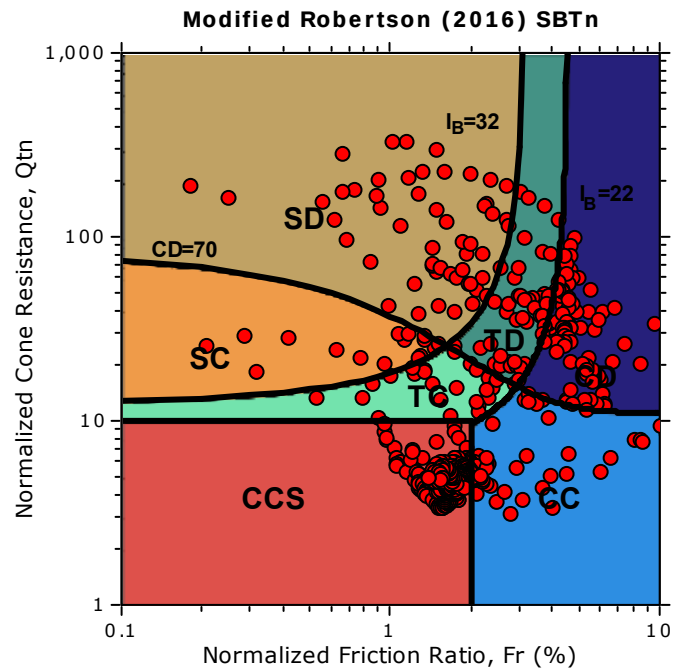




Project: Little River Bridge

Location: Gorham, ME

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

$K^*(G) > 330$: Soils with significant microstructure
(e.g. age/cementation)



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

CPT: CPT-GLR-102

Total depth: 58.24 ft, Date: 8/26/2018

Surface Elevation: 87.50 ft

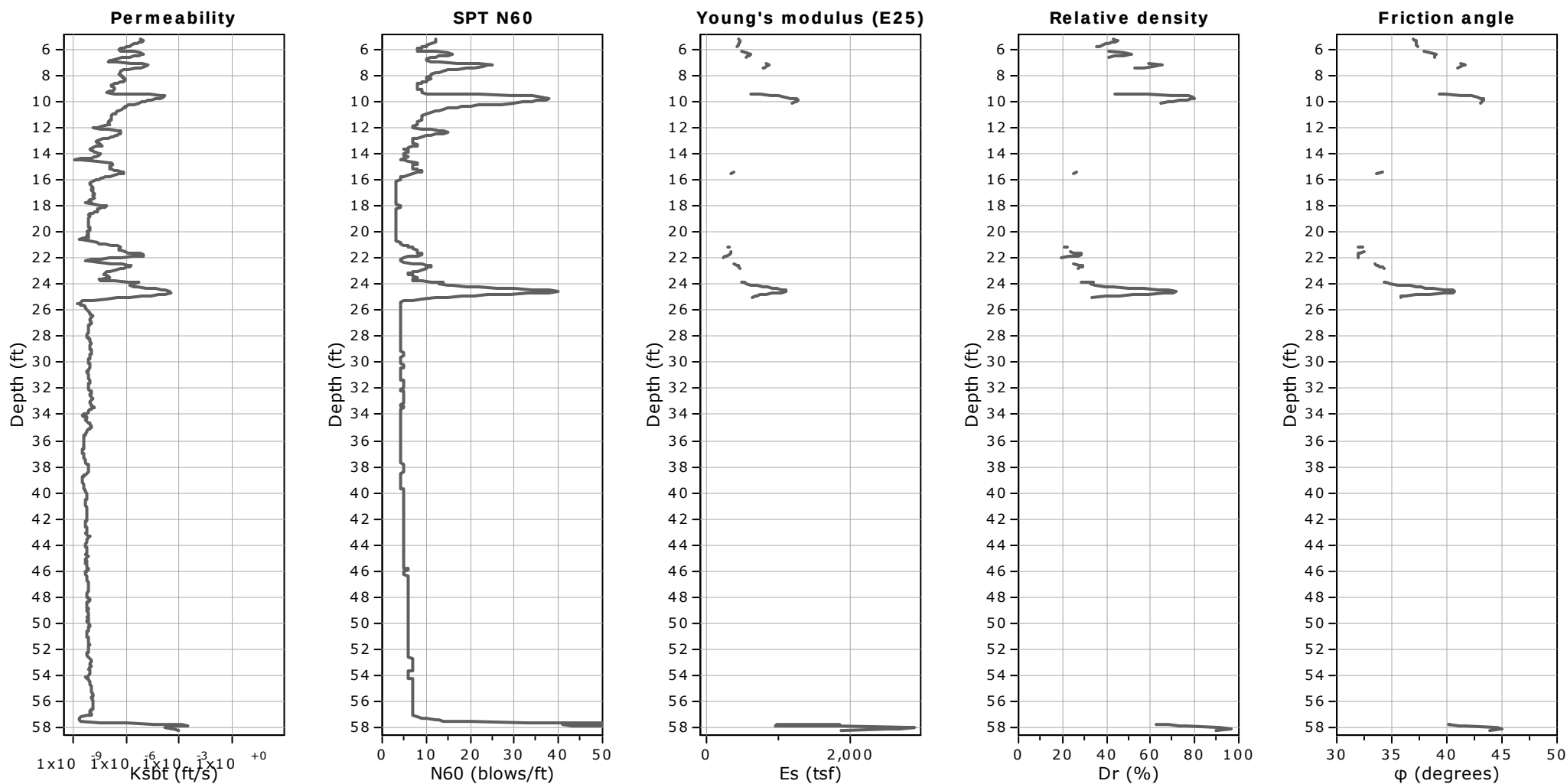
Coords: X:973919.80, Y:320906.30

Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60} : Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr} : 350.0

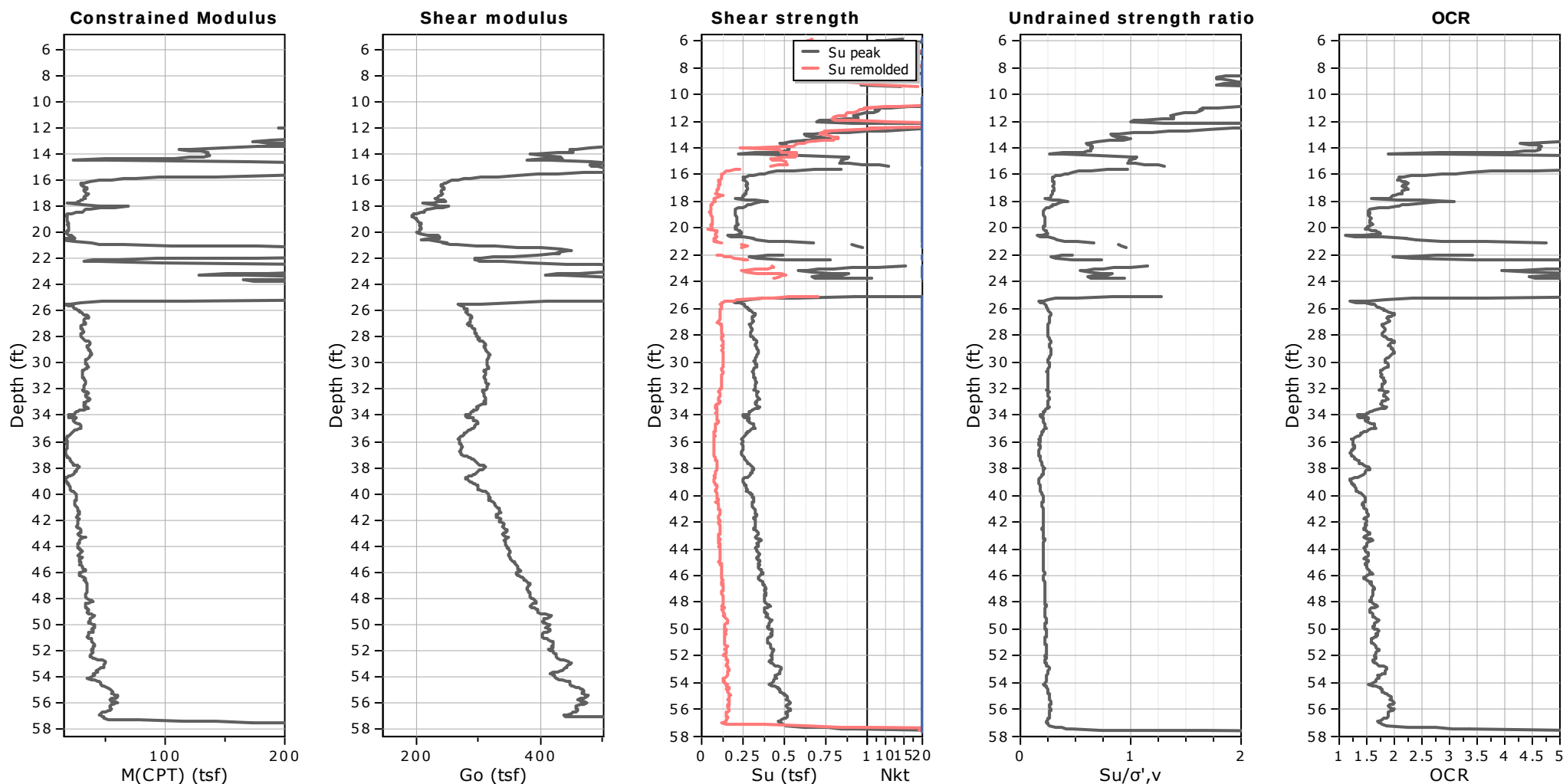
Phi: Based on Kulhawy & Mayne (1990)

—●— User defined estimation data



Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

Constrained modulus: Based on variable α using I_c and Q_{tn} (Robertson, 2009)

Go: Based on variable α using I_c (Robertson, 2009)

Undrained shear strength cone factor for clays, N_{kt} : 20

OCR factor for clays, N_{kt} : 0.36

● User defined estimation data

● Flat Dilatometer Test data



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

CPT: CPT-GLR-102

Total depth: 58.24 ft, Date: 8/26/2018

Surface Elevation: 87.50 ft

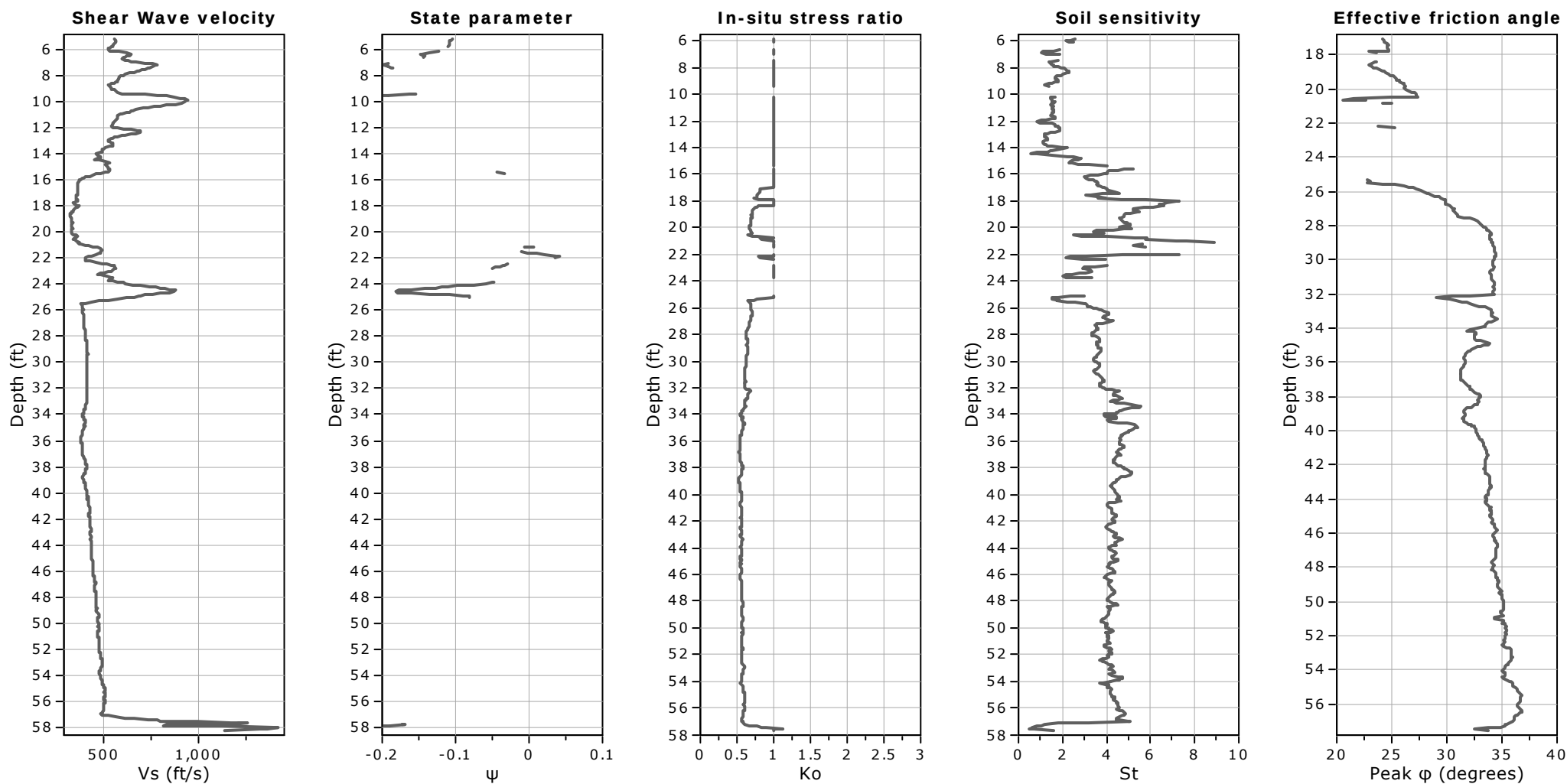
Coords: X:973919.80, Y:320906.30

Cone Type:

Cone Operator: Summit Geoengineering / C. Coolidge

Project: Little River Bridge

Location: Gorham, ME



Calculation parameters

Soil Sensitivity factor, N_s : 7.00

—●— User defined estimation data

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

:: Unit Weight, g (kN/m³) ::

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

where g_w = water unit weight

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

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

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

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

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

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

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

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

(applicable only to $I_c < I_{c_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}} \quad \text{(applicable only to SBT}_n: 5, 6, 7 \text{ and } 8 \text{ or } I_c < I_{c_cutoff})$$

:: State Parameter, ψ ::

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

:: Drained Friction Angle, ϕ (°) ::

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

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

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

If $I_c > 2.20$

$\alpha = 14$ for $Q_{tn} > 14$

$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$

$$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$$

If $I_c \geq 2.20$

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

:: Small strain shear Modulus, G_0 (MPa) ::

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

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

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

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

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

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

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

:: Remolded undrained shear strength, $S_u(rem)$ (kPa) ::

$$S_{u(rem)} = f_s \quad \text{(applicable only to SBT}_n: 1, 2, 3, 4 \text{ and } 9 \text{ or } I_c > I_{c_cutoff})$$

:: Overconsolidation Ratio, OCR ::

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

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

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

:: In situ Stress Ratio, K_0 ::

$$K_0 = (1 - \sin \phi') \cdot OCR^{\sin \phi'}$$

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

:: Soil Sensitivity, S_t ::

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

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

:: Peak Friction Angle, ϕ' (°) ::

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

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX D.2 – CPT DATA REPORT BY SUMMIT GEOENGINEERING SERVICES

August 17, 2018
Summit #18263

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

Reference: Geotechnical Exploration Services
Piezocone Penetration Testing – Little River Bridge on Route 237 Gorham, Maine

Dear Mr. Blaisdell;

We have completed exploration services for a planned bridge project on Route 237 in Gorham, Maine. Summit Geoengineering Services (SGS) is asked to perform piezocone penetration testing (CPT) and prepare this data report summarizing our findings.

Work Description

The explorations were located along the northern embankment of the Little River in Gorham, Maine. At the request of GZA, our scope of services for this project includes the following:

- Performing 2 piezocone penetration tests with shear wave velocity
- Performing 1 dissipation test at location of CPT-GLR-102
- Performing 1 offset boring to conduct field vane shear tests at CPTB-GLR-101

Explorations

Summit Geoengineering Services (SGS) explored the subsurface conditions with 2 piezocone penetration test (CPT) to include shear wave velocity on August 16, 2018. CPT was performed in accordance with ASTM D5778. Shear wave velocity was performed at rod break intervals of 3.3 feet in accordance with ASTM D7400. CPT was advanced using a rubber track mounted PowerProbe 9500 VTR with a Vertek digital cone to a depth of push refusal encountered at depths of 46 and 58 feet below ground surface for CPT-GLR-101 and CPT-GLR-102, respectively. Anchoring was conducted using a single point hollow anchor with start of test depth at 5 feet below ground surface. Parameters obtained include cone resistance (q_c), sleeve friction (f_s), piezocone pore pressure (u_2), and shear wave velocity (V_s). A dissipation test was conducted at CPT-GLR-102 at a depth of 32 feet below ground surface.

Field vane shear tests were conducted at CPTB-GLR-101 at 4-ft intervals from a depth of 16 to 40 feet below ground surface in accordance with ASTM D2573. An Exploration Location Plan and logs of the explorations are attached to this report.

Subsurface Conditions

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

CPT-101

- **5 to 11 feet** - Type 4 (clayey silt to silty clay)
- **11 to 14 feet** – Type 6 (clean to silty sand)
- **14 to 46 feet** - Type 3 (clay to silty clay)

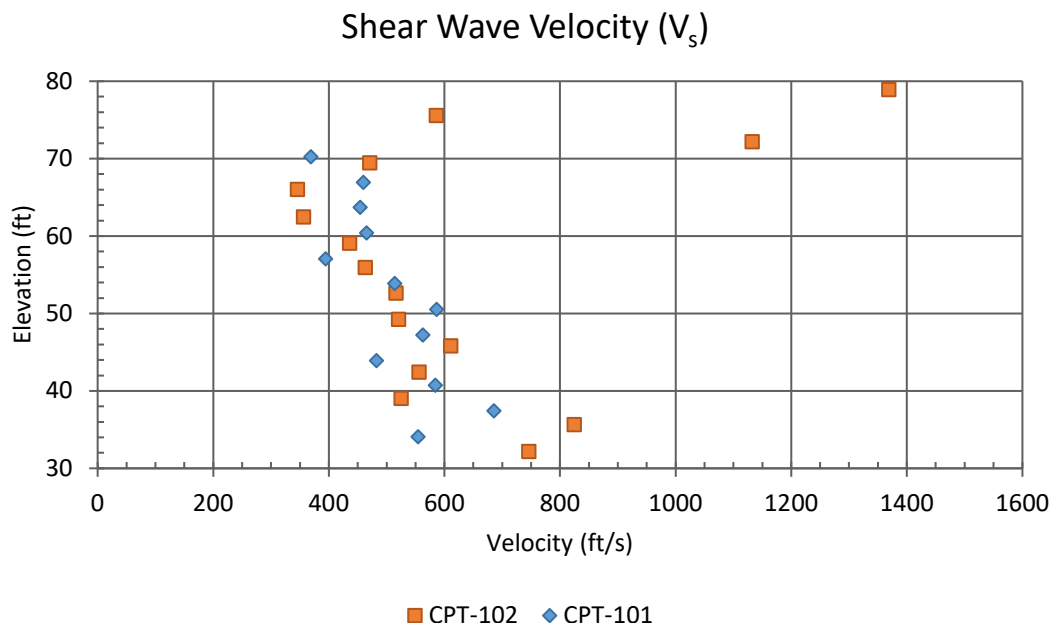
CPT-102

- **5 to 16 feet** - Type 5 (silty sand to sandy silt) to Type 6 (clean sand to silty sand)
- **16 to 21 feet** - Type 3 (clay to silty clay)
- **21 to 25 feet** – Type 4 (clayey silt to silty clay) to Type 6 (clean to silty sand)
- **25 to 58 feet** - Type 3 (clay to silty clay)

Soil behavior type 5 and 6 are typical of granular soils such as roadway fill or native sand layers. Type 3 to type 4 is typical of Presumpscot Formation consisting of silty clay. Groundwater was measured at depths of 5.7 to 14.4 feet below ground surface for CPT-GLR-101 and CPT-GLR-102 respectively. Details of the SBT_N profiling is shown on the attached logs.

Shear Wave Velocity

Shear wave velocity tests were conducted during performed of the cone penetration tests during rod break intervals. Results of the shear wave velocity testing are shown below:

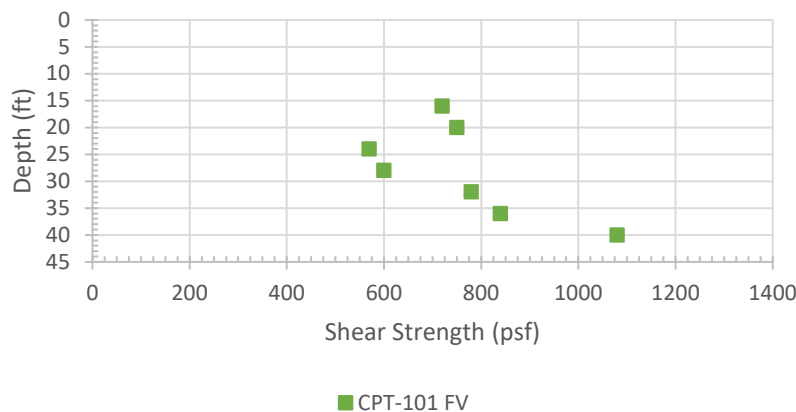


Waterfall plots showing details of the shear wave velocity results are included at the end of this report. In summary, results range from 345 to 1,370 ft/s for an average of 510 ft/s at CPT-GLR-101 and 630 ft/s at CPT-GLR-102.

Field Vane Shear Test

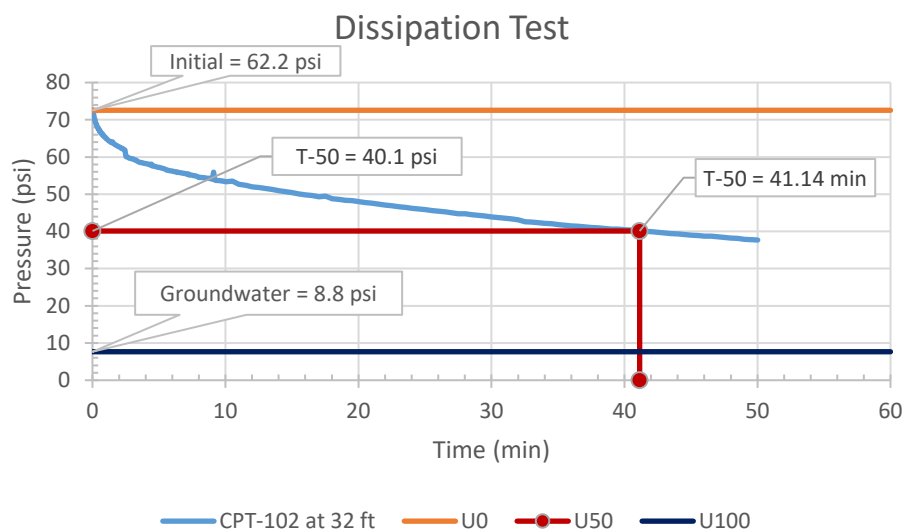
Field vane shear tests were conducted at CPT-GLR-101 at 4-ft intervals from a depth of 16 to 40 feet below ground surface. Results of the field vane shear tests are shown below:

Undrained Shear Strength (S_u)



Dissipation Test

A dissipation test was performed at a depth of 32 feet below ground surface at CPT-GLR-102. The estimated horizontal coefficient of consolidation (C_h) using T_{50} of 41.1 minutes from the dissipation test is 0.015 ft²/day. Graphic results of the dissipation test are shown below:



Closure

Our evaluation is based on professional judgment and generally accepted principles of geotechnical engineering. Some changes in subsurface conditions from those presented in this report may occur and would not be evident until construction. Should subsurface conditions or project information differ materially from those described, Summit should be notified so that we can re-evaluate our recommendations.

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

Sincerely yours,
Summit Geoengineering Services

A handwritten signature in blue ink, reading "Craig W. Coolidge".

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



Attachments: EXPLORATION LOCATION PLAN, CPT LOGS, FIELD VANE SHEAR LOGS

LEGEND



SUMMIT TEST BORING
(AUGUST 16, 2018)



SUMMIT CONE PENETRATION
TEST (AUGUST 16, 2018)

MAP REFERENCE

MAP IMAGERY FROM GOOGLE EARTH,
2018.



EXPLORATION LOCATION PLAN

LITTLE RIVER BRIDGE ON RT 237, GORHAM, ME
PREPARED FOR

GZA GEOENVIRONMENTAL, INC.

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

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

SUMMIT
GEOENGINEERING SERVICES
www.summitgeoeng.com

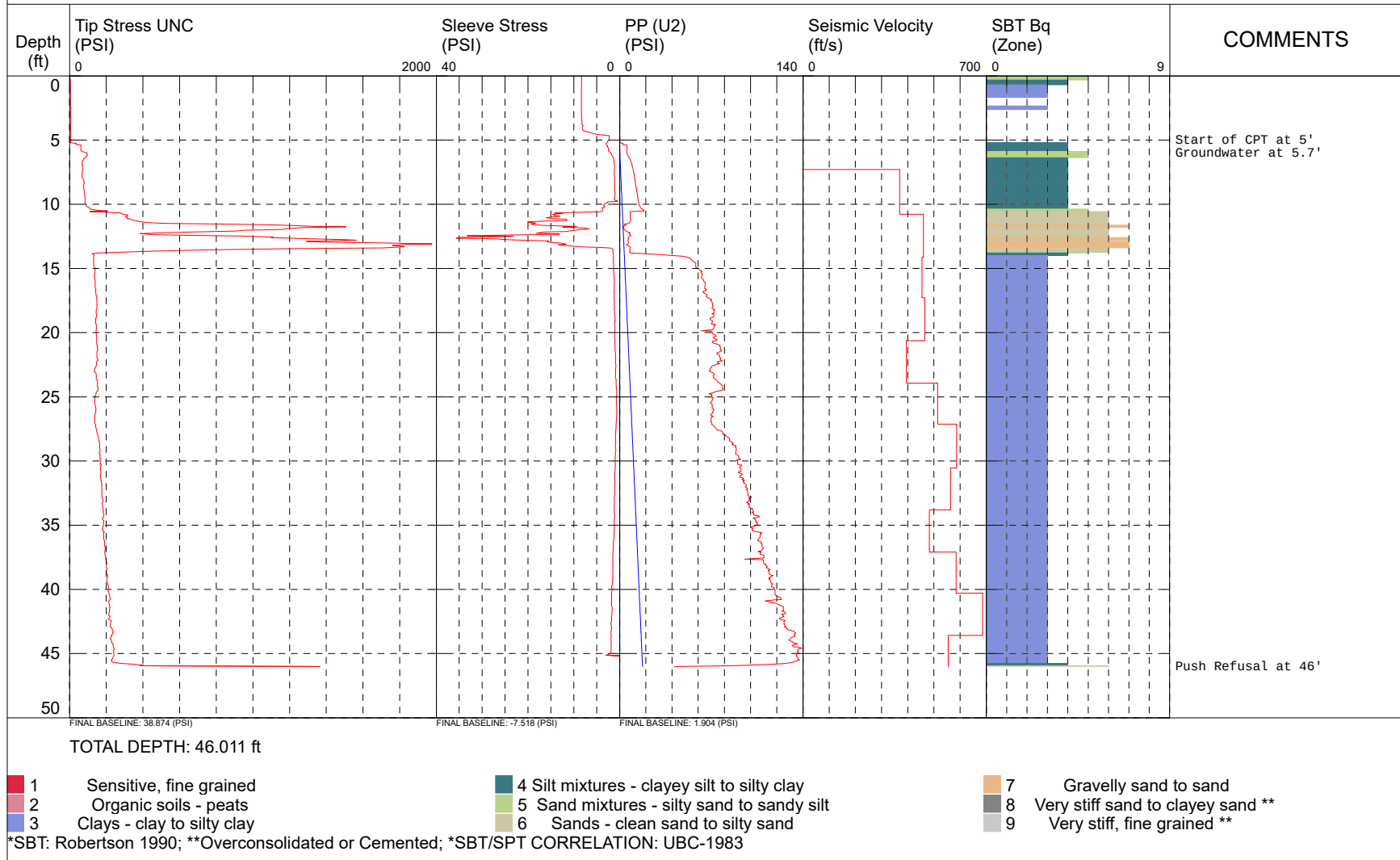
DATE: AUGUST 2018	DRAWN BY: JRG	CHECKED BY: CWC
REV.: 9-25-18	SCALE: 1" = 20'	JOB: 18263

CPT-GLR-101



COMPANY: Summit Geotechnical Services
 OPERATOR: C. Coolidge, P.E.
 CREW: J. Gilbert
 CLIENT: GZA
 CLIENT REP: Erik Friede, E.I.

TEST DATE: Thu 16/Aug/2018
 TEST ID: CPT-101
 PROJECT: 18263
 SITE: 240 Mosher Road
 LOCATION: Gorham, ME

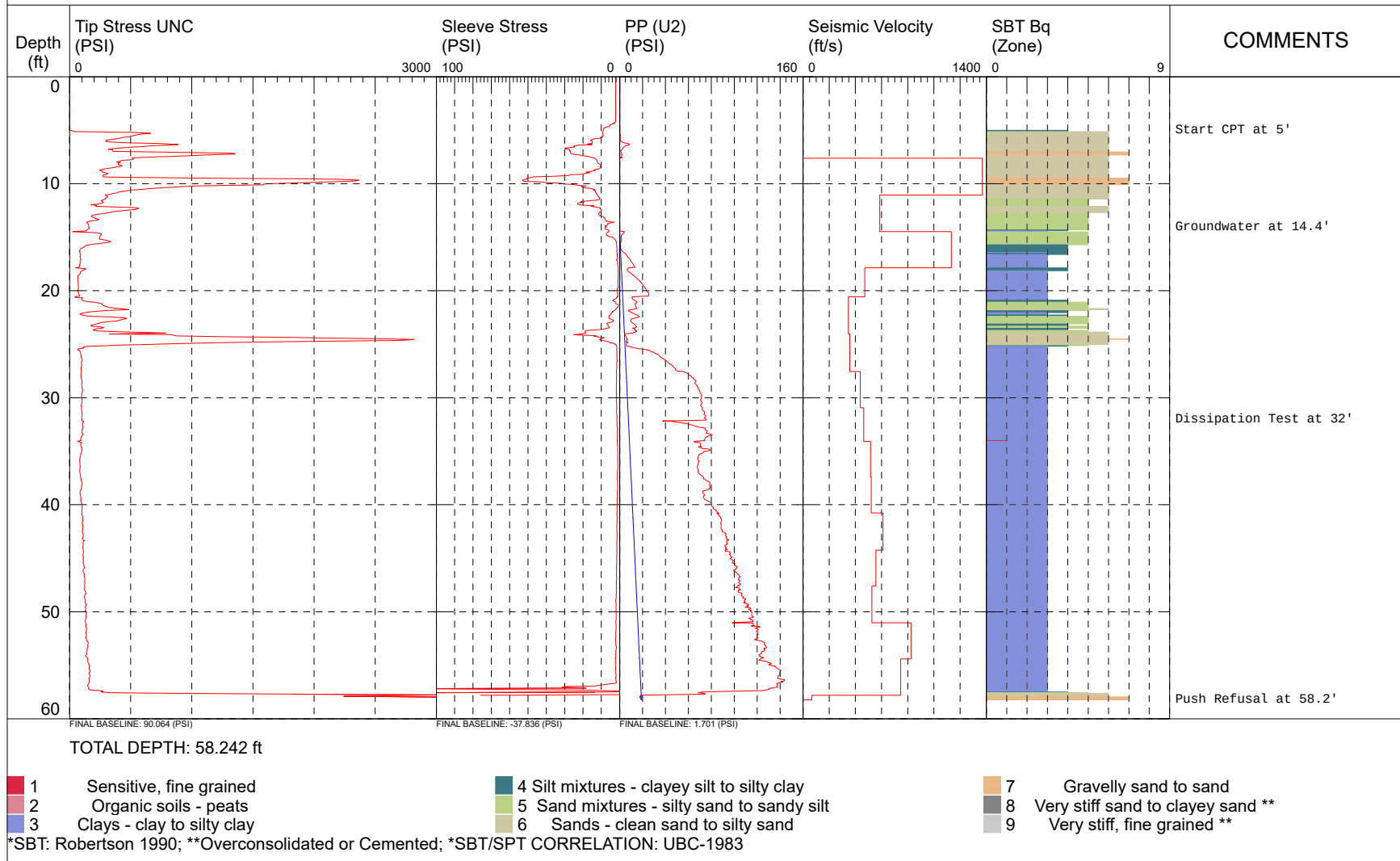


CPT-GLR-102

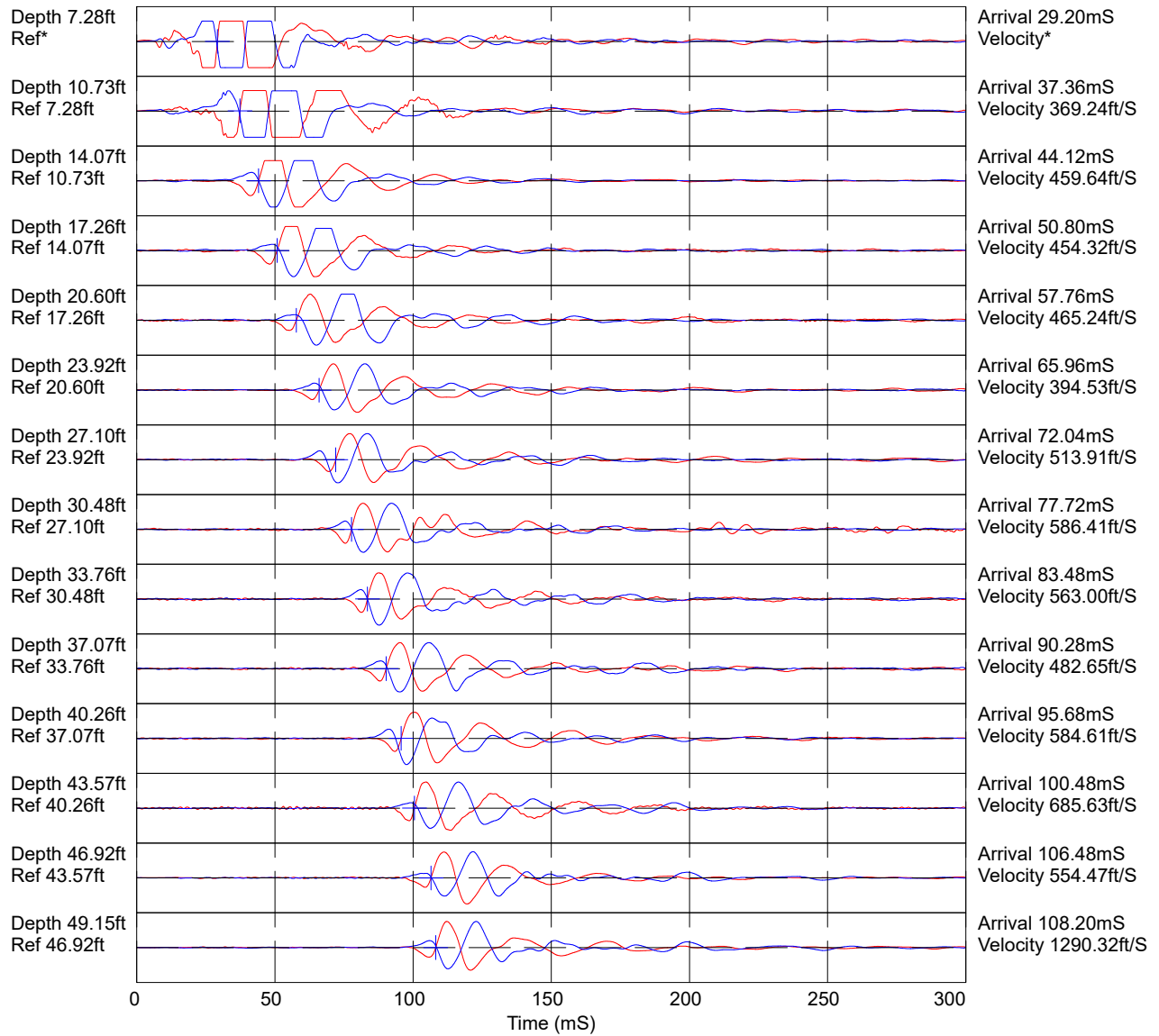


COMPANY: Summit Geotechnical Services
 OPERATOR: C. Coolidge, P.E.
 CREW: J. Gilbert
 CLIENT: GZA
 CLIENT REP: Erik Friede, E.I.

TEST DATE: Thu 16/Aug/2018
 TEST ID: CPT-102
 PROJECT: 18263
 SITE: 240 Mosher Road
 LOCATION: Gorham, ME



PROJECT: 18263

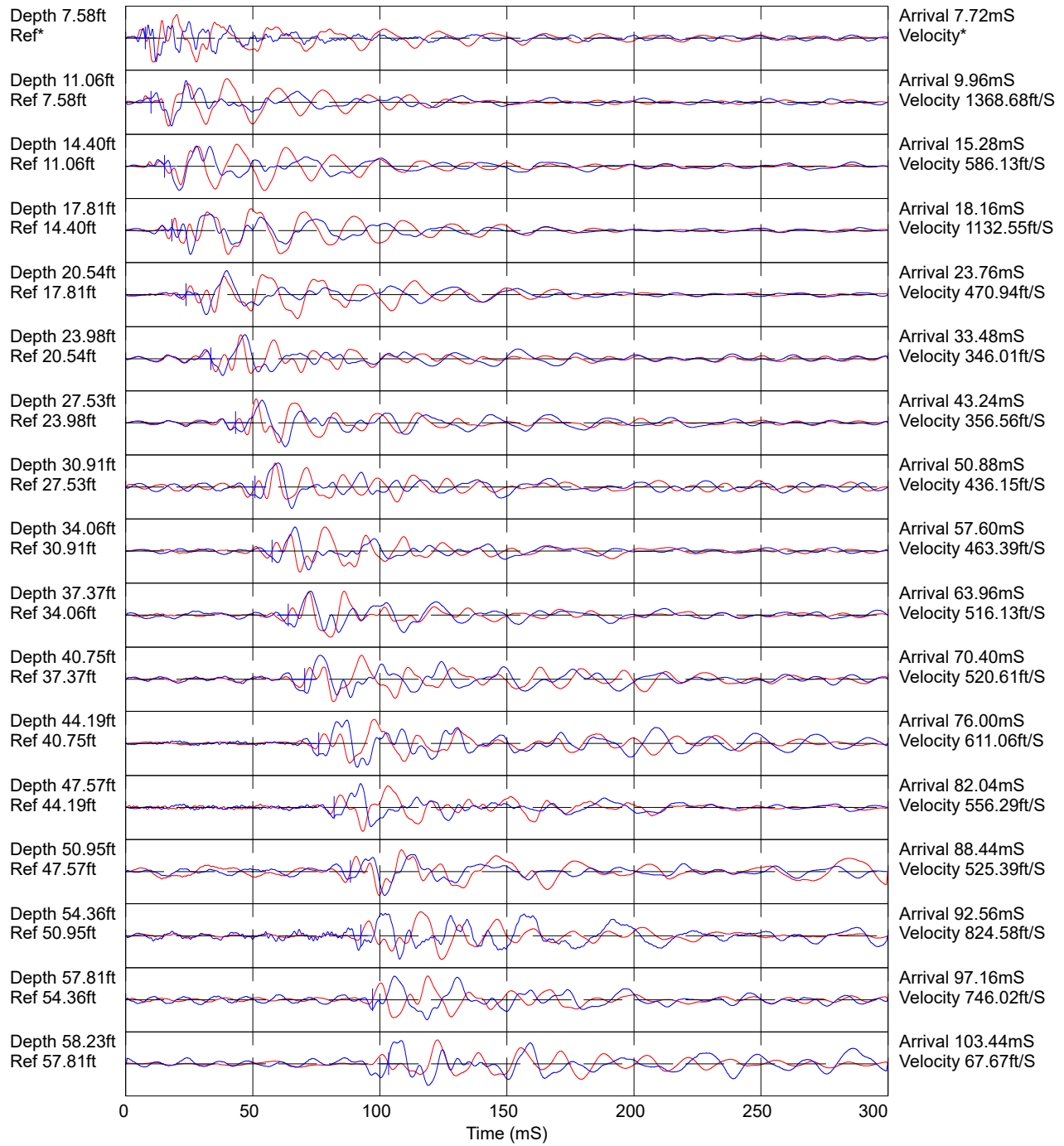


Hammer to Rod String Distance (ft): 4.92

* = Not Determined

TEST ID: CPT-GLR-101

PROJECT: 18263



Hammer to Rod String Distance (ft): 4.92

* = Not Determined

TEST ID: CPT-GLR-102

					SOIL BORING LOG				Boring #: CPTB-GLR-101	
					Project: Bridge Replacement				Project #: 18263	
					Location: Little River Bridge on Route 237				Sheet: 1 of 2	
					City, State: Gorham, ME				Chkd by: CWC	
Drilling Co: Summit Geoengineering, Inc.					Boring Elevation: 79.5 feet +/-					
Driller: C. Coolidge, P.E.					Reference: Survey Elevation Provided by GZA					
Summit Staff: J. Gilbert, S. Floyd					Date started: 8/16/2018 Date Completed: 8/16/2018					
DRILLING METHOD			SAMPLER		ESTIMATED GROUND WATER DEPTH					
Vehicle: AMS			Length: 24" SS		Date	Depth	Elevation	Reference		
Model: 9500 VTR			Diameter: 2"OD/1.5"ID		8/16/2018	5.7 feet +/-	73.8 feet +/-	Measured in Open Hole		
Method: 3-inch Casing			Hammer: 140 lb							
Hammer Style: Auto			Method: ASTM D1586							
Depth (ft.)			Depth (ft.)		Elev. (ft.)	SAMPLE DESCRIPTION		Geological/ Test Data	Geological Stratum	
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										
FIELD VANES						65' +/- $S_u = 720 \text{ psf}$, $S_{u(r)} = 60 \text{ psf}$ (24 ft-lb, 2 ft-lb)				
16			Tip of Vane							
	FV-1		16							
17										
18										
19										
20										
21										
22										
						61' +/- $S_u = 750 \text{ psf}$, $S_{u(r)} = 75 \text{ psf}$ (25 ft-lb, 2.5 ft-lb)				
	FV-2		20							

Granular Soils		Cohesive Soils		% Composition ASTM D2487	NOTES: PP = Pocket Penetrometer, MC = Moisture Content LL = Liquid Limit, PI = Plastic Index, FV = Field Vane Test <u>Bedrock Joints</u> S_u = Undrained Shear Strength, $S_{u(r)}$ = Remolded Shear Strength Shallow = 0 to 35 degrees Dipping = 35 to 55 degrees Steep = 55 to 90 degrees Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200	Soil Moisture Condition Dry: S = 0% Humid: S = 1 to 25% Damp: S = 26 to 50% Moist: S = 51 to 75% Wet: S = 76 to 99% Saturated: S = 100%
Blows/ft.	Density	Blows/ft.	Consistency			
0-4	V. Loose	<2	V. soft			
5-10	Loose	2-4	Soft	< 5% Trace		
11-30	Compact	5-8	Firm	5-15% Little		
31-50	Dense	9-15	Stiff	15-30% Some		
>50	V. Dense	16-30	V. Stiff	> 30% With		
		>30	Hard			

					SOIL BORING LOG				Boring #: CPTB-GLR-101		
Drilling Co: Summit Geoengineering, Inc.					Project: Bridge Replacement				Project #: 18263		
Driller: C. Coolidge, P.E.					Location: Little River Bridge on Route 237				Sheet: 2 of 2		
Summit Staff: J. Gilbert, S. Floyd					City, State: Gorham, ME				Chkd by: CWC		
Boring Elevation: 79.5 feet +/-					Reference: Survey Elevation Provided by GZA						
Date started: 8/16/2018					Date Completed: 8/16/2018						
DRILLING METHOD			SAMPLER		ESTIMATED GROUND WATER DEPTH						
Vehicle: AMS			Length: 24" SS		Date	Depth	Elevation	Reference			
Model: 9500 VTR			Diameter: 2"OD/1.5"ID		8/16/2018	5.7 feet +/-	73.8 feet +/-	Measured in Open Hole			
Method: 3-inch Casing			Hammer: 140 lb								
Hammer Style: Auto			Method: ASTM D1586								
Depth (ft.)		Depth (ft.)		Elev. (ft.)	SAMPLE DESCRIPTION			Geological/ Test Data		Geological Stratum	
23					FIELD VANES S _u = 570 psf, S _{u(r)} = 45 psf (19 ft-lb, 1.5 ft-lb)						
24											
	FV-3	24									
25											
26											
27					S _u = 600 psf, S _{u(r)} = 30 psf (20 ft-lb, 1 ft-lb)						
28											
	FV-4	28									
29											
30											
31					S _u = 780 psf, S _{u(r)} = 45 psf (26 ft-lb, 1.5 ft-lb)						
32											
	FV-5	32									
33											
34											
35					S _u = 840 psf, S _{u(r)} = 60 psf (28 ft-lb, 2 ft-lb)						
36											
	FV-6	36									
37											
38											
39					S _u = 1,080 psf, S _{u(r)} = 90 psf (36 ft-lb, 3 ft-lb)						
40											
	FV-7	40									
41											
42											
43					End of Exploration at 40'						
44											

Granular Soils		Cohesive Soils		% Composition	NOTES:	Soil Moisture Condition
Blows/ft.	Density	Blows/ft.	Consistency	ASTM D2487		
0-4	V. Loose	<2	V. soft		PP = Pocket Penetrometer, MC = Moisture Content	Dry: S = 0%
5-10	Loose	2-4	Soft	< 5% Trace	LL = Liquid Limit, PI = Plastic Index, FV = Field Vane Test	Humid: S = 1 to 25%
11-30	Compact	5-8	Firm	5-15% Little	Su = Undrained Shear Strength, Su(r) = Remolded Shear Strength	Damp: S = 26 to 50%
31-50	Dense	9-15	Stiff	15-30% Some	Shallow = 0 to 35 degrees	Moist: S = 51 to 75%
>50	V. Dense	16-30	V. Stiff	> 30% With	Dipping = 35 to 55 degrees	Wet: S = 76 to 99%
		>30	Hard		Steep = 55 to 90 degrees	Saturated: S = 100%
					Boulders = diameter > 12 inches, Cobbles = diameter < 12 inches and > 3 inches	
					Gravel = < 3 inch and > No 4, Sand = < No 4 and >No 200, Silt/Clay = < No 200	



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX E – 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: E. Freide Assigned By: E. Freide Collected By: Client	Project Information: Little River Bridge, No 3112 Gorham, ME GZA Project Number: 09.0025988.00 Summary Page: 1 of 2 Report Date: 09.17.18

LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W _{opt} (%)	γ_d MAX (pcf) W _{opt} (%) (Corr.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability (cm/sec)	
				D2216	D4318		D6913			D2874	D854			D1557			D1883			
BB-GLR-101	1D	1-3	S-1	4.1			22.2	63.6	14.2											Brown f-c SAND, some f-c Gravel, little Silt
BB-GLR-101	2D	5-7	S-2	12.9			3.7	52.5	43.8											Brown Clayey fine to medium SAND, trace fine Gravel
BB-GLR-101	3D	10-12	S-3	27.1	34	20														Grey Silty CLAY
BB-GLR-101	4D	15-17	S-4	42.5			0.0	4.5	95.5											Brown Silty CLAY, trace fine Sand
BB-GLR-101	5D	20-22	S-5	49.3	52	23														Grey Silty CLAY
BB-GLR-101	6D	27-29	S-6	43.2	47	22														Grey Silty CLAY

Reviewed By SKH

09.18.2018

	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: E. Freide Assigned By: E. Freide Collected By: Client	Project Information: Little River Bridge, No 3112 Gorham, ME GZA Project Number: 09.0025988.00 Summary Page: 2 of 2 Report Date: 09.17.18

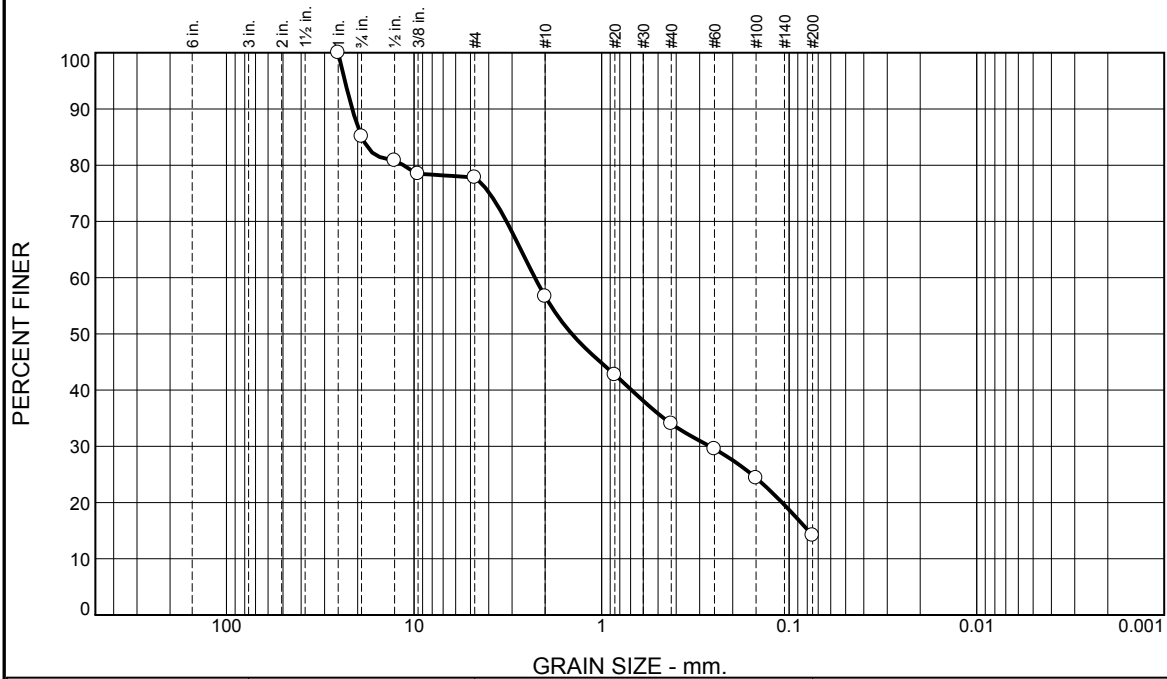
LABORATORY TESTING DATA SHEET

Boring ID	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests								Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ_d MAX (pcf) W _{opt} (%)	γ_d MAX (pcf) W _{opt} (%) (Corr.)	Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"	Permeability (cm/sec)	
				D2216	D4318		D6913			D2874	D854			D1557			D1883			
BB-GLR-102	1D	1-3	S-7	4.4			19.2	69.3	11.5											Brown f-c SAND, little fine Gravel, little Silt
BB-GLR-102	2D	5-7	S-8	18.6			7.7	36.1	56.2											Brown Sandy CLAY, trace fine Gravel
BB-GLR-102	3D	10-12	S-9	28.3																
BB-GLR-102	4D	15-17	S-10	28.8			0.0	50.6	49.4											Grey - Brown Clayey fine SAND
BB-GLR-102	6D	20-22	S-11	25.1			0.0	56.1	43.9											Grey Clayey fine SAND
BB-GLR-102	7D	25-27	S-12	57.7																
BB-GLR-102	8D	35-37	S-13	42.4																
BB-GLR-102	9D	45-47	S-14	42.7																

Reviewed By Sta Ro

09.18.2018

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.9	7.3	21.1	22.7	19.8	14.2	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	85.1		
0.5"	80.8		
0.375"	78.5		
#4	77.8		
#10	56.7		
#20	42.7		
#40	34.0		
#60	29.5		
#100	24.4		
#200	14.2		

* (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= 21.3525 D₈₅= 18.9730 D₆₀= 2.2687
D₅₀= 1.4405 D₃₀= 0.2647 D₁₅= 0.0791
D₁₀= C_u= C_c=

Remarks

Date Received: 09.04.18 Date Tested: 09.06.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-101
Sample Number: 1D

Depth: 1-3'

Date Sampled:

Thielsch Engineering Inc.

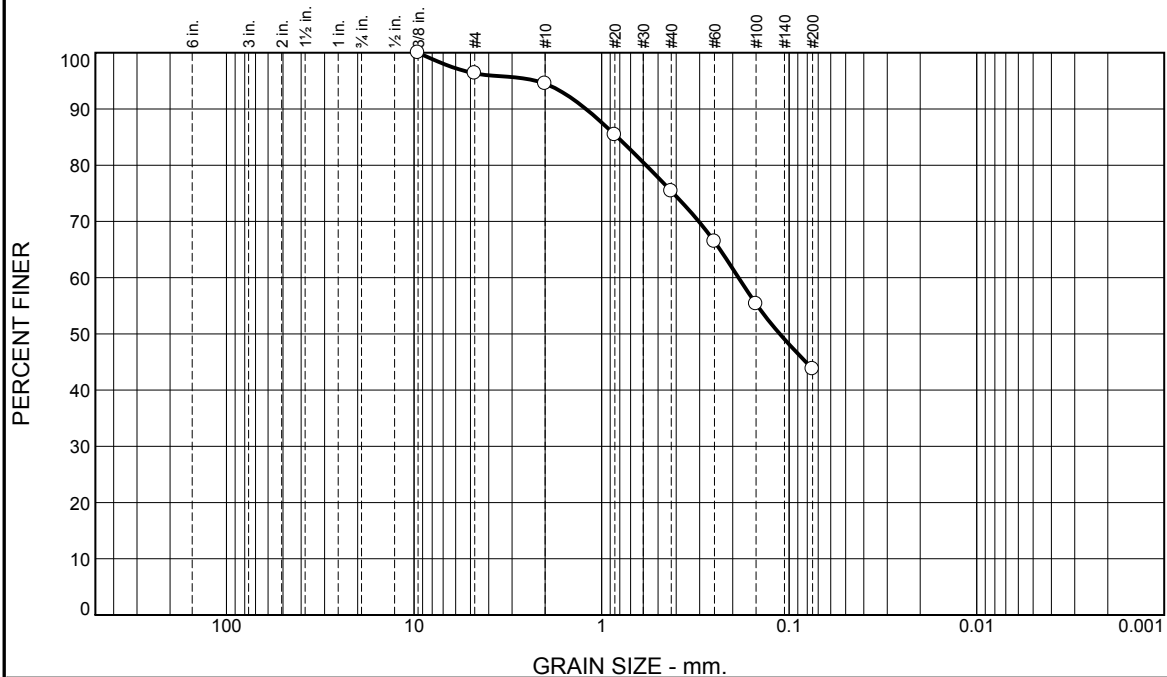
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.7	1.8	19.1	31.6	43.8	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375"	100.0		
#4	96.3		
#10	94.5		
#20	85.4		
#40	75.4		
#60	66.4		
#100	55.3		
#200	43.8		

* (no specification provided)

Material Description

Brown Clayey fine to medium SAND, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 1.2116 D₈₅= 0.8243 D₆₀= 0.1861
D₅₀= 0.1122 D₃₀= C_u=
D₁₀= C_c=

Remarks

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

Date Received: 09.04.18 Date Tested: 09.06.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-101
Sample Number: 2D

Depth: 5-7'

Date Sampled:

Thielsch Engineering Inc.

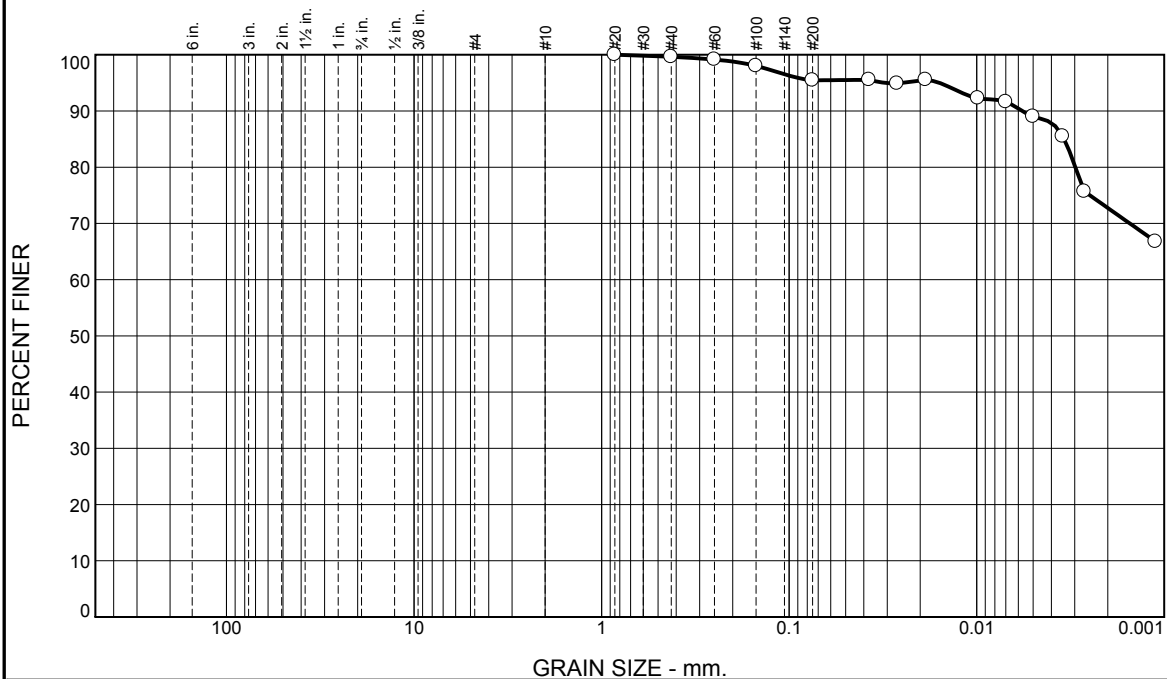
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.4	4.1	6.5	89.0

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#20	100.0		
#40	99.6		
#60	99.1		
#100	98.0		
#200	95.5		
0.0375 mm.	95.5		
0.0266 mm.	94.9		
0.0187 mm.	95.5		
0.0098 mm.	92.3		
0.0070 mm.	91.6		
0.0050 mm.	89.0		
0.0035 mm.	85.5		
0.0027 mm.	75.7		
0.0011 mm.	66.8		

* (no specification provided)

Material Description

Brown Silty CLAY, trace fine Sand

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 0.0057 D₈₅= 0.0034 D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/16".

Date Received: 09.04.18 Date Tested: 09.07.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-101
Sample Number: 4D

Depth: 15-17'

Date Sampled:

Thielsch Engineering Inc.

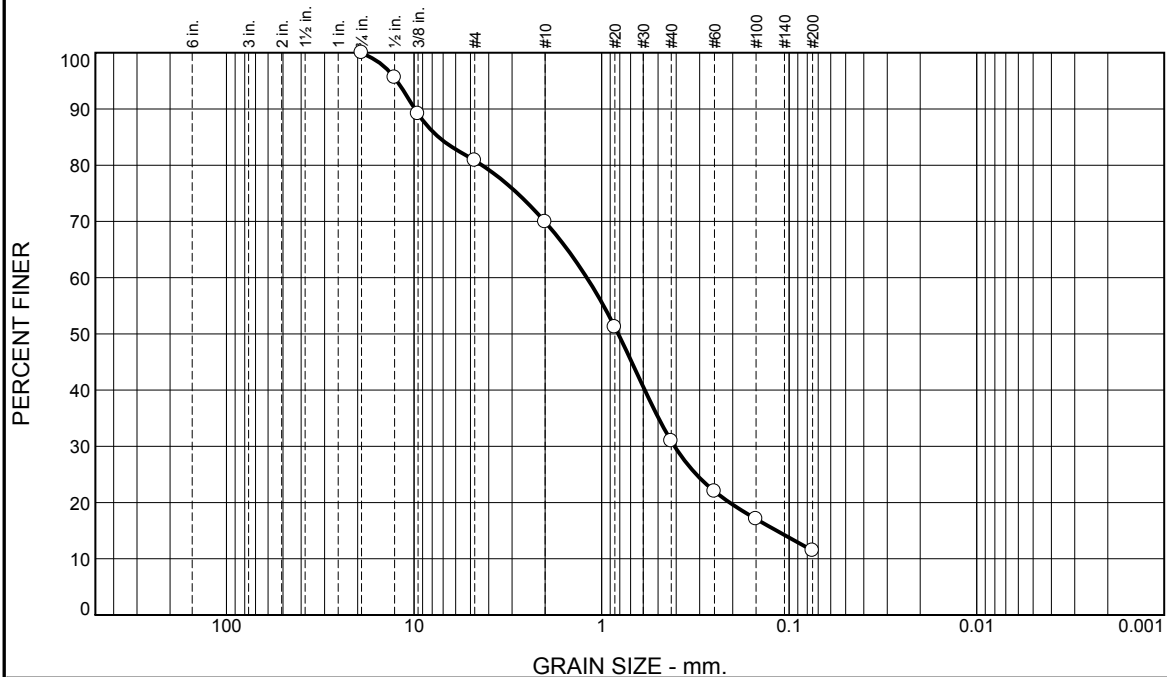
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.2	10.9	39.0	19.4	11.5	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	95.6		
0.375"	89.1		
#4	80.8		
#10	69.9		
#20	51.2		
#40	30.9		
#60	22.0		
#100	17.1		
#200	11.5		

* (no specification provided)

Material Description

Brown f-c SAND, little fine Gravel, little Silt

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

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

Coefficients

D₉₀= 9.9114 D₈₅= 7.3944 D₆₀= 1.1988
D₅₀= 0.8154 D₃₀= 0.4081 D₁₅= 0.1166
D₁₀= C_u= C_c=

Remarks

Date Received: 09.04.18 Date Tested: 09.07.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-102
Sample Number: 1D

Depth: 1-3'

Date Sampled:

Thielsch Engineering Inc.

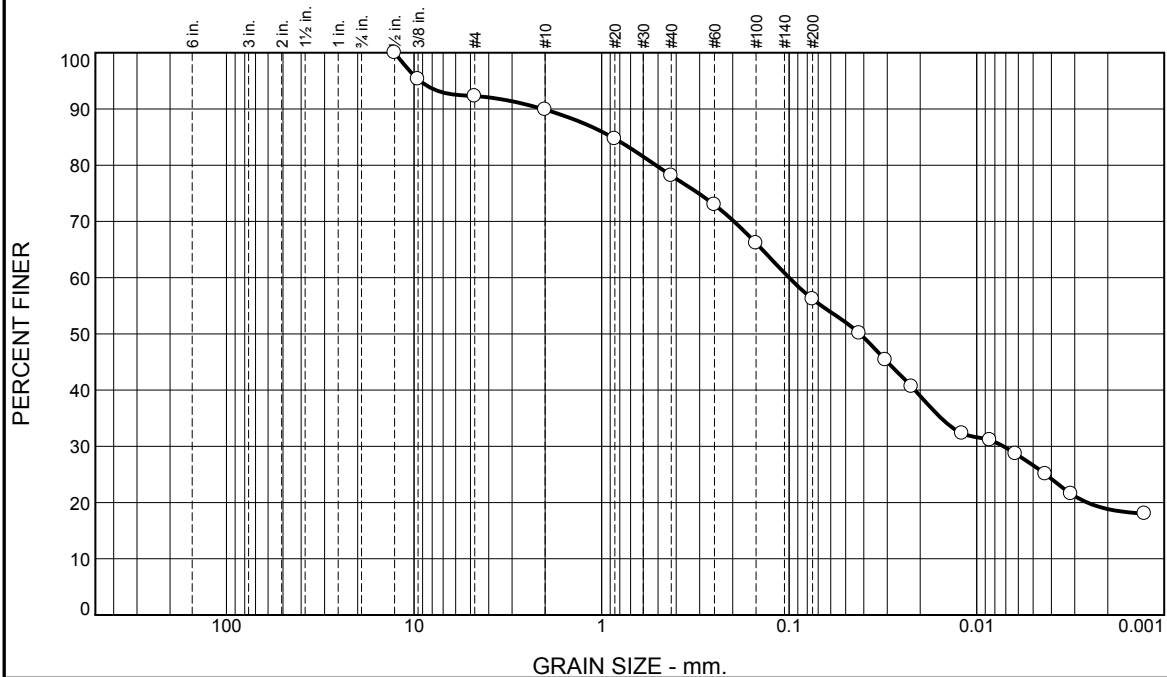
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.7	2.4	11.7	22.0	29.6	26.6

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	95.3		
#4	92.3		
#10	89.9		
#20	84.7		
#40	78.2		
#60	73.0		
#100	66.1		
#200	56.2		
0.0423 mm.	50.1		
0.0307 mm.	45.4		
0.0222 mm.	40.6		
0.0120 mm.	32.3		
0.0085 mm.	31.1		
0.0062 mm.	28.7		
0.0043 mm.	25.1		
0.0031 mm.	21.6		
0.0013 mm.	18.0		

* (no specification provided)

Material Description

Brown Sandy CLAY, trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 2.0539 D₈₅= 0.8805 D₆₀= 0.1000
D₅₀= 0.0420 D₃₀= 0.0072 D₁₅=
D₁₀= C_u= C_c=

Remarks

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

Date Received: 09.04.18 Date Tested: 09.07.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-102
Sample Number: 2D

Depth: 5-7'

Date Sampled:

Thielsch Engineering Inc.

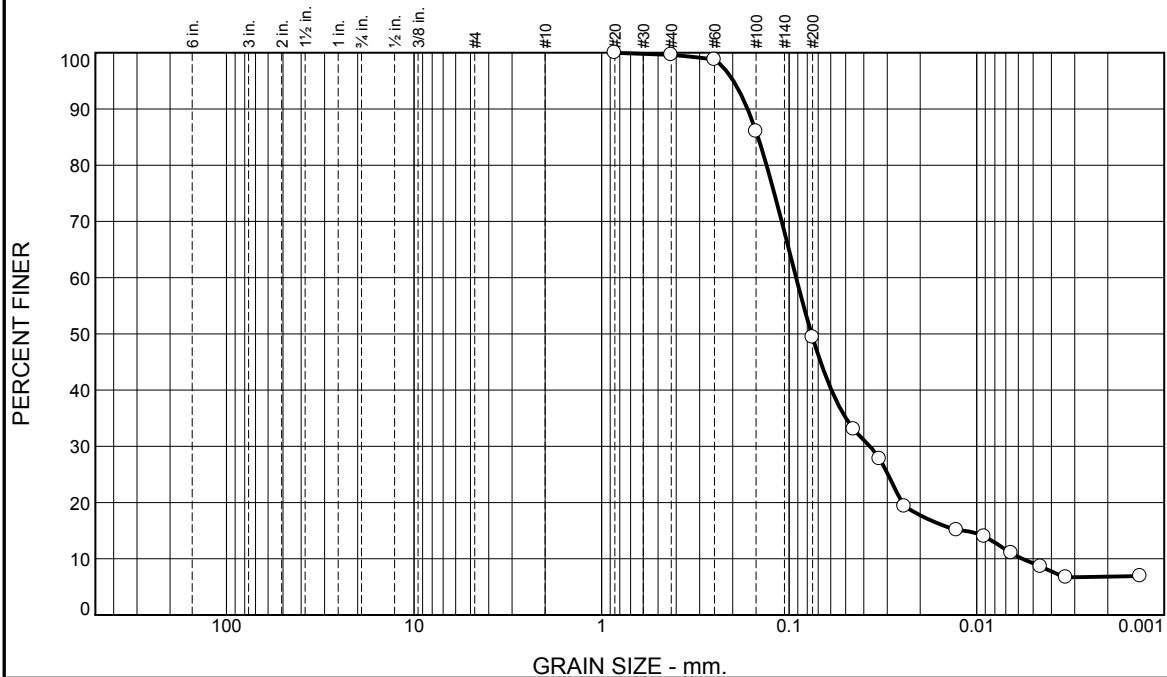
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.4	50.2	40.2	9.2

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#20	100.0		
#40	99.6		
#60	98.8		
#100	86.0		
#200	49.4		
0.0453 mm.	33.1		
0.0330 mm.	27.8		
0.0243 mm.	19.3		
0.0128 mm.	15.1		
0.0091 mm.	14.0		
0.0065 mm.	11.0		
0.0046 mm.	8.6		
0.0033 mm.	6.7		
0.0013 mm.	6.9		

* (no specification provided)

Material Description

Grey - Brown Clayey fine SAND

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 0.1664 D₈₅= 0.1465 D₆₀= 0.0919
D₅₀= 0.0760 D₃₀= 0.0371 D₁₅= 0.0116
D₁₀= 0.0057 C_u= 16.13 C_c= 2.63

Remarks

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

Date Received: 09.04.18 Date Tested: 09.07.18

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-102
Sample Number: 4D

Depth: 15-17'

Date Sampled:

Thielsch Engineering Inc.

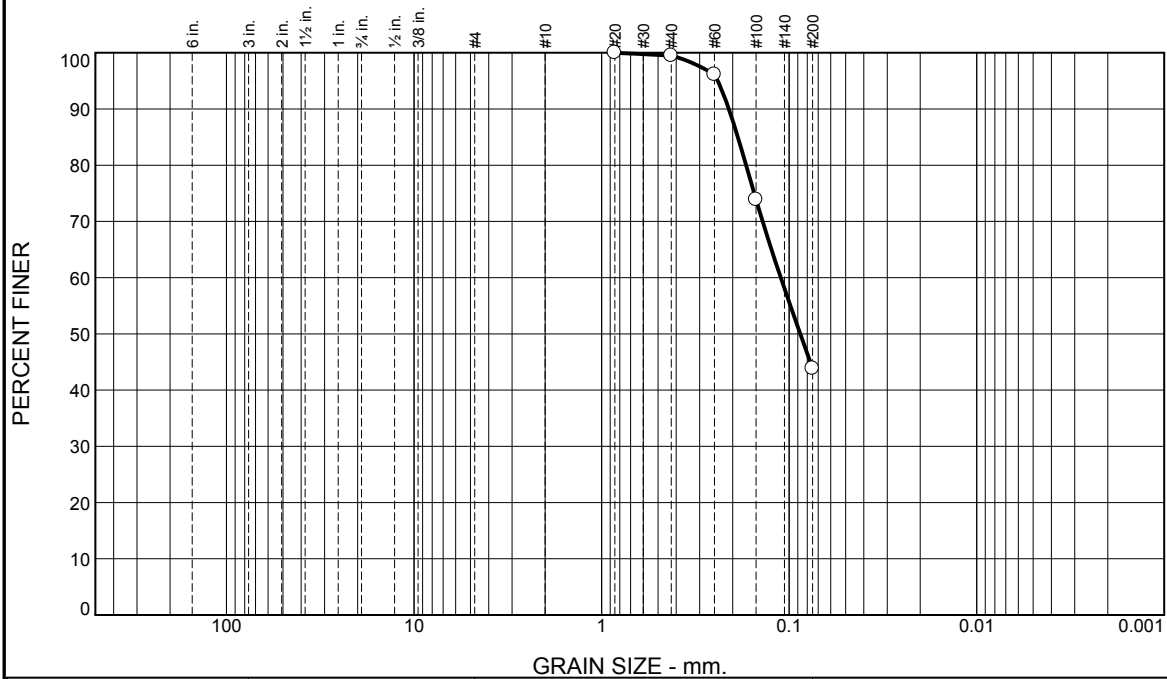
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.5	55.6	43.9	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#20	100.0		
#40	99.5		
#60	96.1		
#100	73.9		
#200	43.9		

* (no specification provided)

Material Description

Grey Clayey fine SAND

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D₉₀= 0.2095 D₈₅= 0.1876 D₆₀= 0.1108
D₅₀= 0.0873 D₃₀= C_u=
D₁₀= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/16".

Date Received: 09.04.18 Date Tested: 09.06.17

Tested By: JD

Checked By: Rebecca Roth

Title: Laboratory Coordinator

Source of Sample: BB-GLR-102
Sample Number: 6D

Depth: 20-22'

Date Sampled:

Thielsch Engineering Inc.

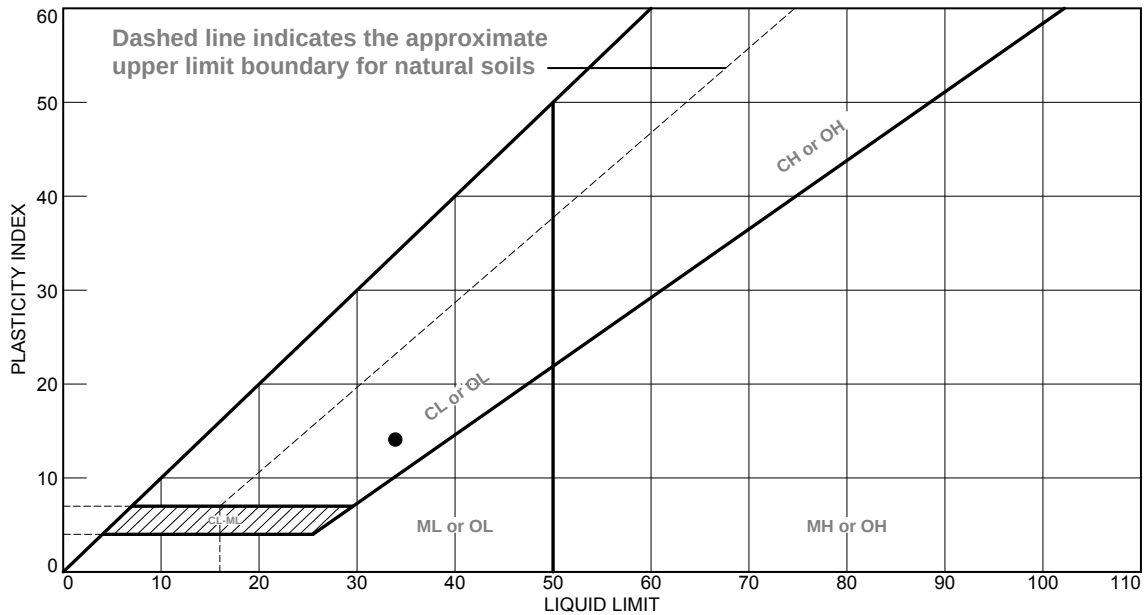
Cranston, RI

Client: GZA GeoEnvironmental
Project: Little River Bridge No 3112
Maine DOT WIN 23909.00

Project No: 09.0025988.00

Figure S-11

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Grey Silty CLAY	34	20	14			

Project No. 09.0025988.00 **Client:** GZA GeoEnvironmental

Project: Little River Bridge No 3112

Maine DOT WIN 23909.00

Source of Sample: BB-GLR-101 **Depth:** 10-12'

Sample Number: 3D

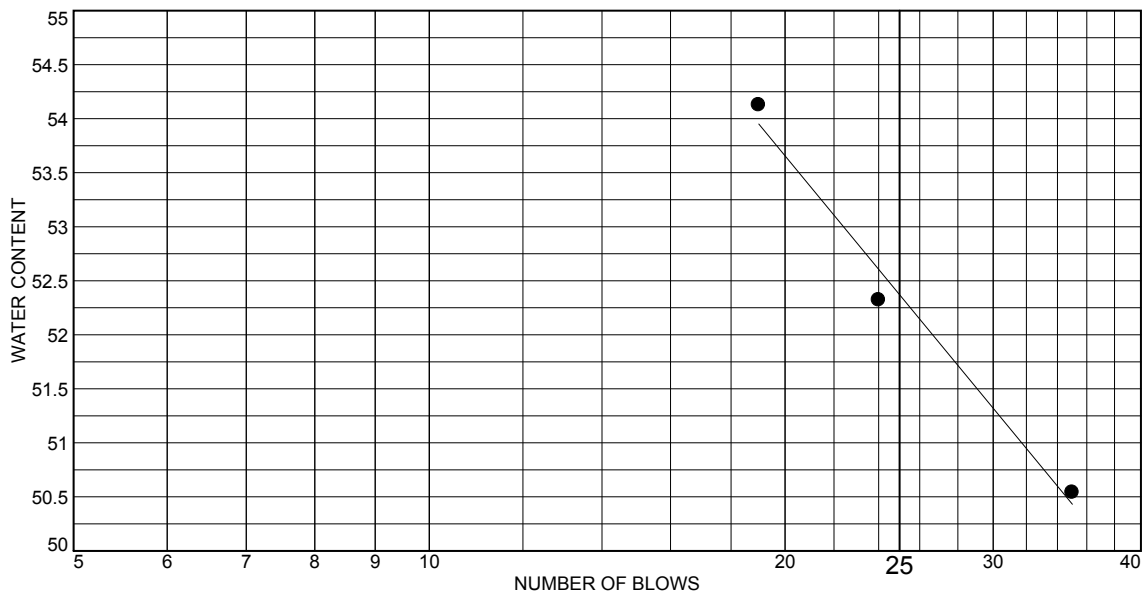
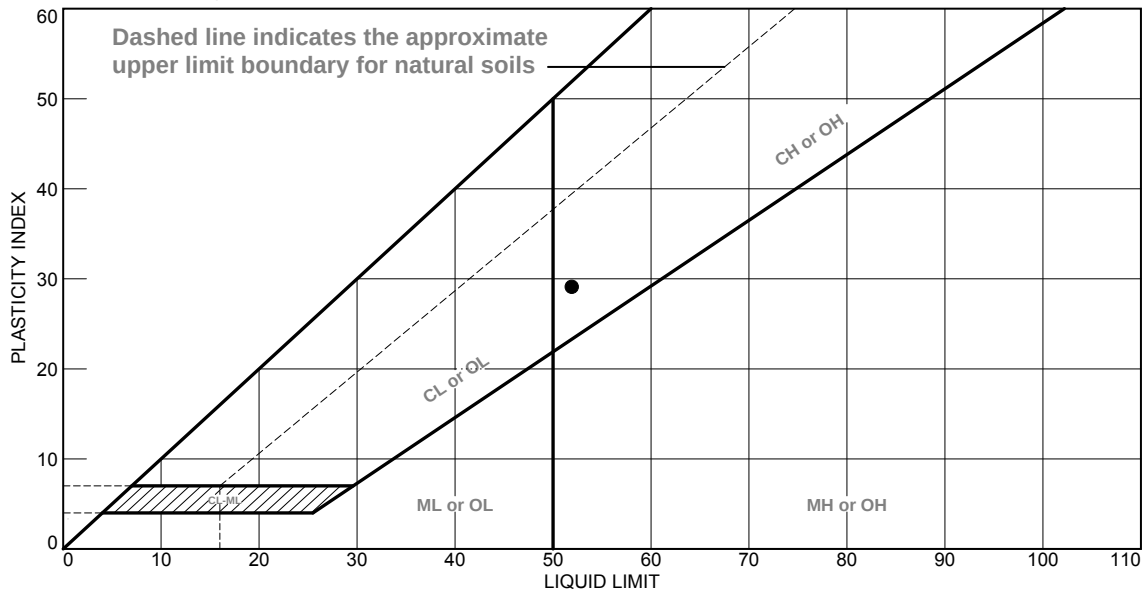
Thielsch Engineering Inc.

Cranston, RI

Remarks:

Figure L-3

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grey Silty CLAY	52	23	29			

Project No. 09.0025988.00 **Client:** GZA GeoEnvironmental
Project: Little River Bridge No 3112
 Maine DOT WIN 23909.00
Source of Sample: BB-GLR-101 **Depth:** 20-22'
Sample Number: 5D

Thielsch Engineering Inc.

Cranston, RI

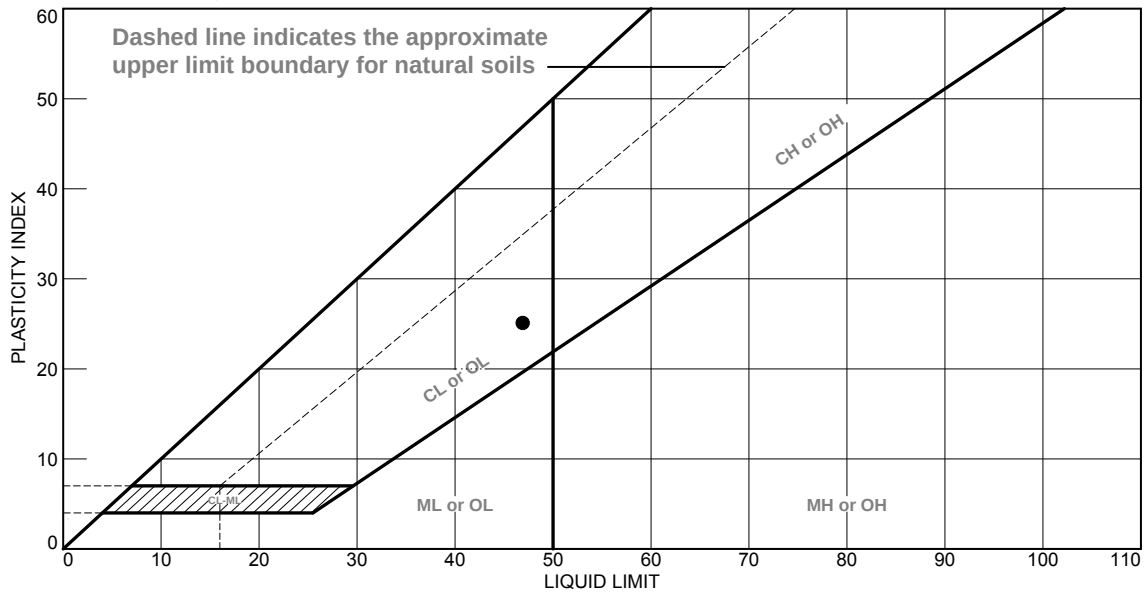
Remarks:

● Sample contained dark grey organic silt pockets

Figure L-5

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

LIQUID AND PLASTIC LIMITS TEST REPORT



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

Project No. 09.0025988.00 **Client:** GZA GeoEnvironmental
Project: Little River Bridge No 3112
 Maine DOT WIN 23909.00
Source of Sample: BB-GLR-101 **Depth:** 27-29'
Sample Number: 6D

Thielsch Engineering Inc.

Cranston, RI

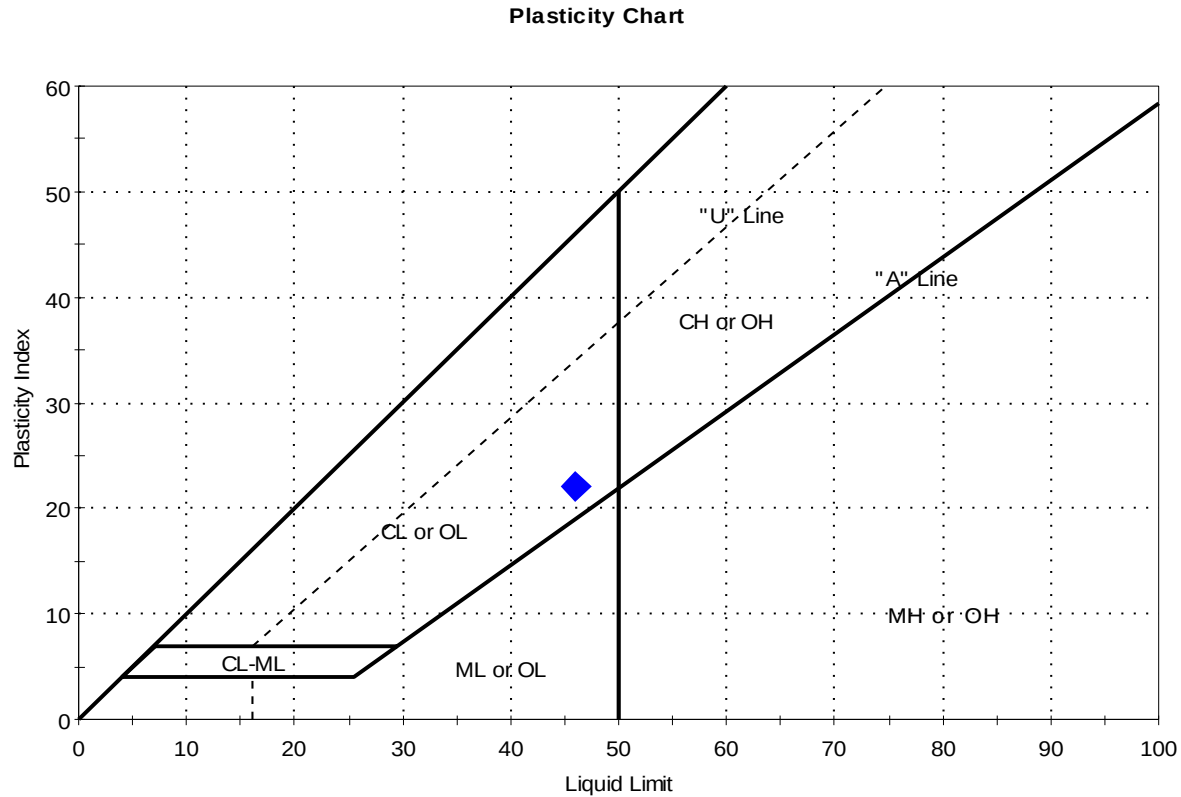
Remarks:

Figure L-6

Tested By: IA Checked By: RR

Client:	GZA GeoEnvironmental, Inc.		
Project:	Little River Bridge No. 3112		
Location:	Gorham, ME	Project No:	GTX-308833
Boring ID:	BB-GLR-101	Sample Type:	tube
Sample ID:	1U	Test Date:	10/01/18
Depth :	25-27	Test Id:	474295
Test Comment:	---		
Visual Description:	Moist, dark gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-GLR-10	25-27	45	46	24	22	1	

Sample Prepared using the WET method

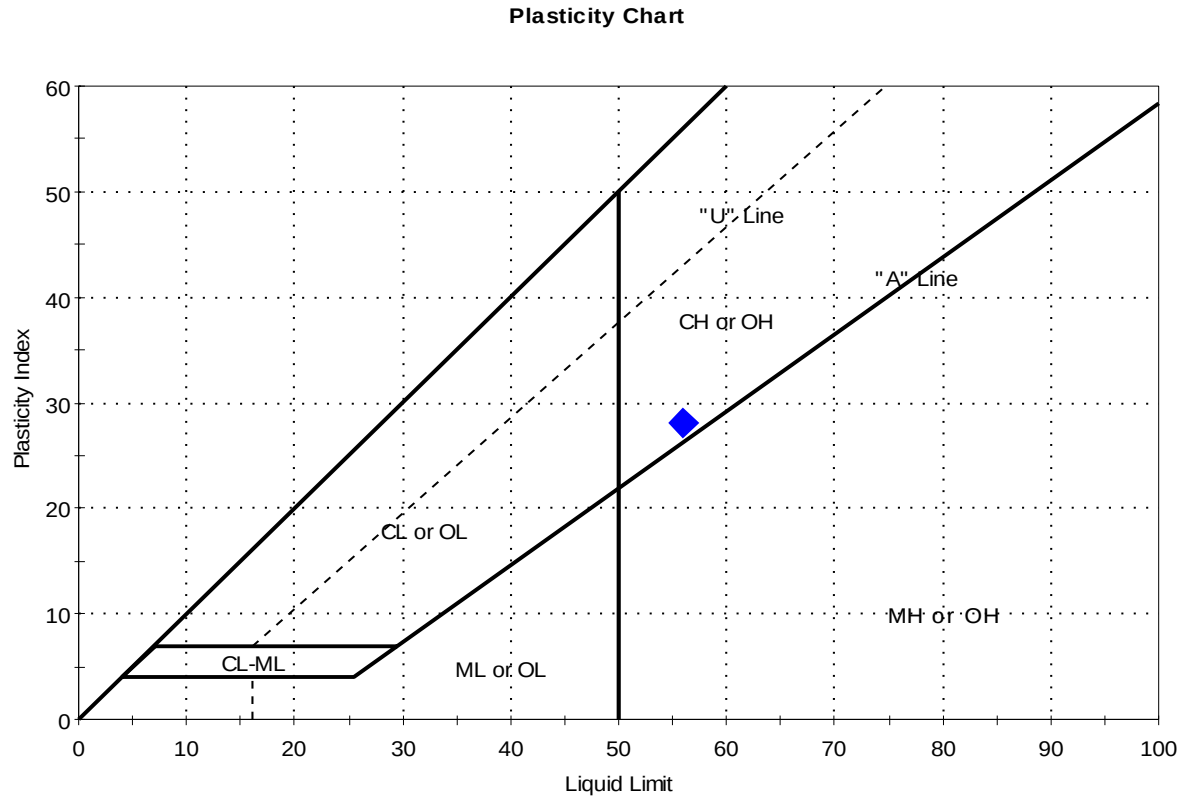
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	GZA GeoEnvironmental, Inc.		
Project:	Little River Bridge No. 3112		
Location:	Gorham, ME	Project No:	GTX-308833
Boring ID:	BB-GLR-102	Sample Type:	tube
Sample ID:	1U	Test Date:	10/01/18
Depth :	30-32	Test Id:	474296
Test Comment:	---		
Visual Description:	Moist, dark gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-GLR-10	30-32	59	56	28	28	1.1	

Sample Prepared using the WET method

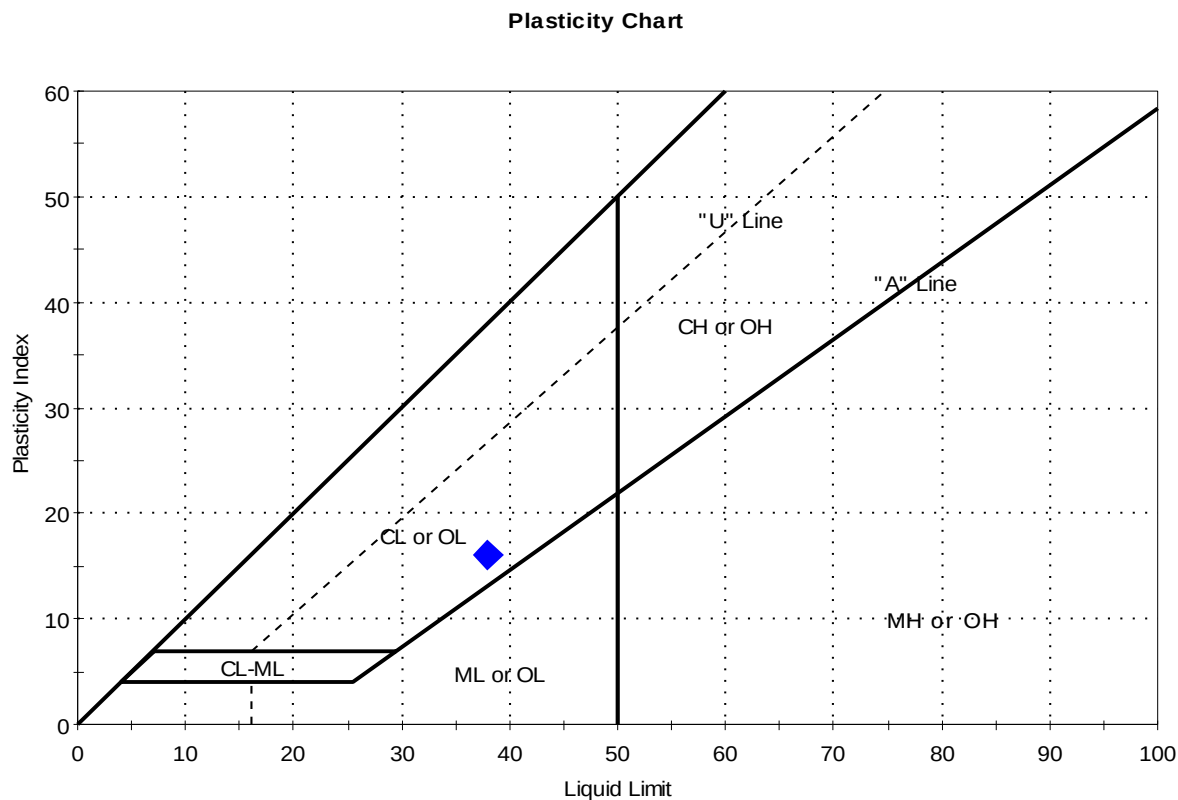
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	GZA GeoEnvironmental, Inc.		
Project:	Little River Bridge No. 3112		
Location:	Gorham, ME	Project No:	GTX-308833
Boring ID:	BB-GLR-102	Sample Type:	tube
Sample ID:	2U	Test Date:	10/02/18
Depth :	40-42	Test Id:	474297
Test Comment:	---		
Visual Description:	Wet, very dark greenish gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2U	B-GLR-10	40-42	43	38	22	16	1.3	

Sample Prepared using the WET method

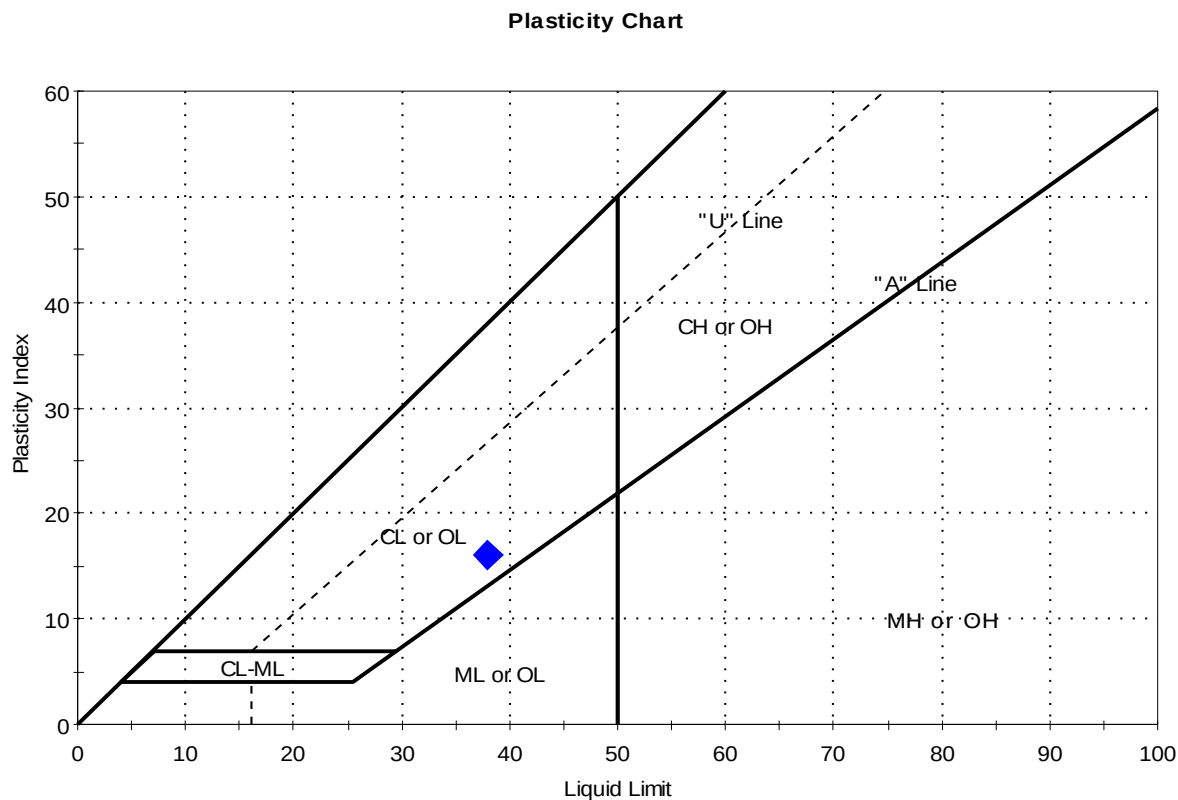
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	GZA GeoEnvironmental, Inc.		
Project:	Little River Bridge No. 3112		
Location:	Gorham, ME	Project No:	GTX-308833
Boring ID:	BB-GLR-102	Sample Type:	tube
Sample ID:	3U	Test Date:	10/02/18
Depth :	50-52	Test Id:	474298
Test Comment:	---		
Visual Description:	Moist, dark greenish gray clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3U	B-GLR-10	50-52	39	38	22	16	1	

Sample Prepared using the WET method

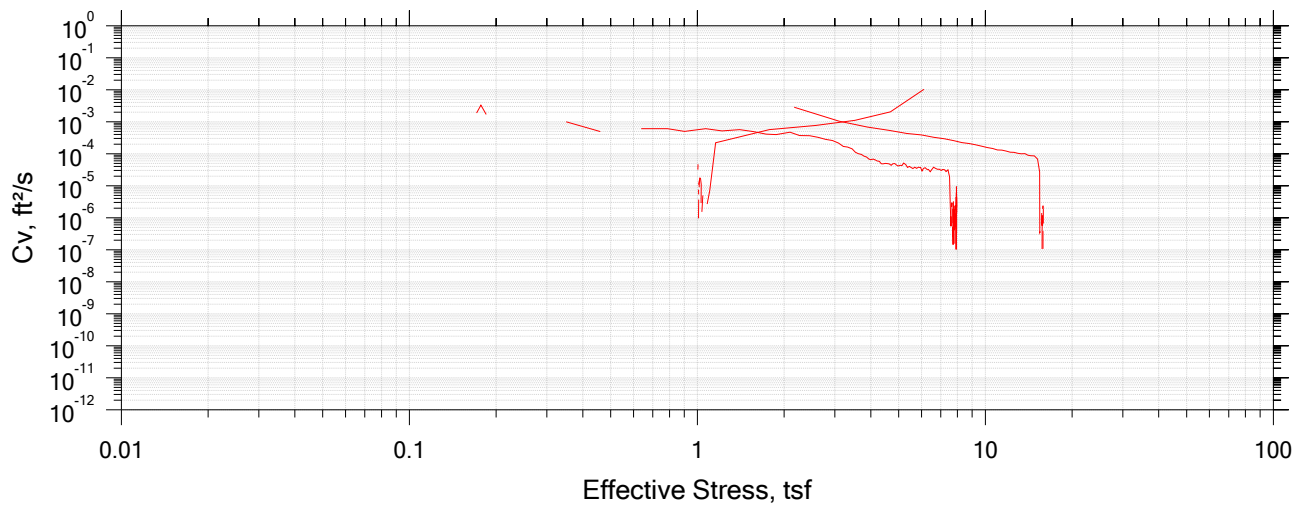
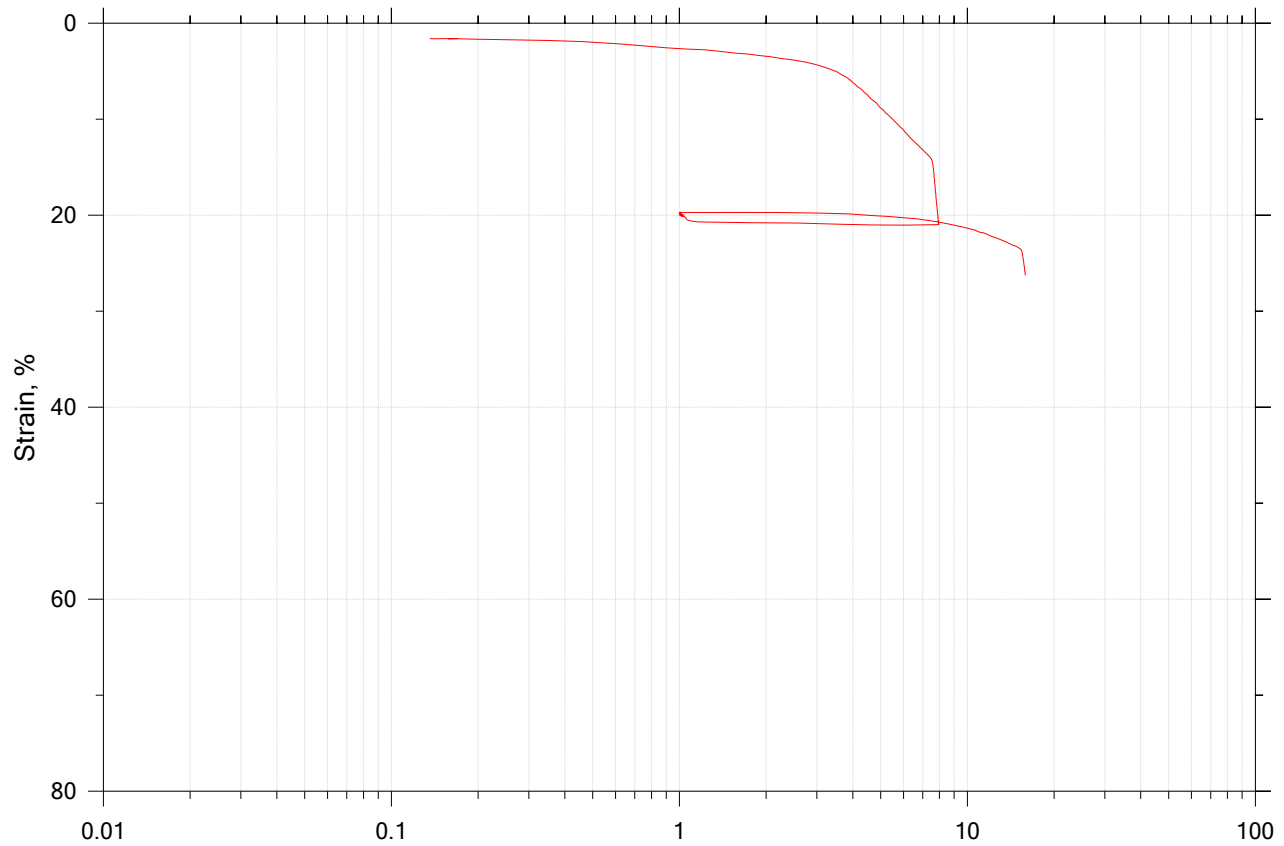
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

CRC TEST

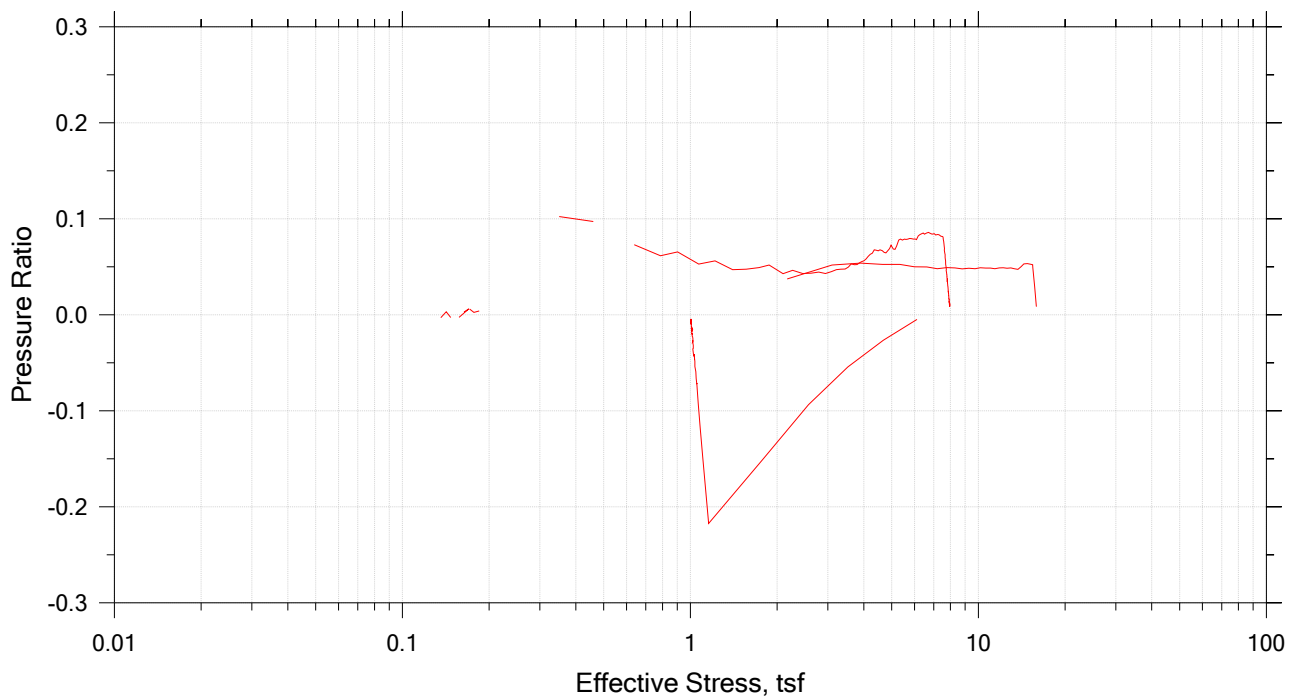
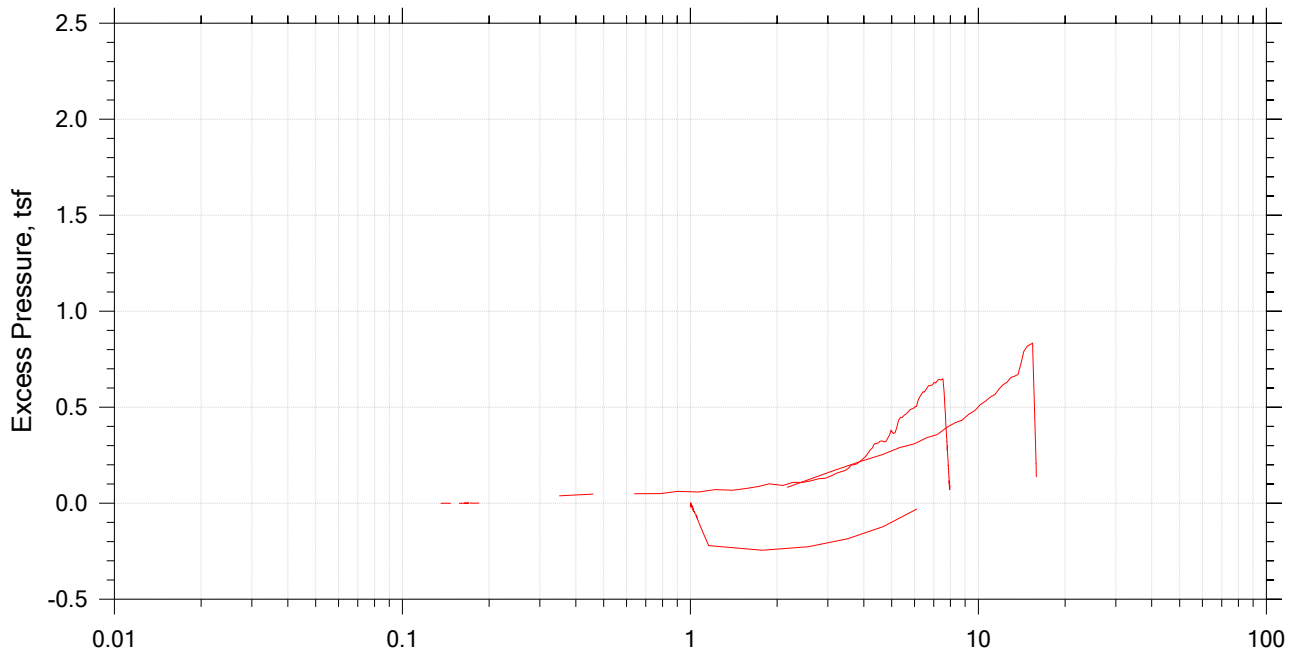
Summary Curves



	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 09/24/18	Depth: 30-32 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System W		

CRC TEST

Pressure Curves



	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 09/24/18	Depth: 30-32 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System W		

CRC TEST

Specimen Diameter: 2.50 in	Estimated: 2.74	Liquid Limit: 56
Initial Height: 1.00 in	Initial Void Ratio: 1.54	Plastic Limit: 28
Final Height: 0.77 in	Final Void Ratio: 0.956	Plasticity Index: 28

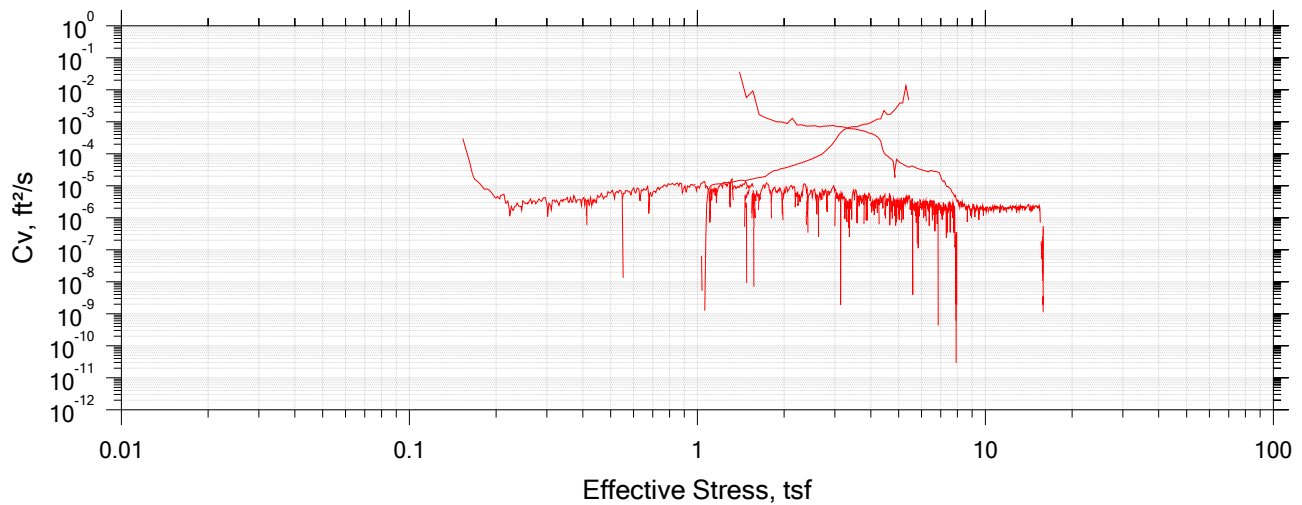
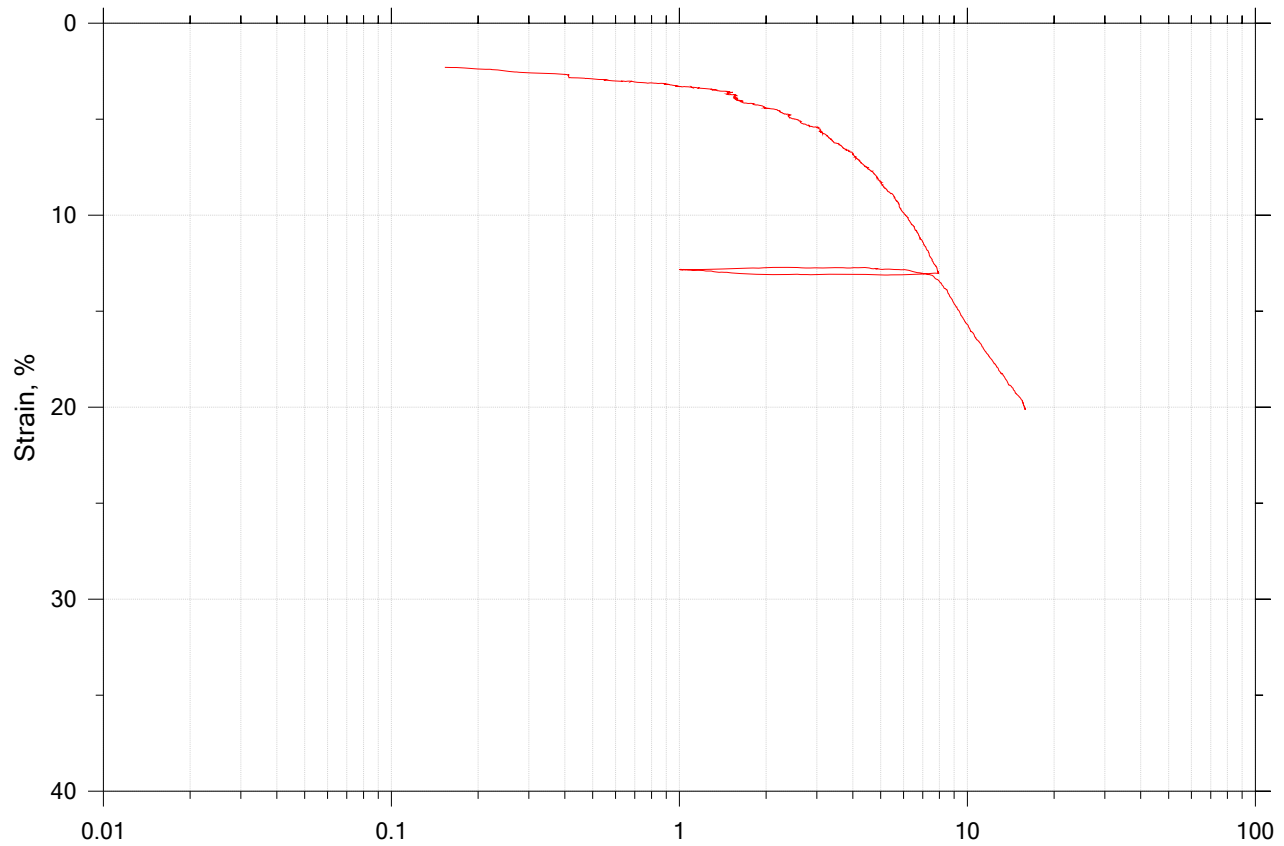
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	D-1589	RING		A-2414
Mass Container, gm	8.28	109.51	109.51	8.09
Mass Container + Wet Soil, gm	205.92	244.86	226.56	126.24
Mass Container + Dry Soil, gm	132.97	196.28	196.28	95.68
Mass Dry Soil, gm	124.69	86.775	86.775	87.59
Water Content, %	58.51	55.98	34.89	34.89
Void Ratio	---	1.54	0.96	---
Degree of Saturation, %	---	99.59	100.00	---
Dry Unit Weight, pcf	---	67.344	87.46	---

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

	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 09/24/18	Depth: 30-32 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System W		

CRC TEST

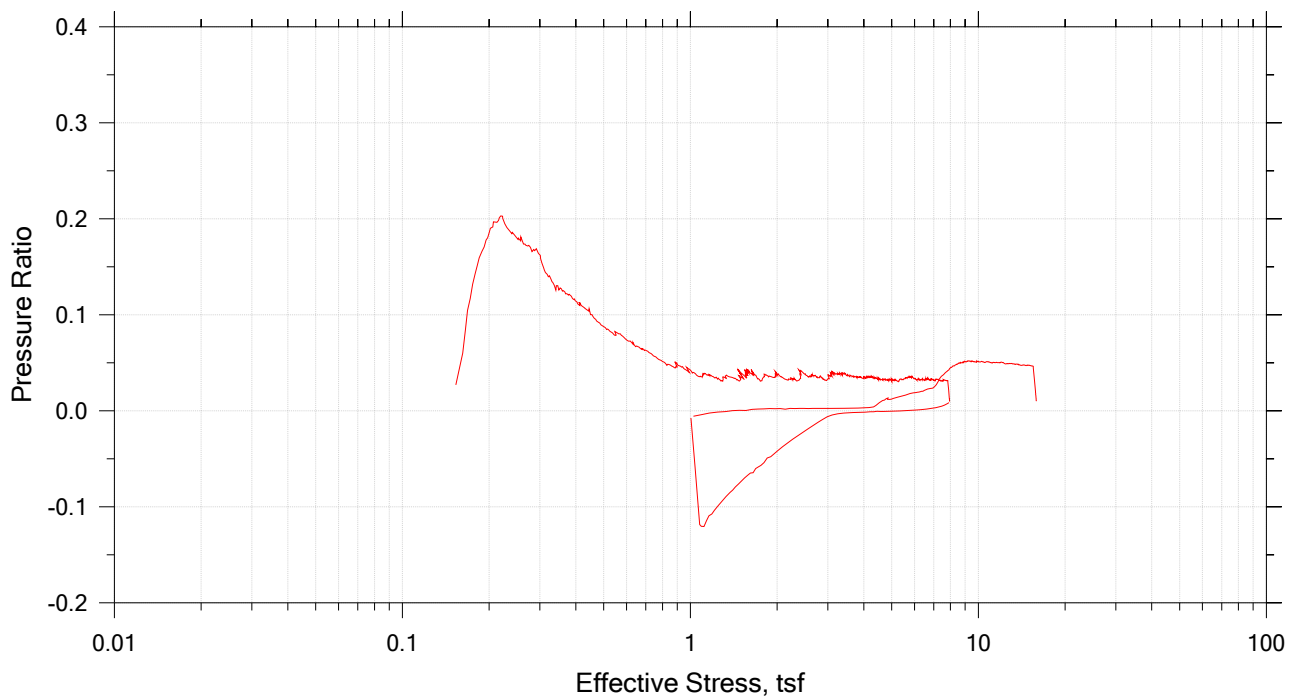
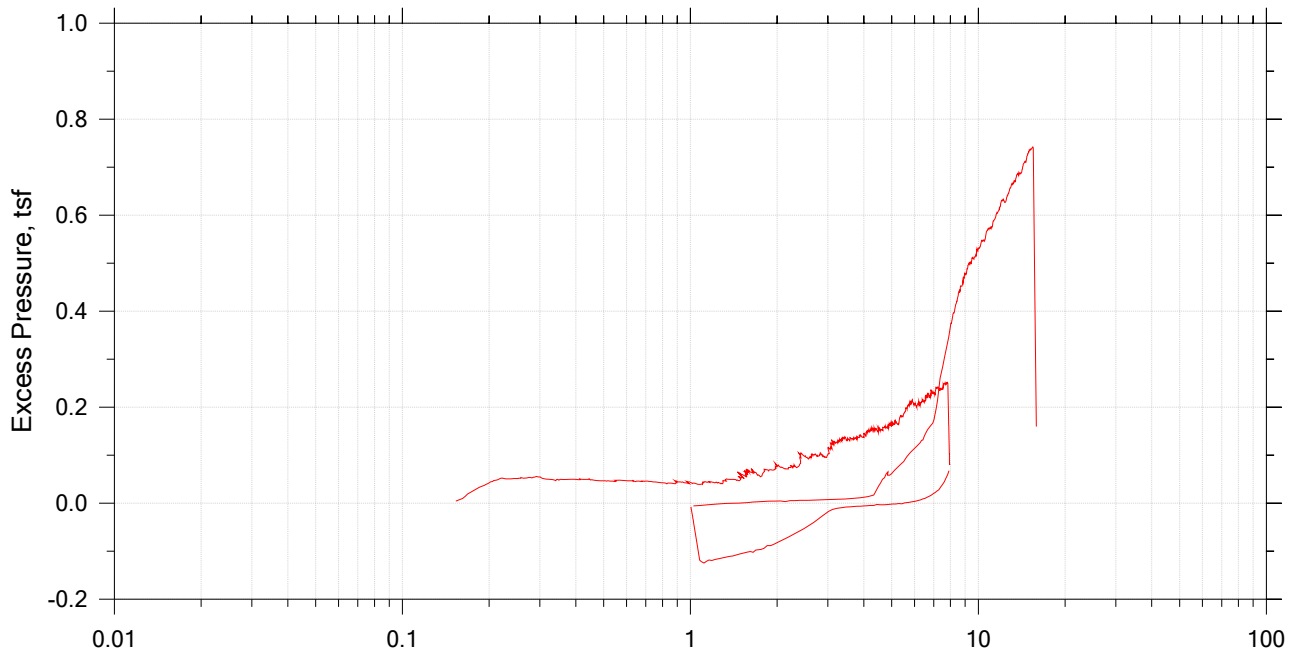
Summary Curves



	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 3U	Test Date: 09/24/18	Depth: 50-52 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, dark greenish gray clay		
	Remarks: System T		

CRC TEST

Pressure Curves



	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 3U	Test Date: 09/24/18	Depth: 50-52 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, dark greenish gray clay		
	Remarks: System T		

CRC TEST

Specimen Diameter: 2.50 in	Estimated: 2.74	Liquid Limit: Unknown
Initial Height: 1.00 in	Initial Void Ratio: 1.07	Plastic Limit: Unknown
Final Height: 0.80 in	Final Void Ratio: 0.656	Plasticity Index: Unknown

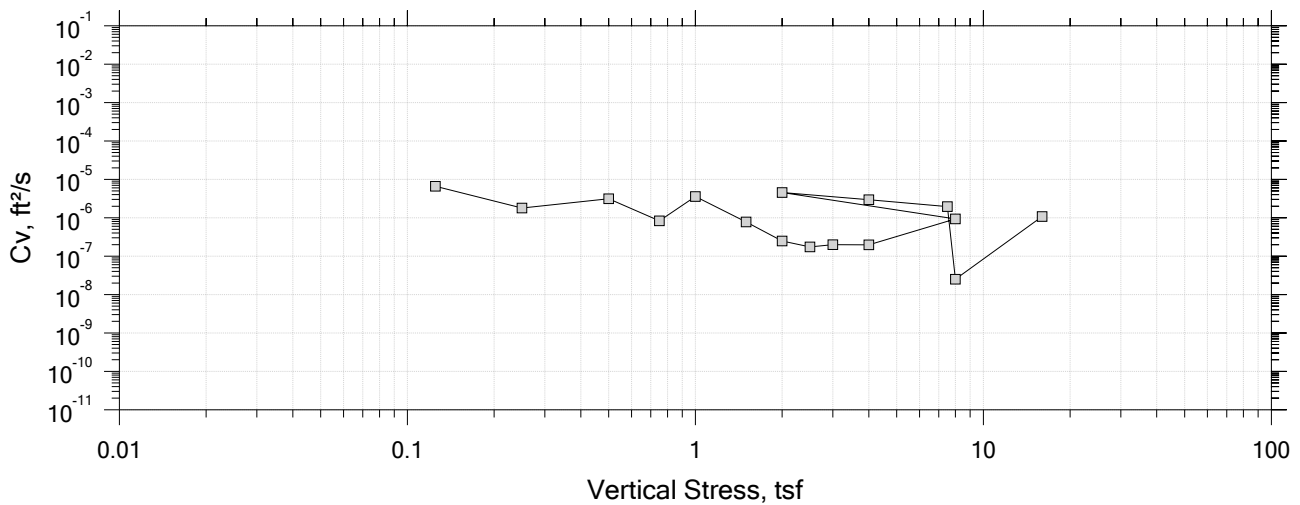
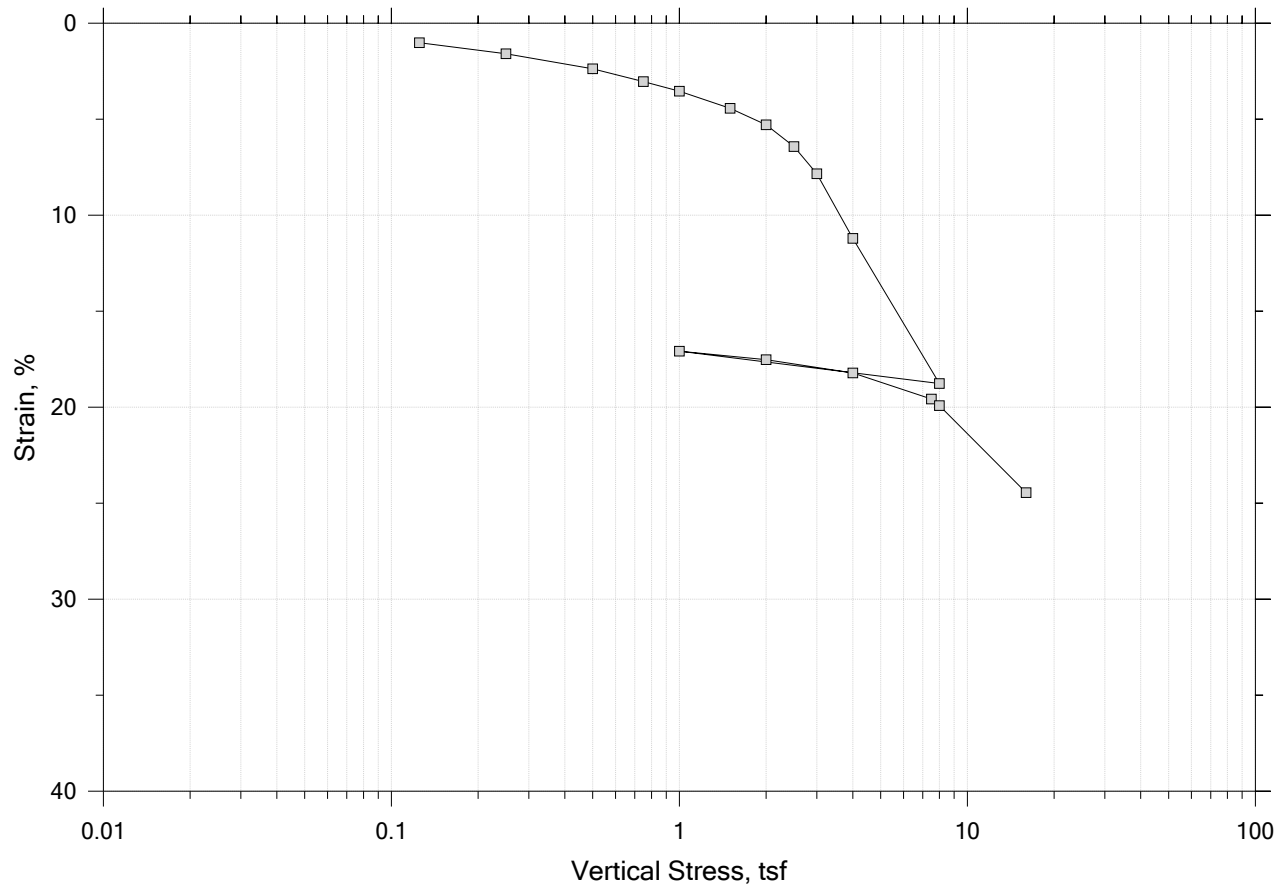
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-1598	RING		A-2299
Mass Container, gm	8.31	111.2	111.2	8.34
Mass Container + Wet Soil, gm	202.89	258.78	243.27	143.11
Mass Container + Dry Soil, gm	148.64	217.78	217.78	117.1
Mass Dry Soil, gm	140.33	106.58	106.58	108.76
Water Content, %	38.66	38.47	23.92	23.92
Void Ratio	---	1.07	0.66	---
Degree of Saturation, %	---	98.61	100.00	---
Dry Unit Weight, pcf	---	82.716	103.39	---

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 spec.

	Project Name: Little River Bridge No. 3112	Location: Gorham, ME	Project Number: GTX-308833
	Boring Number: BB-GLR-102	Tester: md	Checker: njh
	Sample Number: 3U	Test Date: 09/24/18	Depth: 50-52 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, dark greenish gray clay		
	Remarks: System T		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report



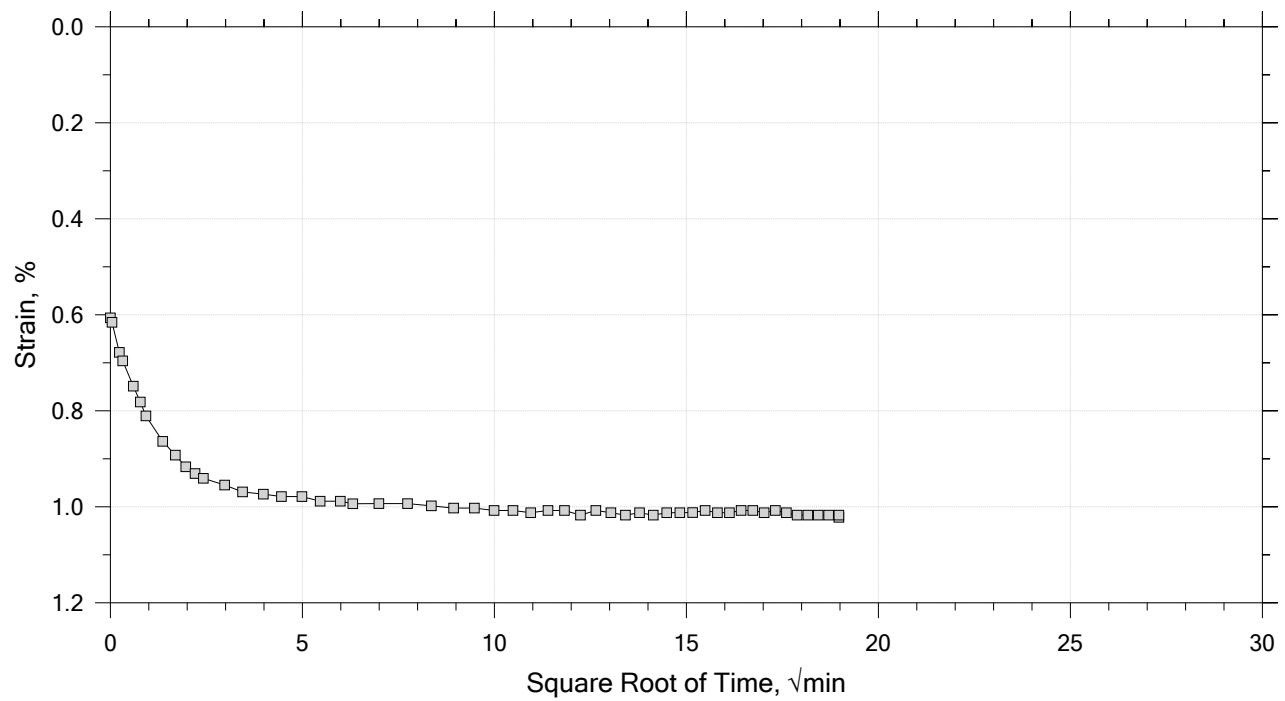
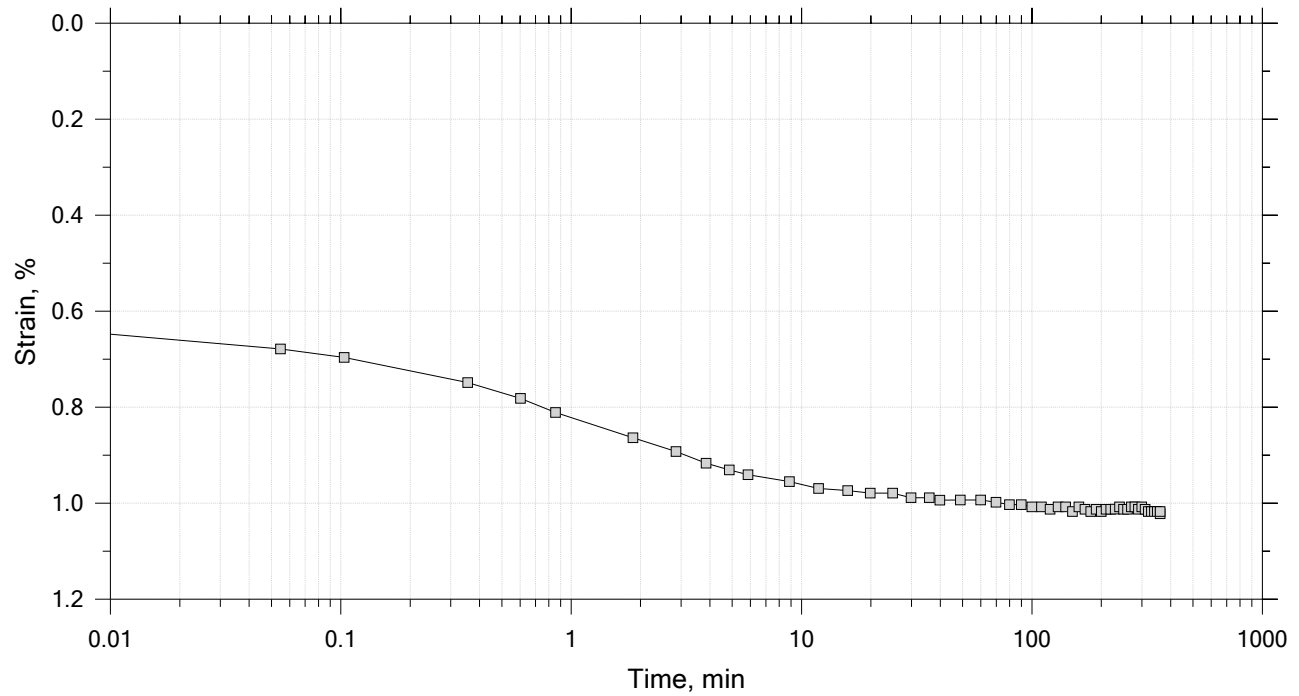
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 17

Constant Load Step

Stress: 0.125 tsf



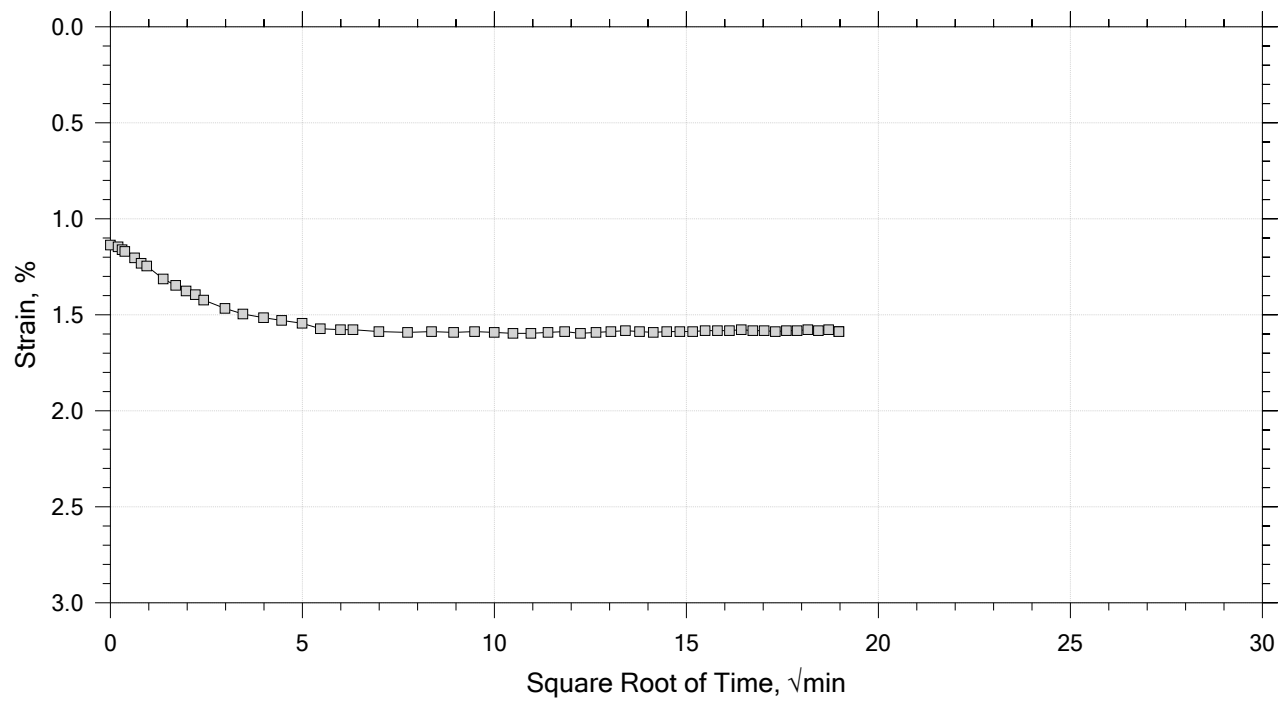
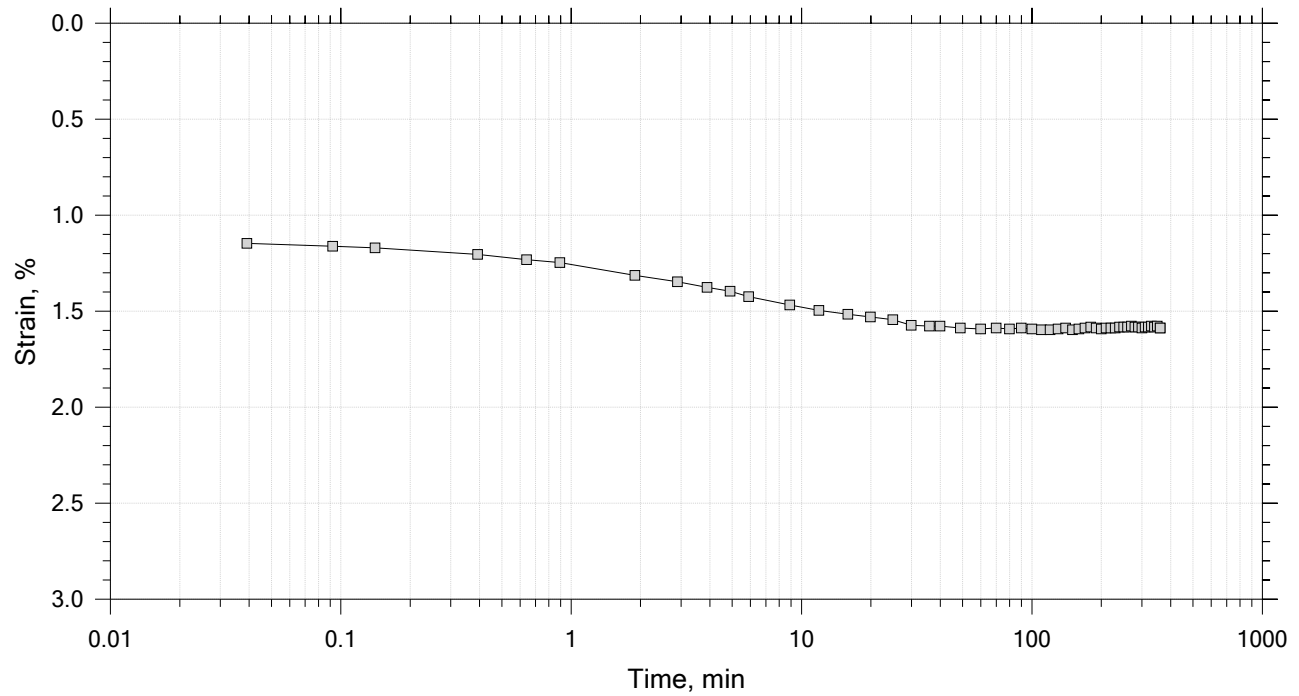
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 17

Constant Load Step

Stress: 0.25 tsf



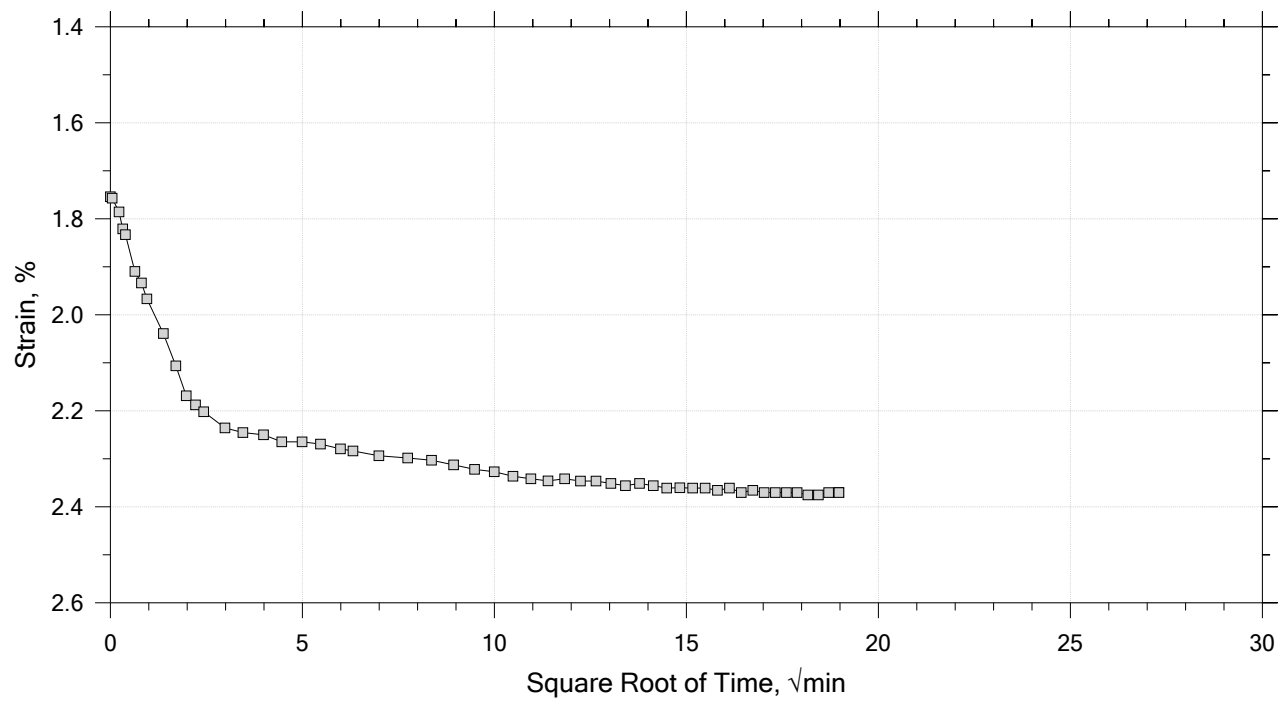
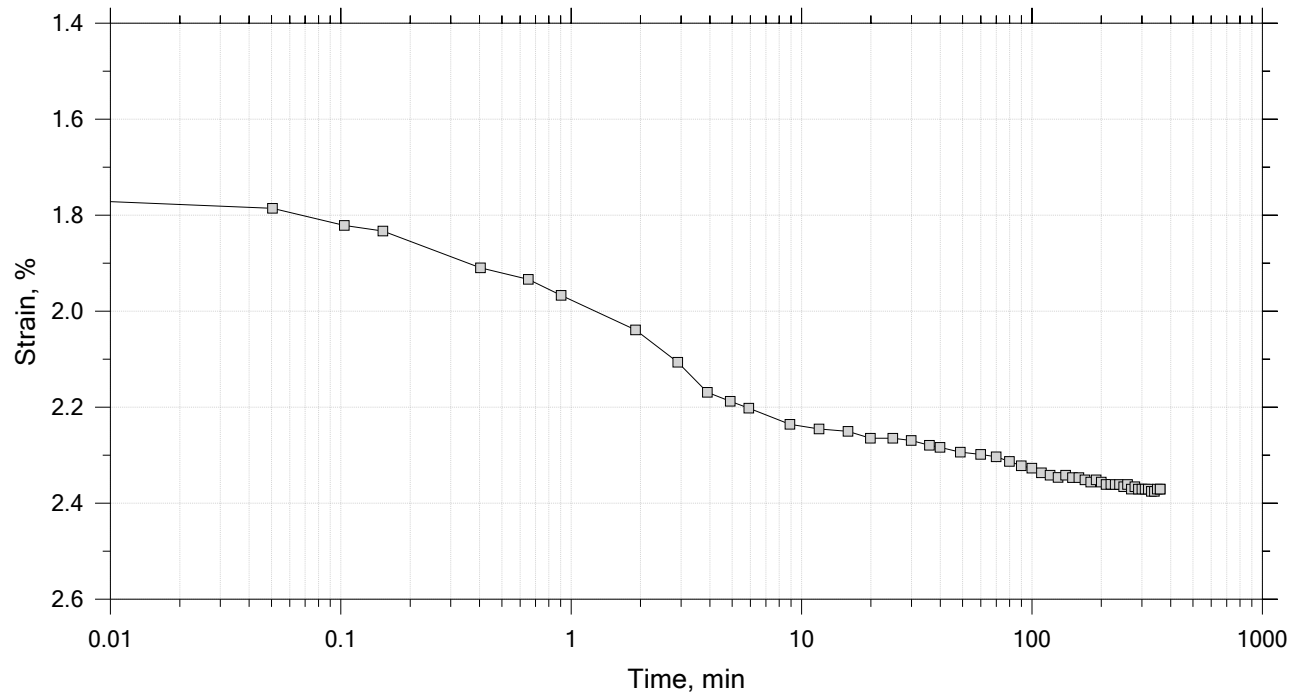
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 17

Constant Load Step

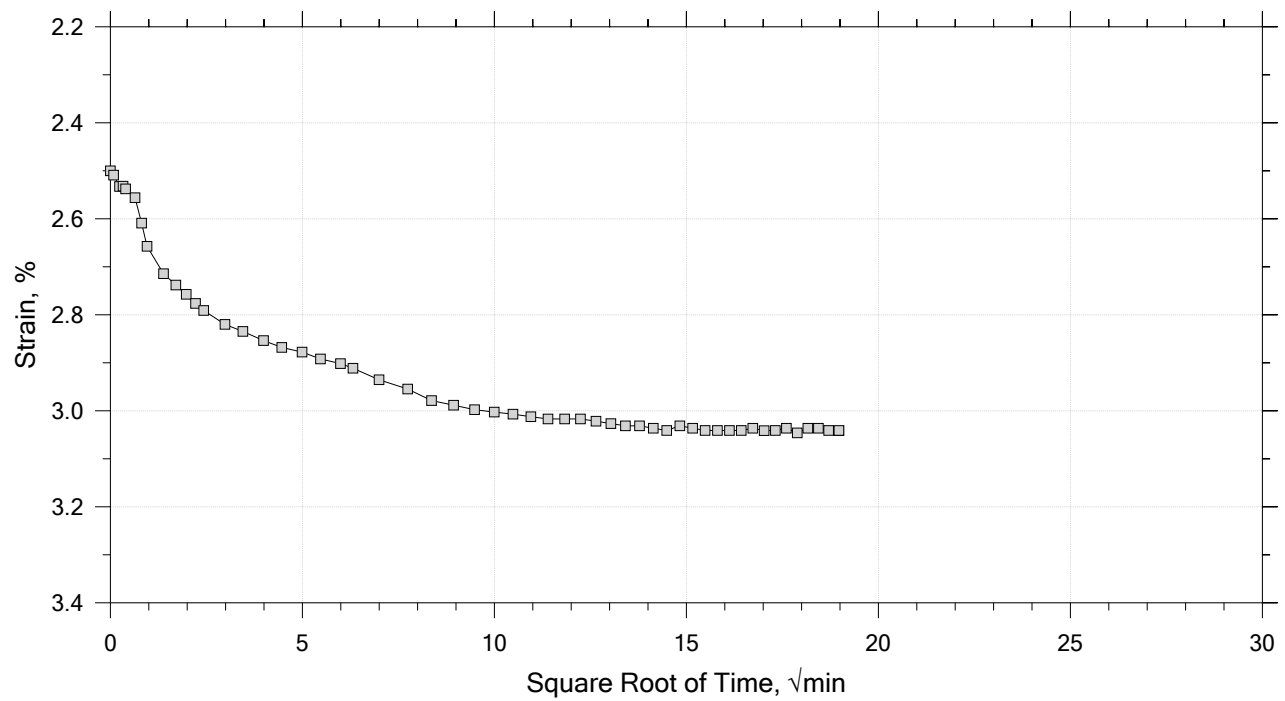
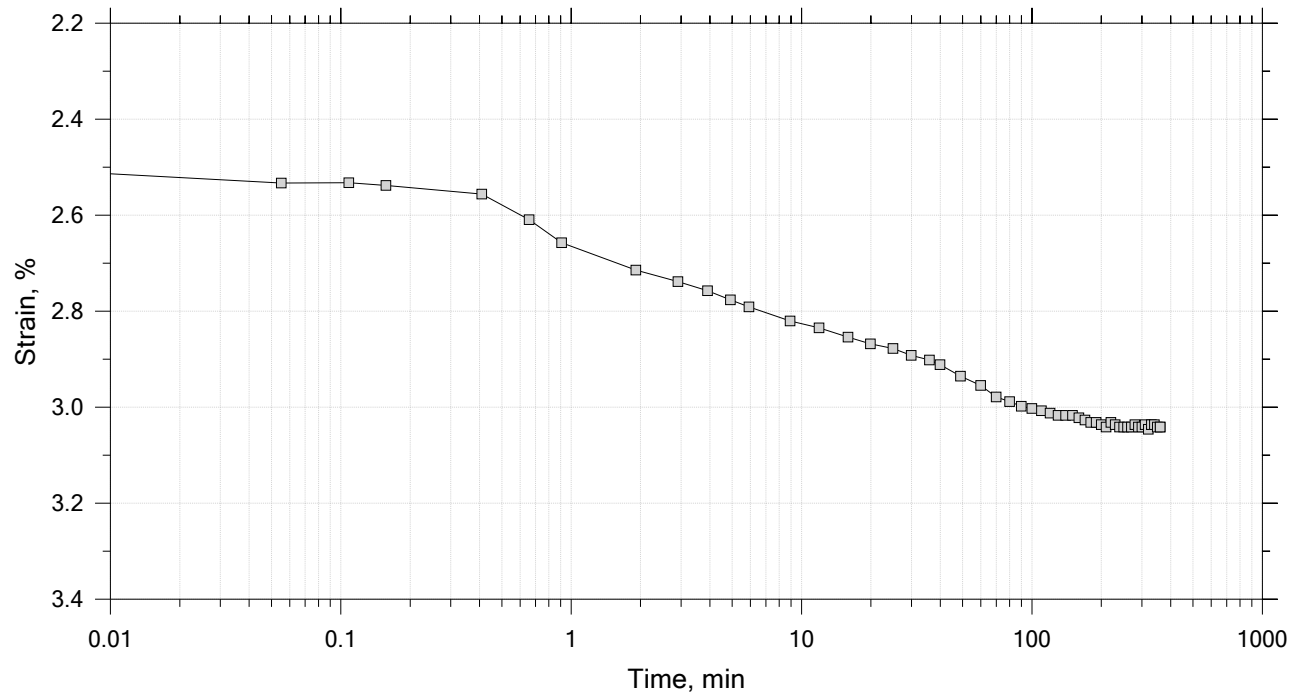
Stress: 0.5 tsf



	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 17
Constant Load Step
Stress: 0.75 tsf



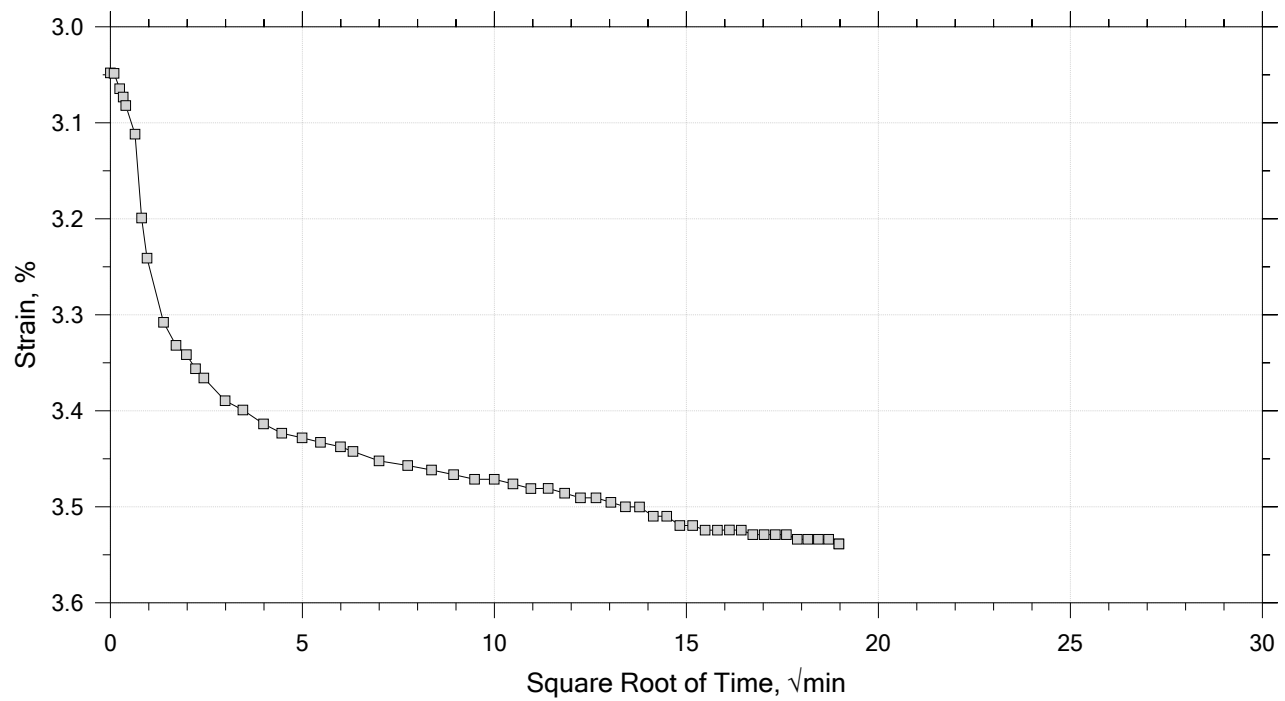
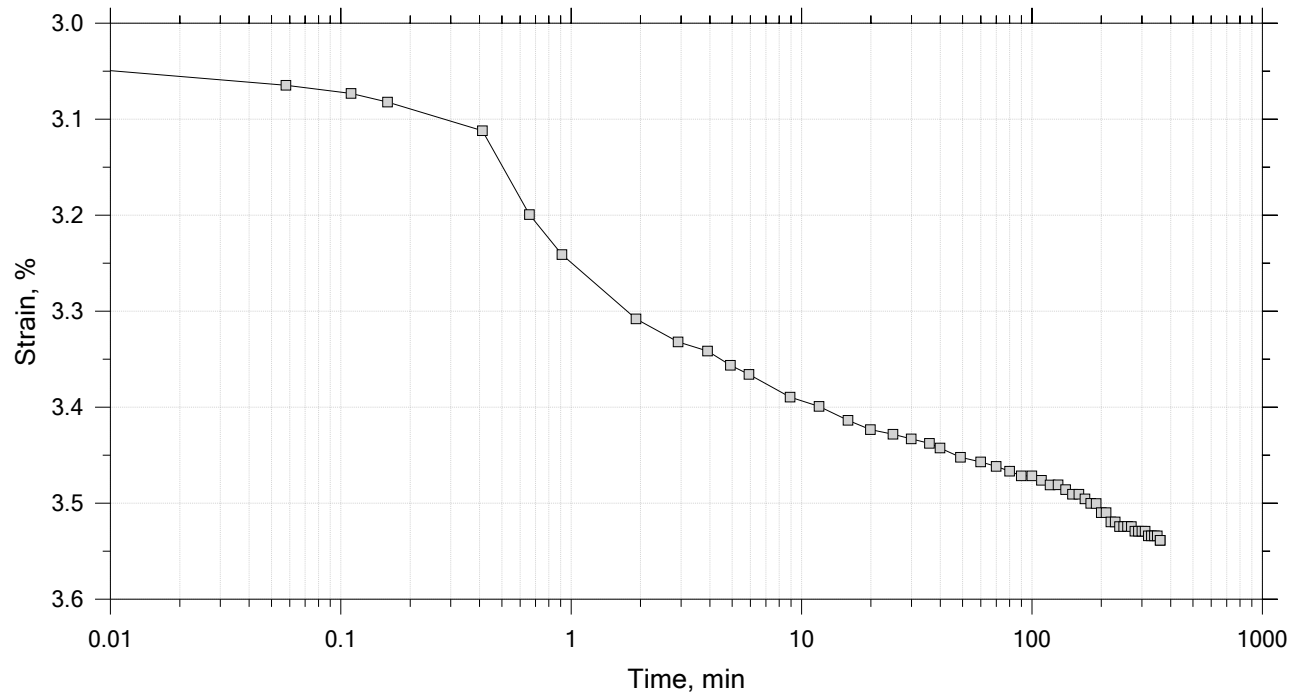
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 17

Constant Load Step

Stress: 1 tsf



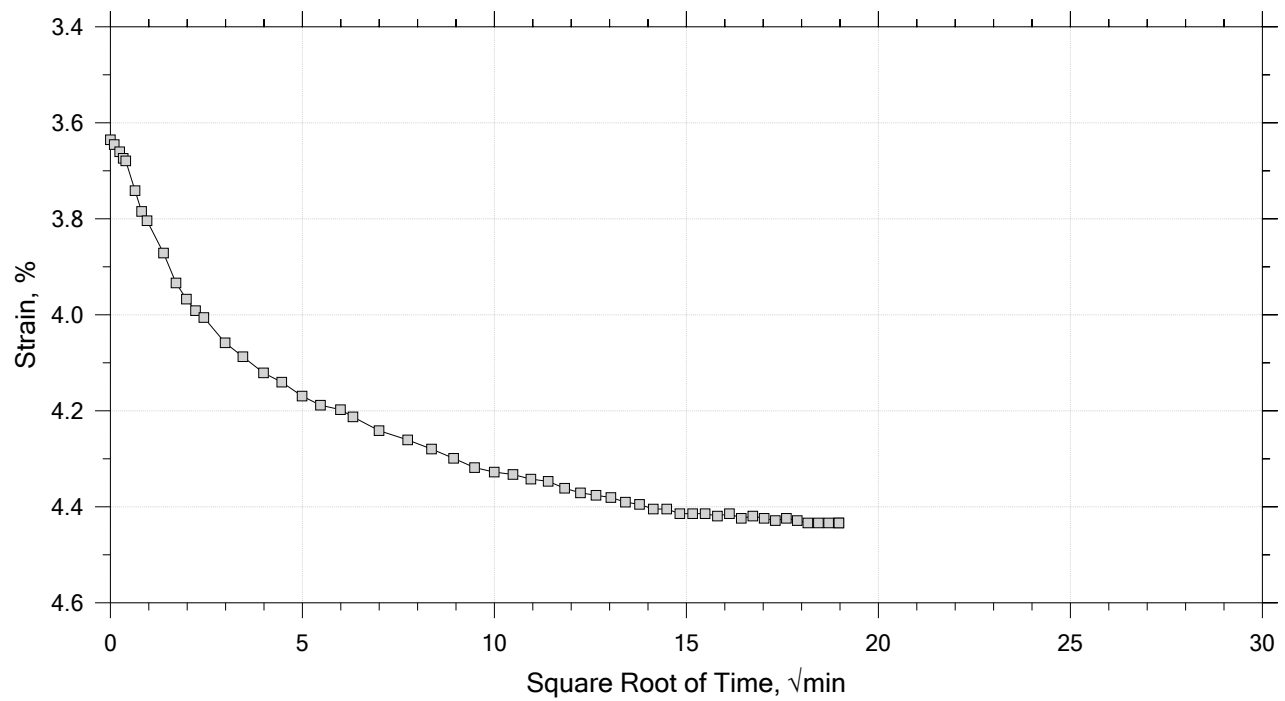
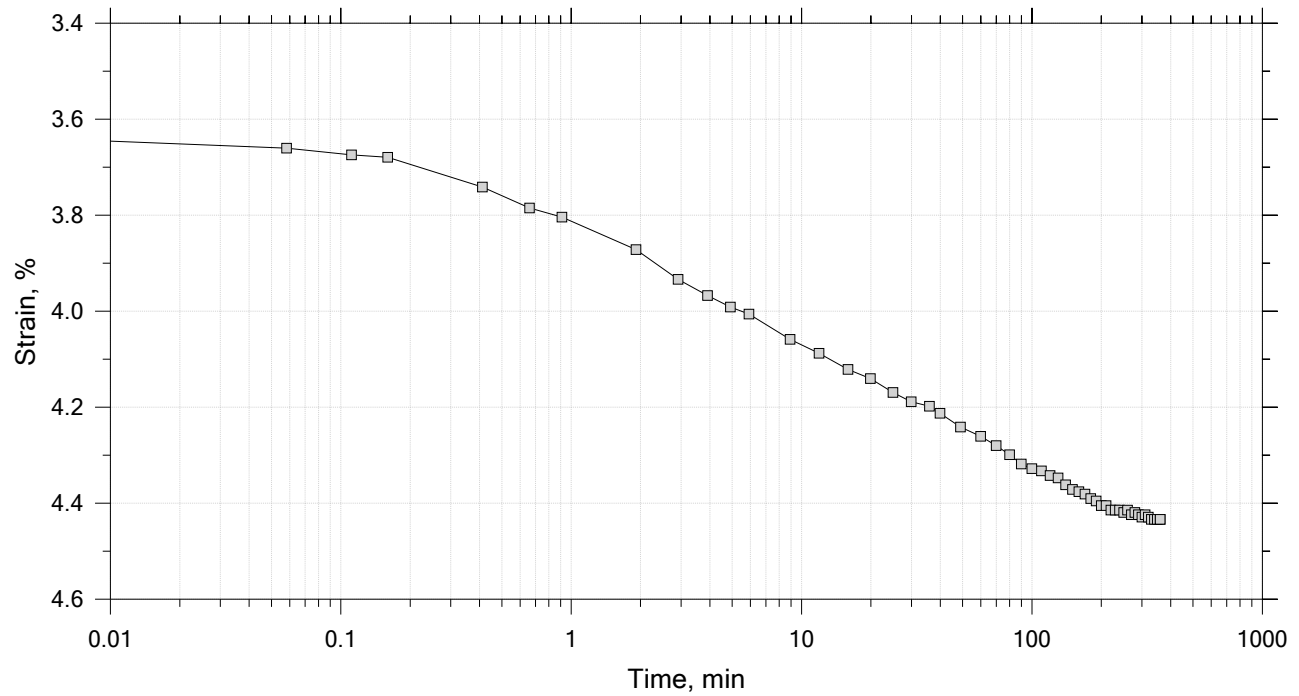
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 17

Constant Load Step

Stress: 1.5 tsf



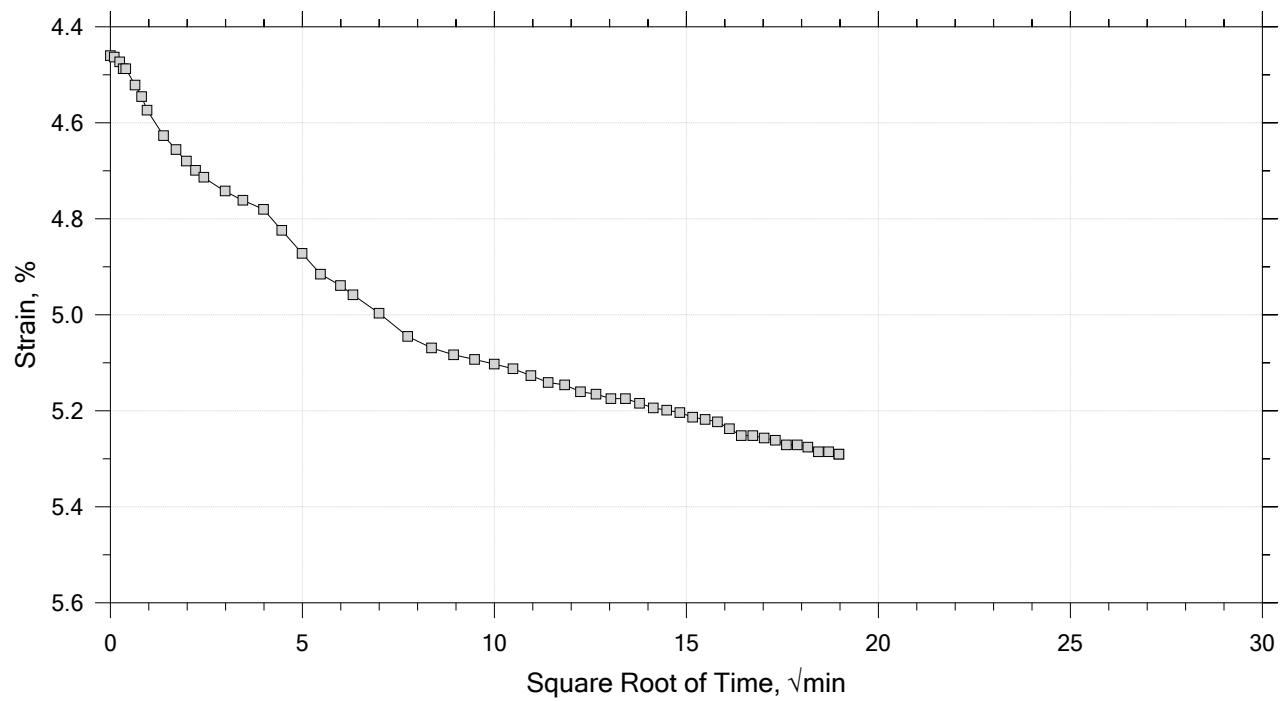
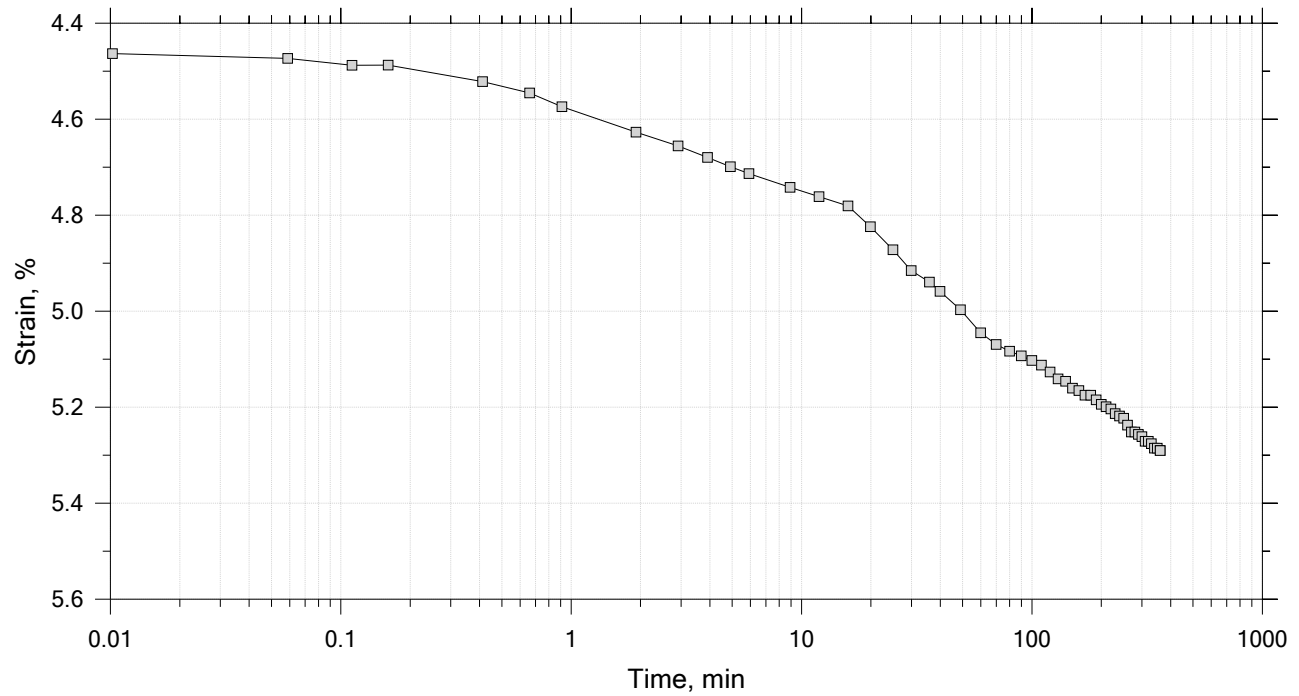
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 17

Constant Load Step

Stress: 2 tsf



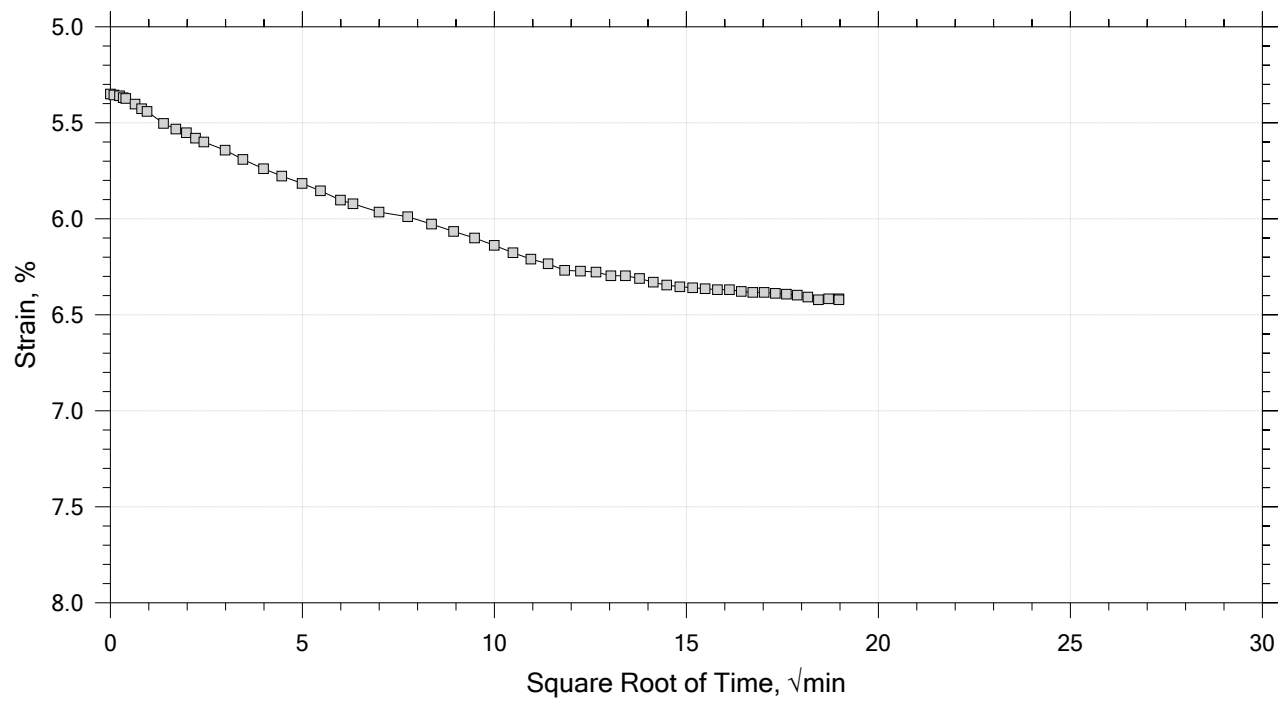
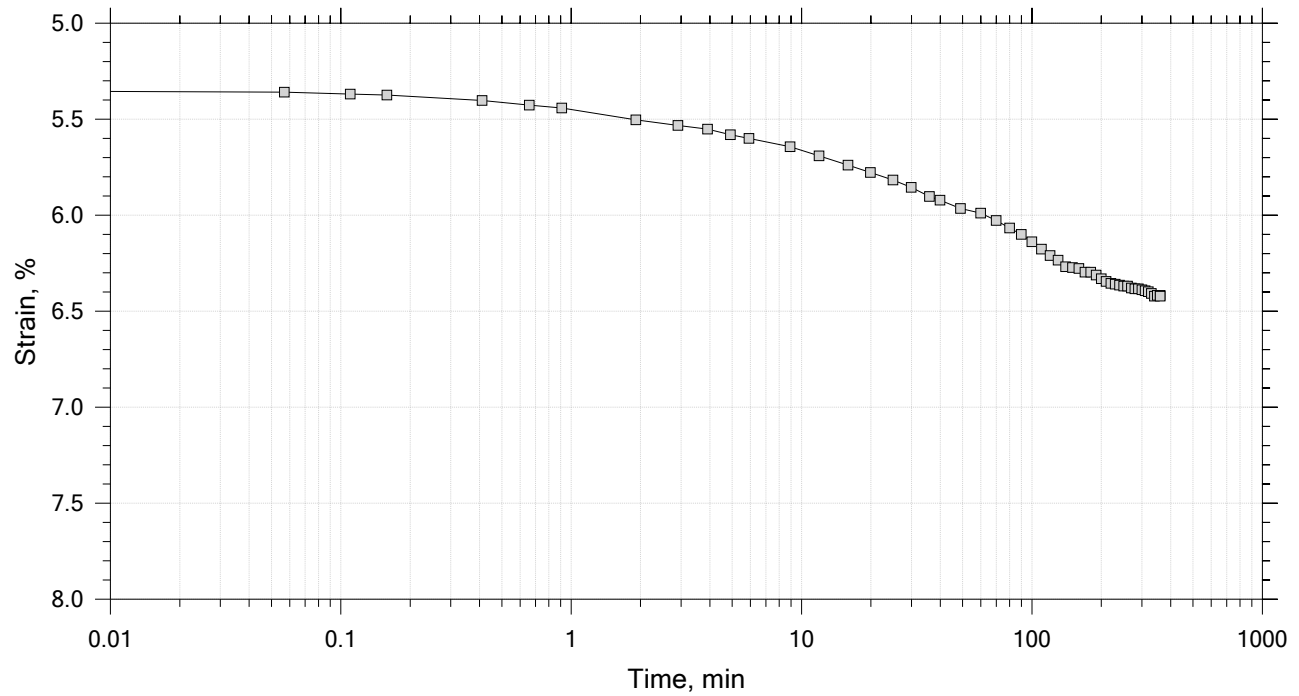
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 17

Constant Load Step

Stress: 2.5 tsf



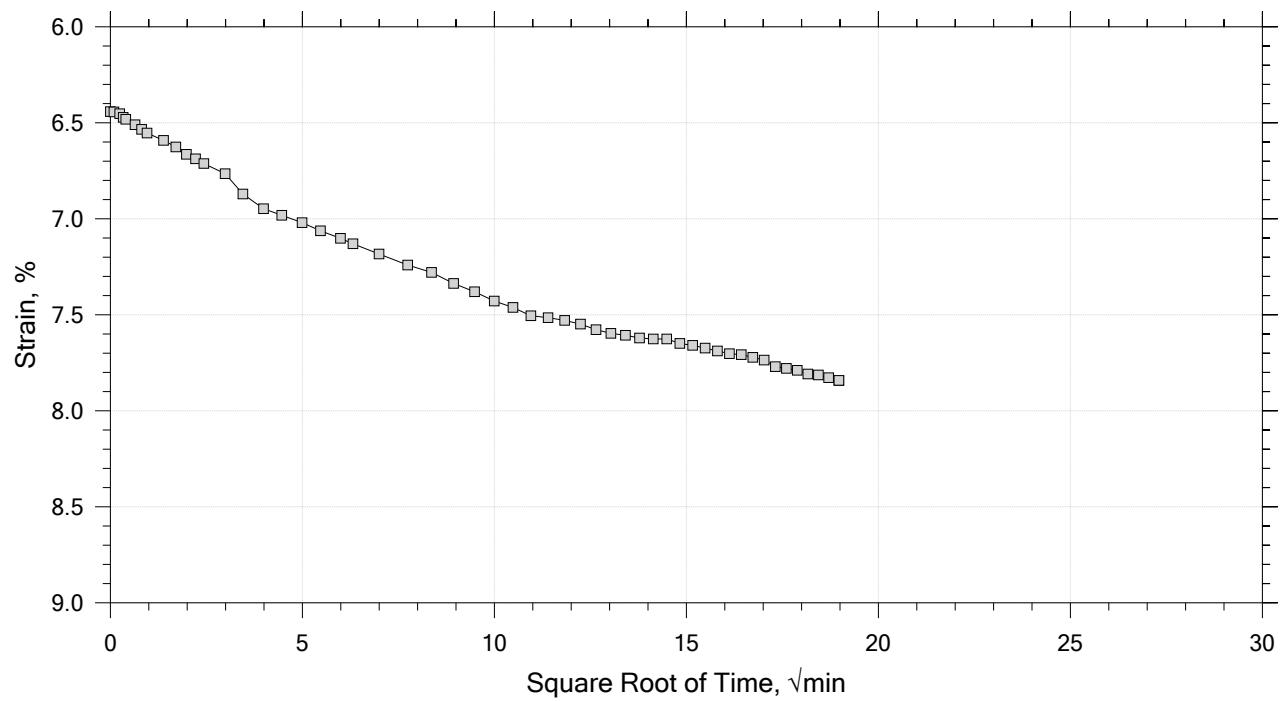
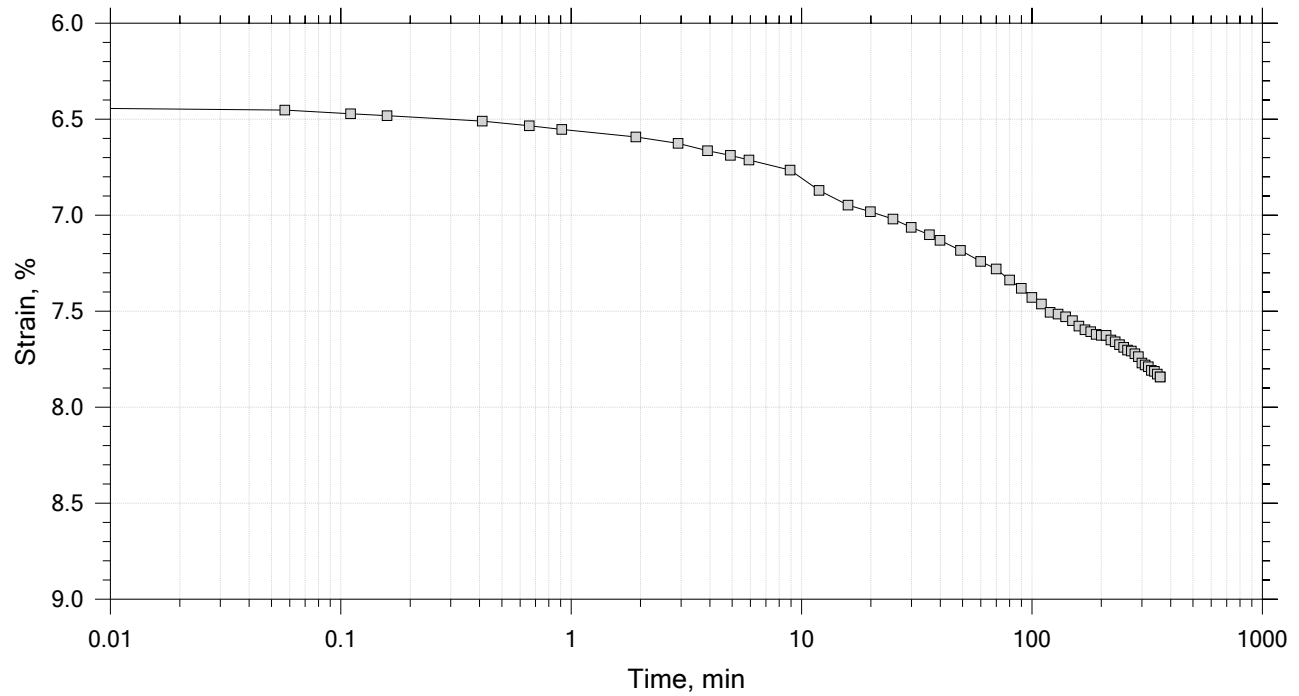
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 17

Constant Load Step

Stress: 3 tsf



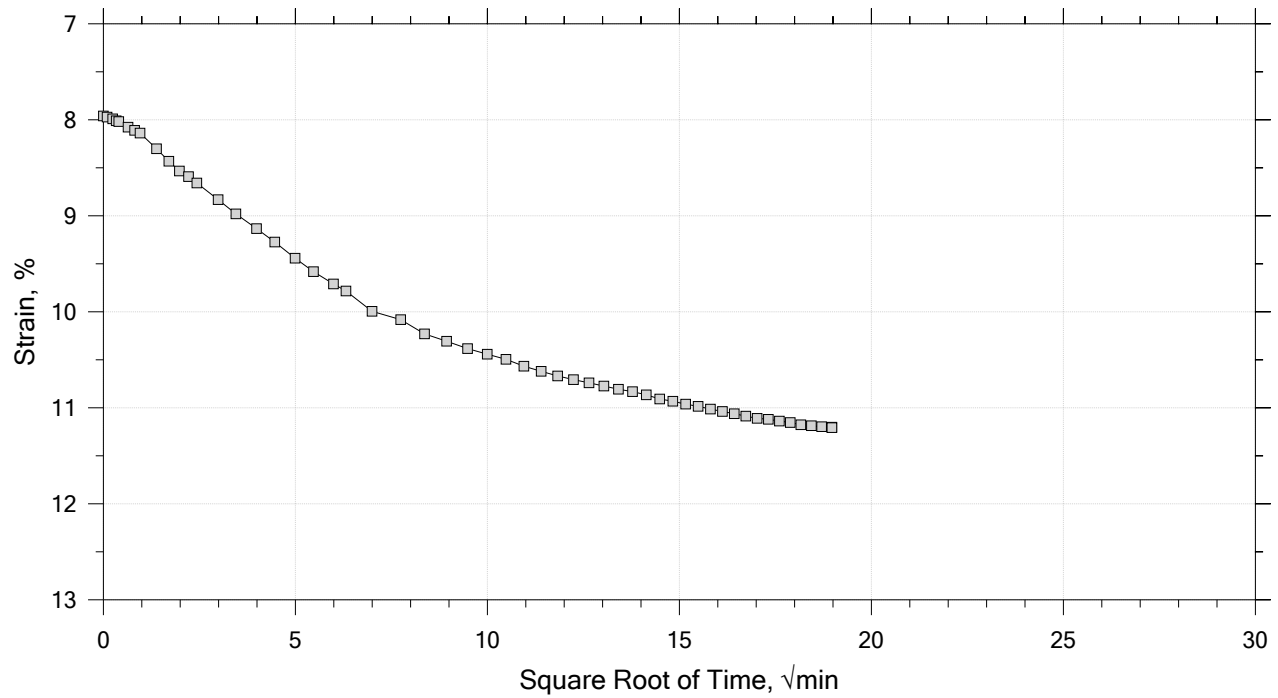
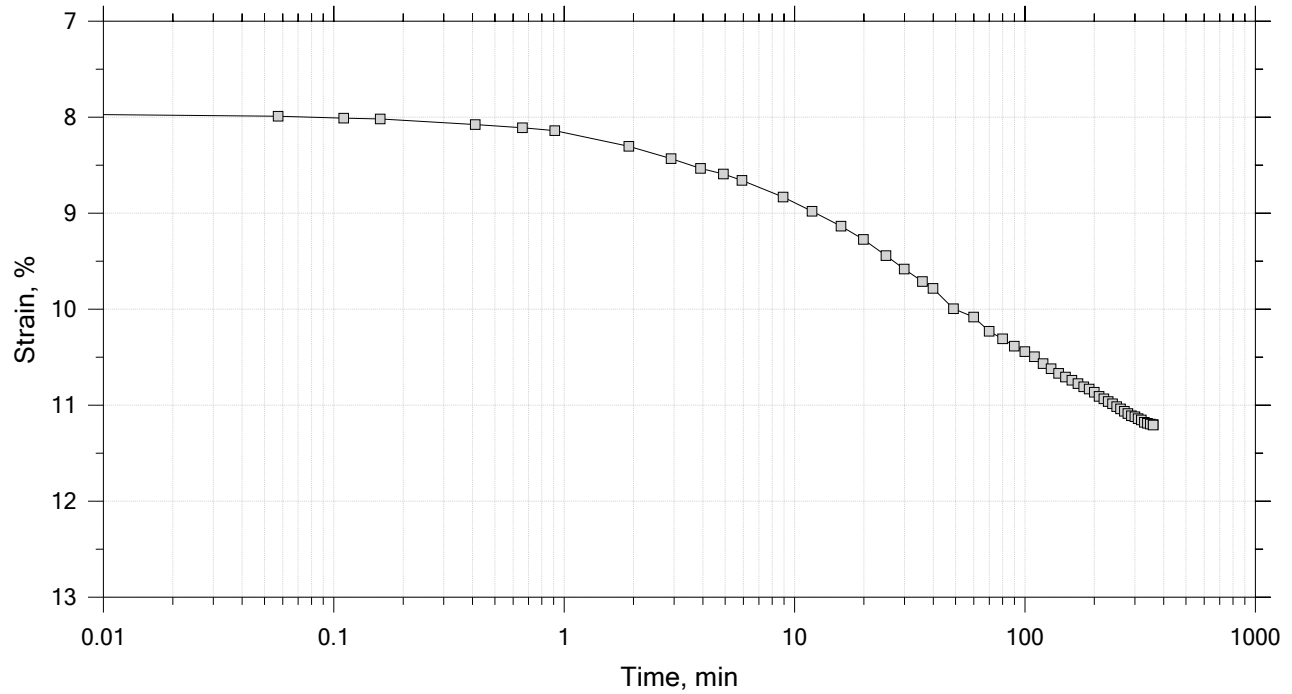
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 17

Constant Load Step

Stress: 4 tsf



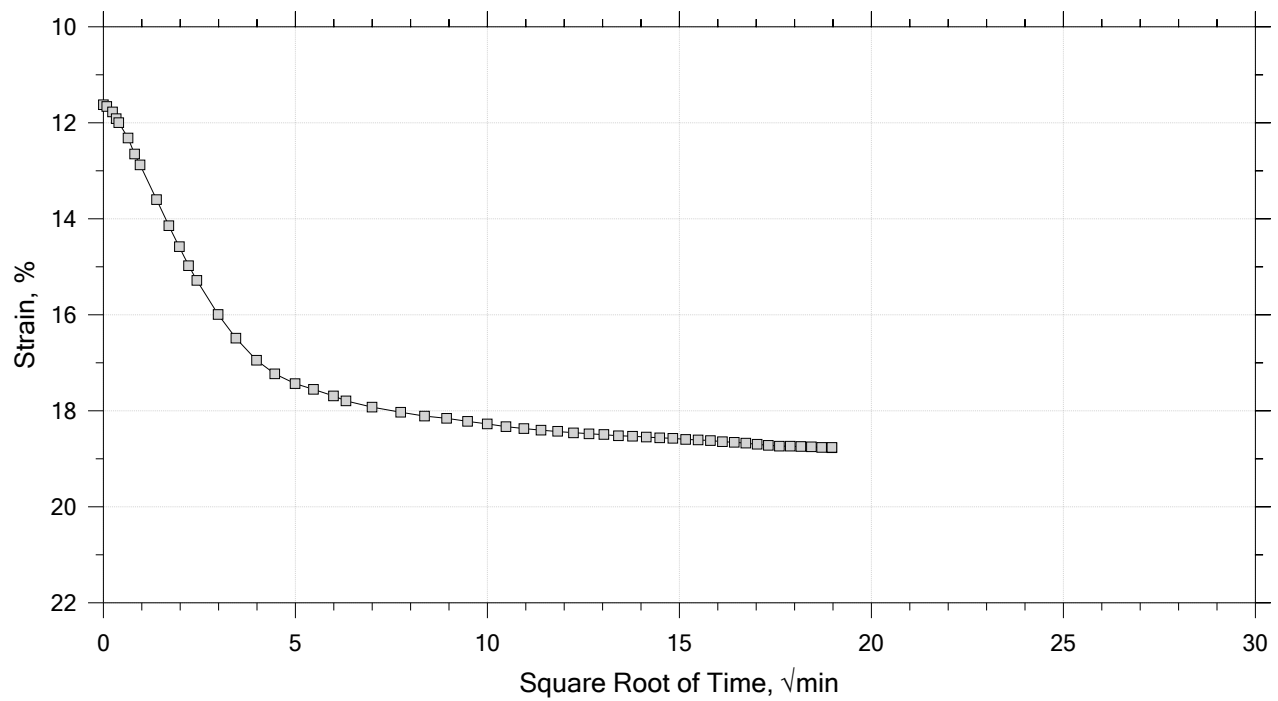
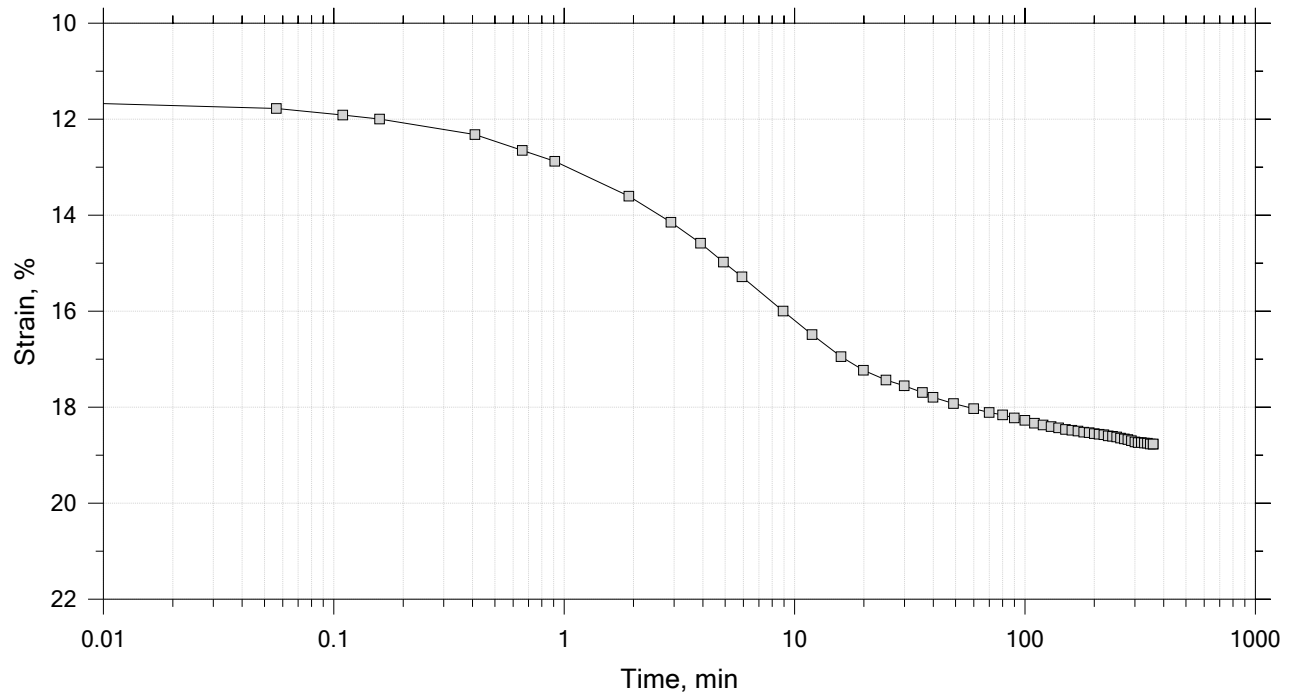
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	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 17

Constant Load Step

Stress: 8 tsf



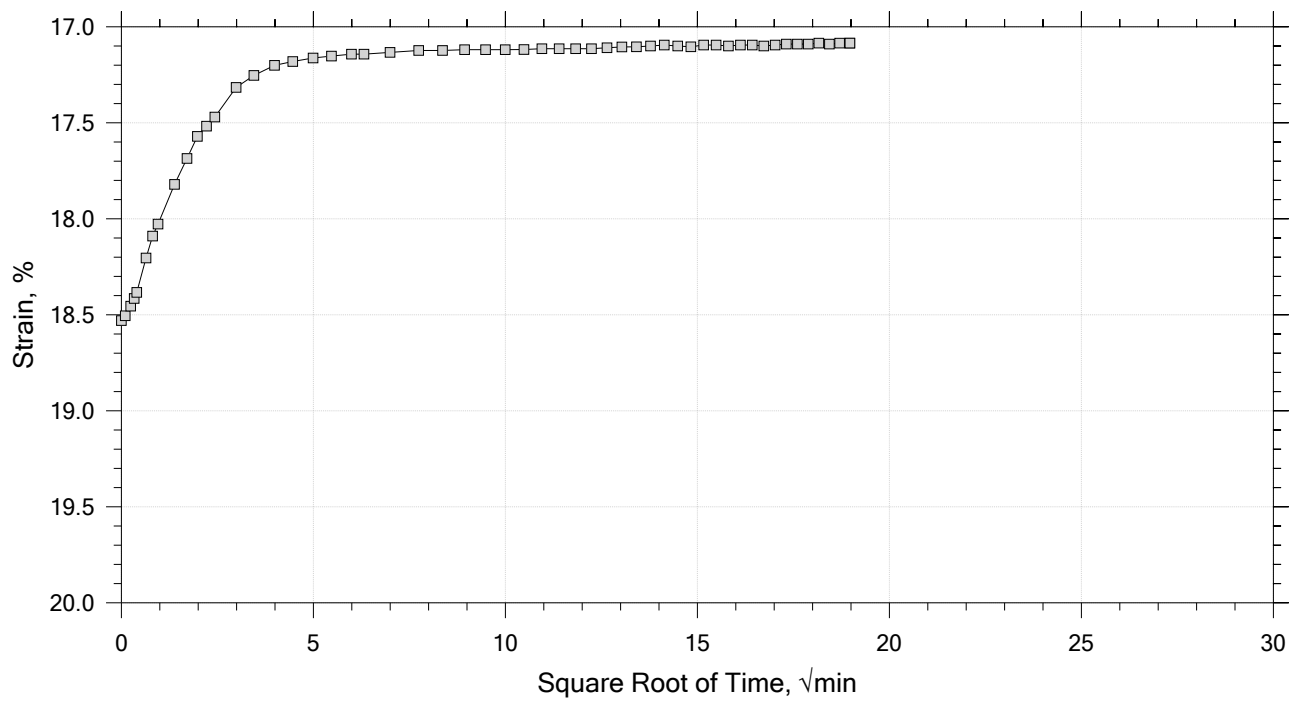
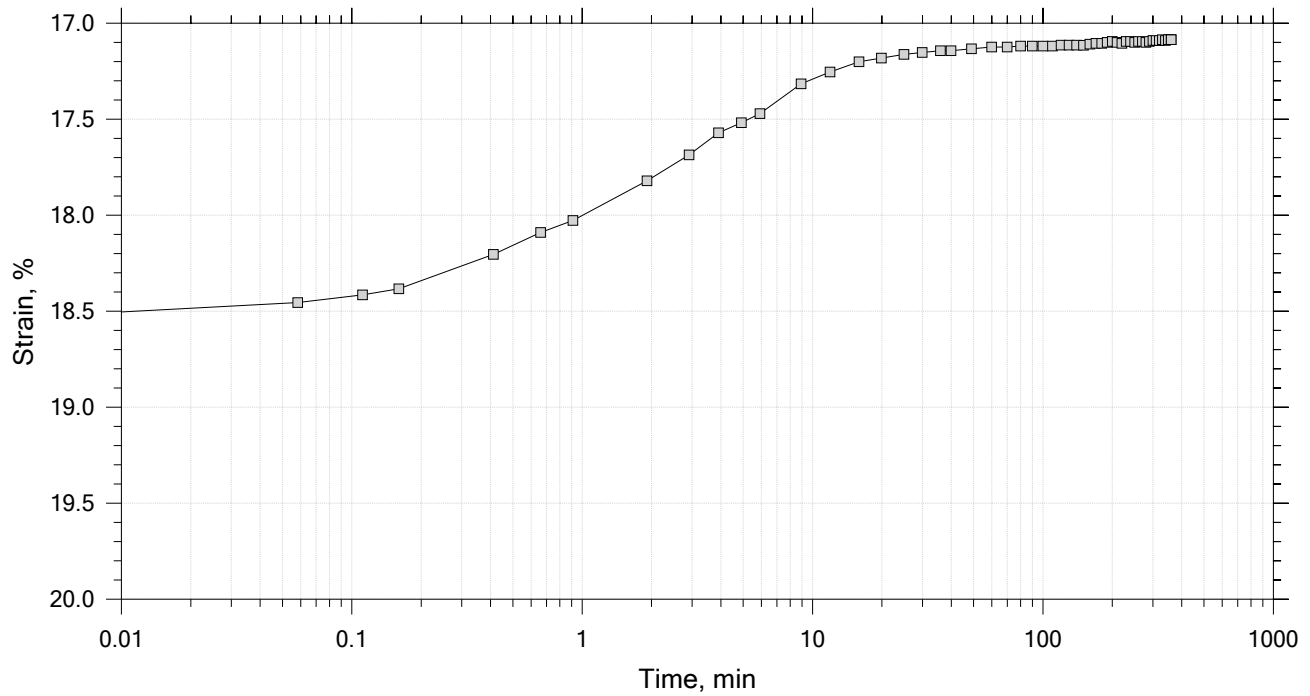
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 17

Constant Load Step

Stress: 1 tsf



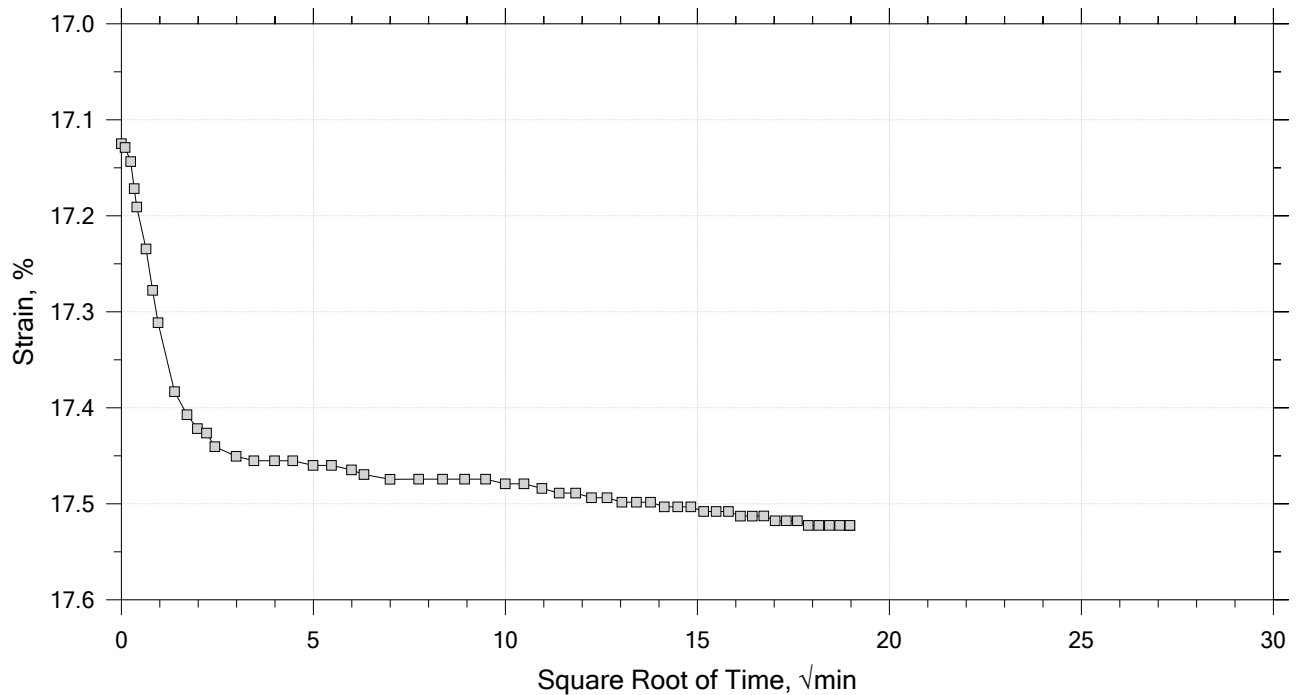
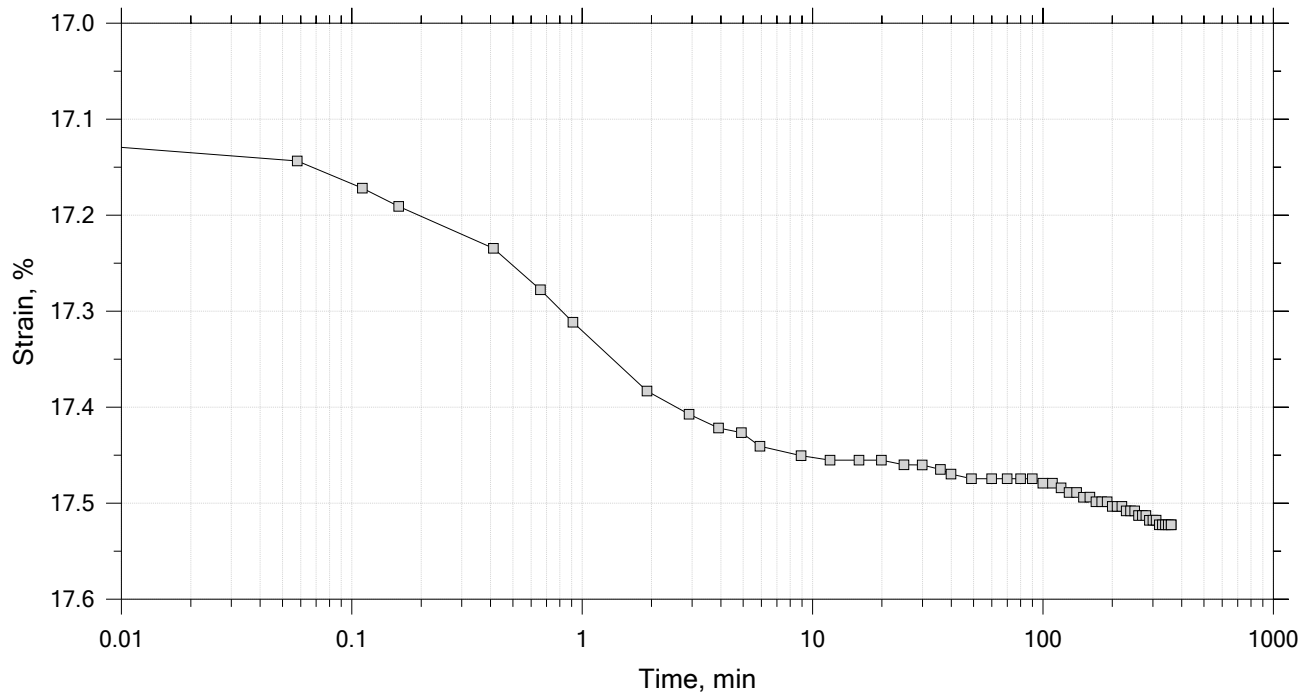
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	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 17

Constant Load Step

Stress: 2 tsf



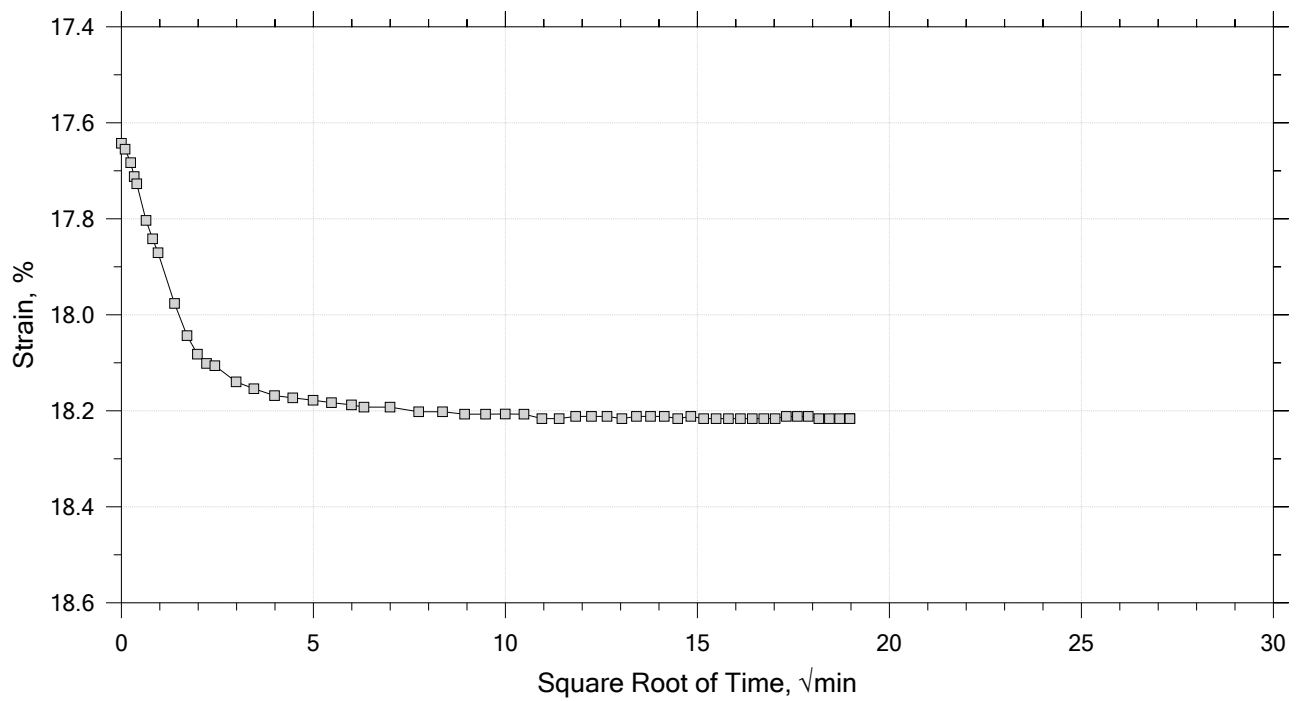
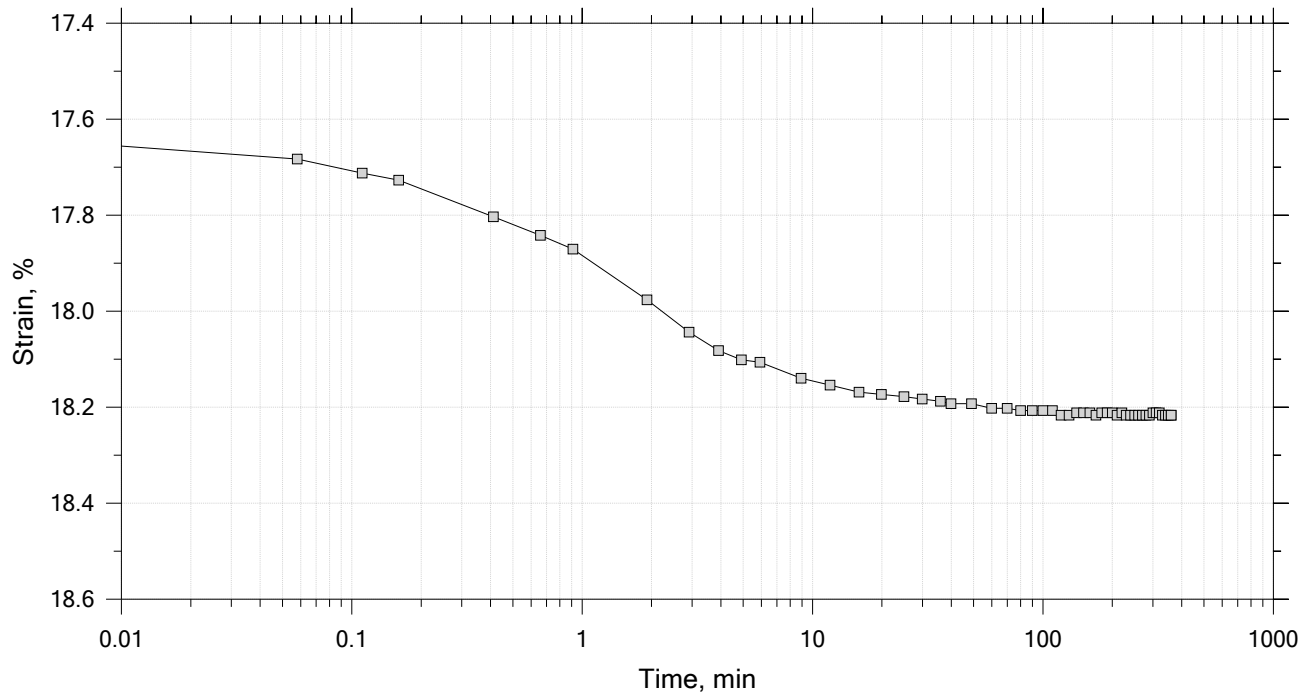
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	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 17

Constant Load Step

Stress: 4 tsf



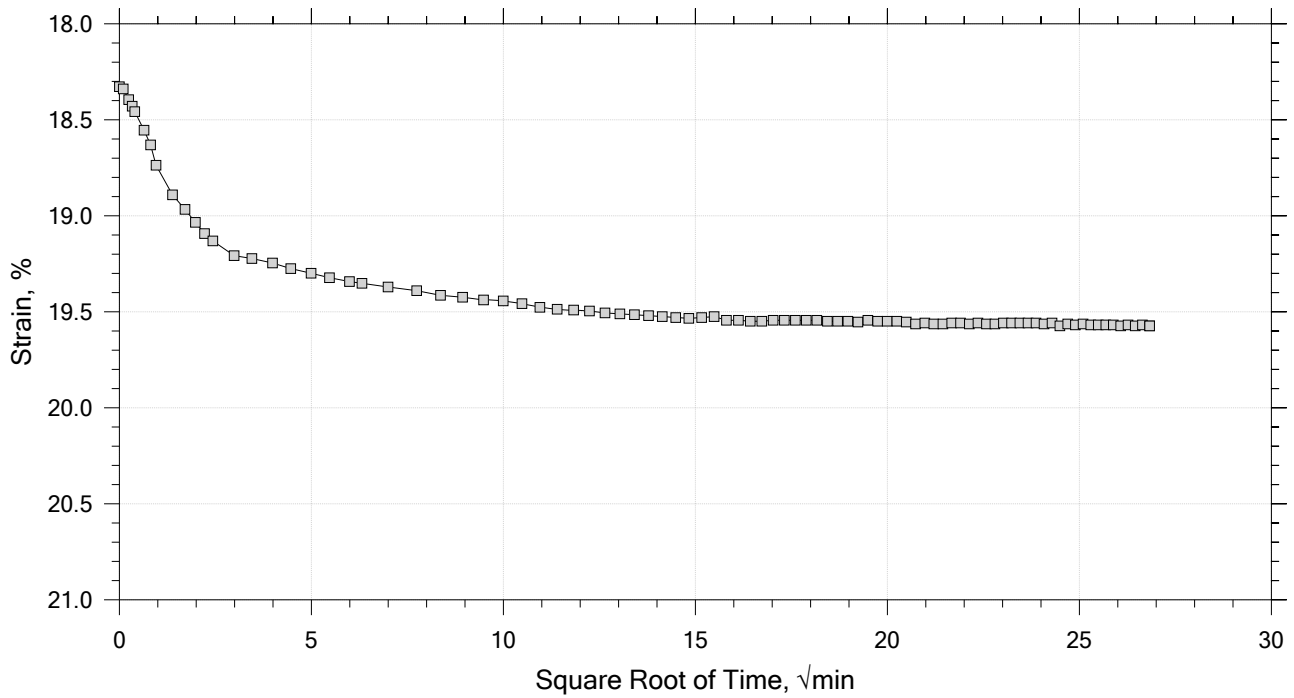
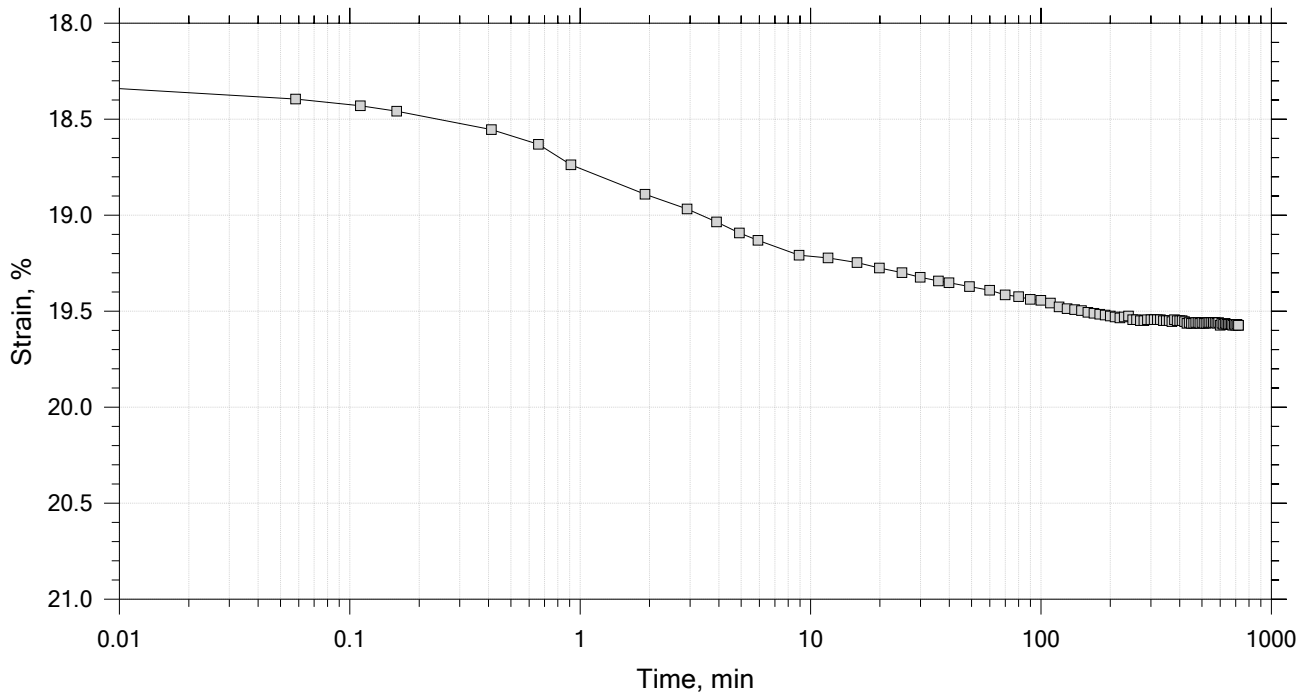
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 17

Constant Load Step

Stress: 7.5 tsf



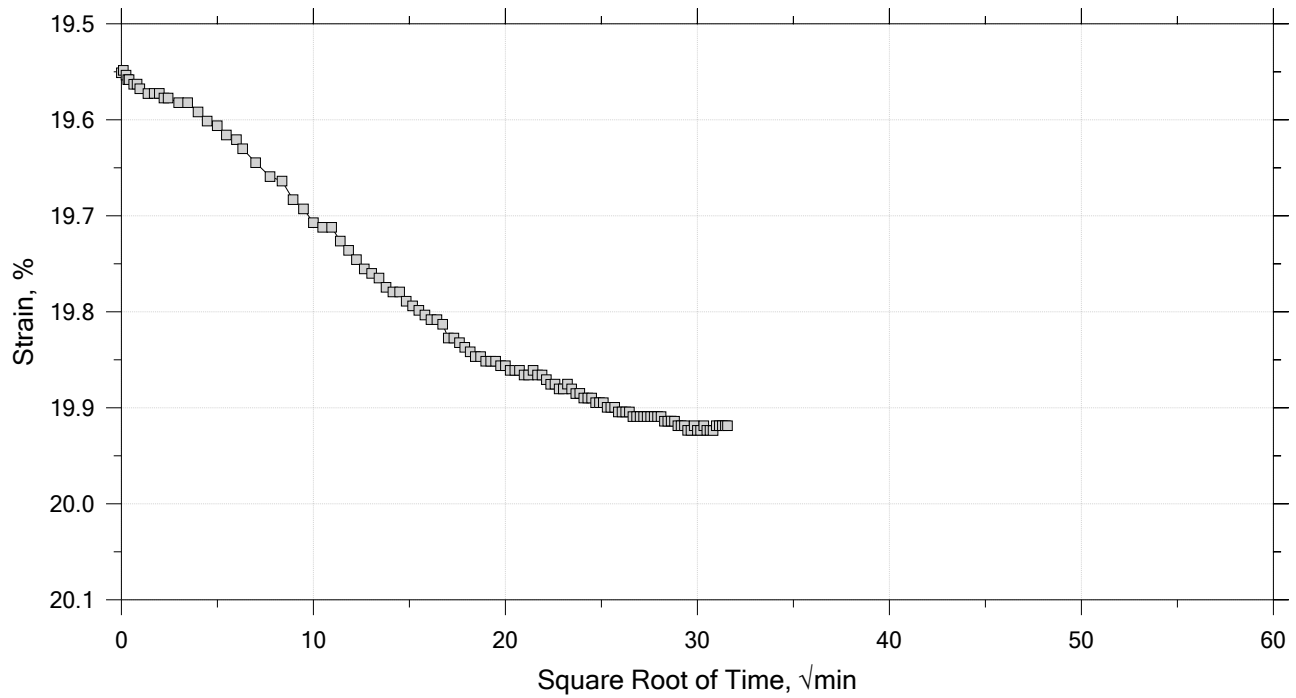
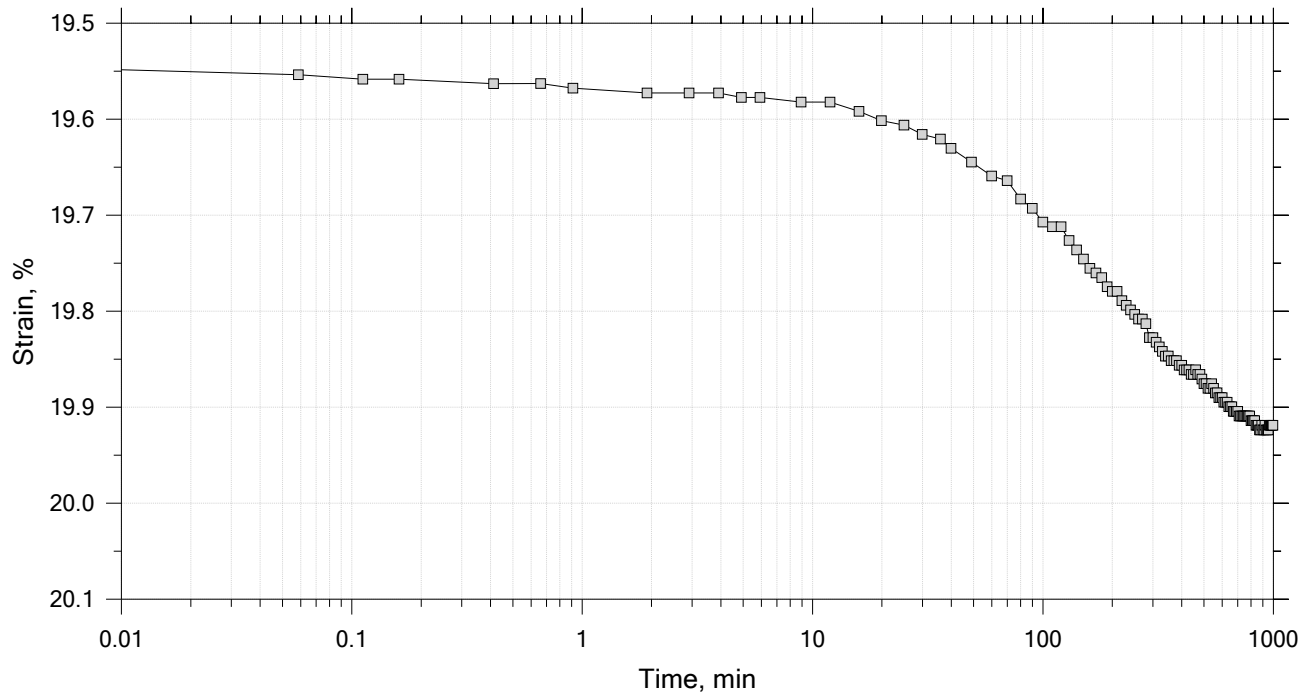
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	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 17

Constant Load Step

Stress: 8 tsf



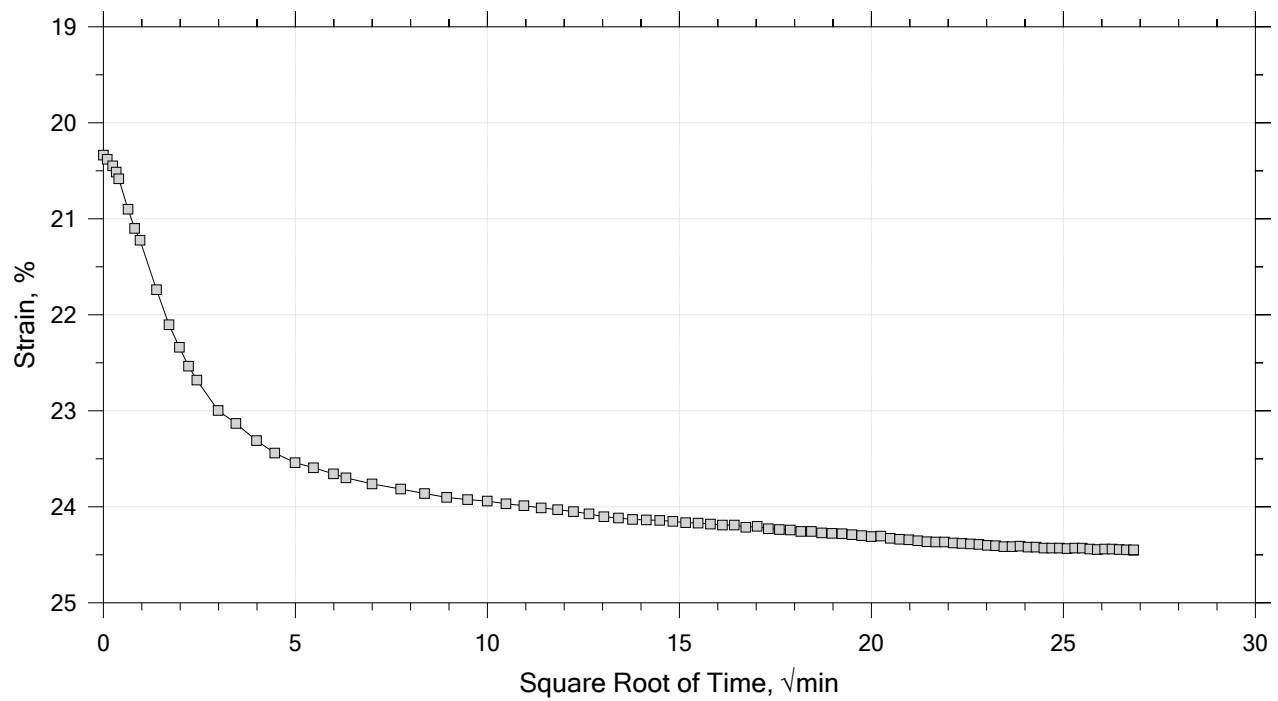
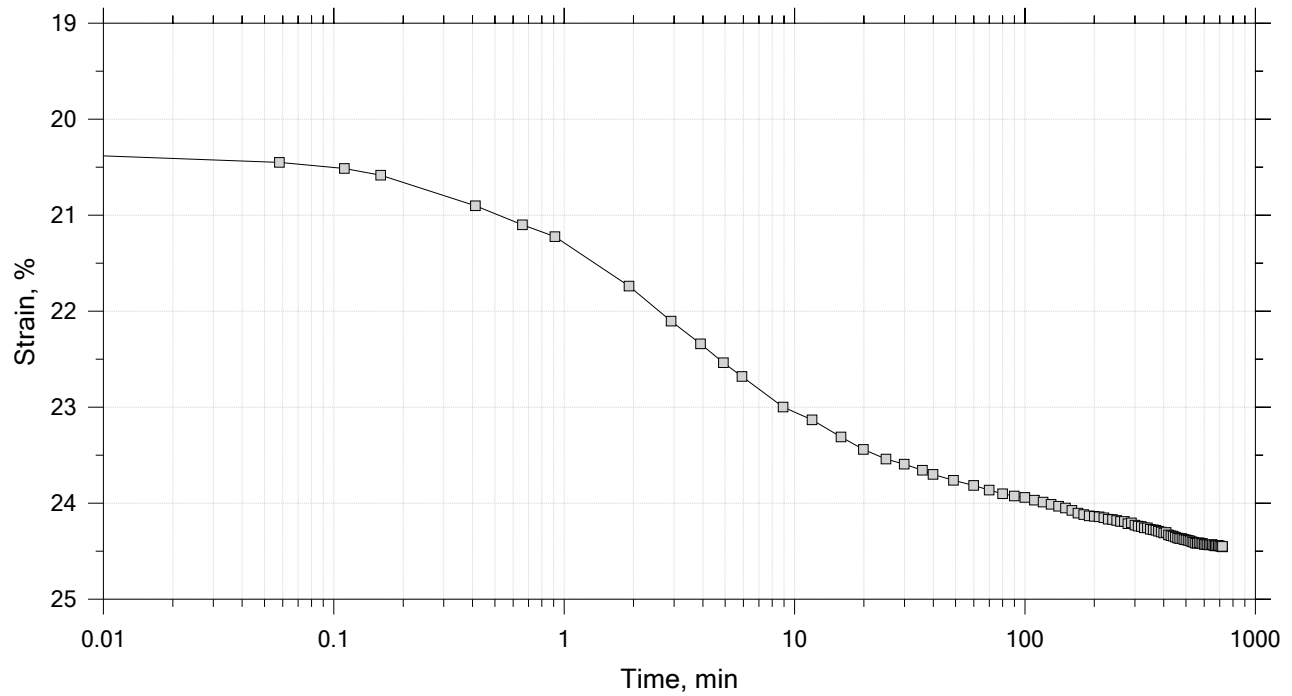
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 17

Constant Load Step

Stress: 16 tsf



	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.78	Liquid Limit: 46
Initial Height: 1.00 in	Initial Void Ratio: 1.28	Plastic Limit: 24
Final Height: 0.81 in	Final Void Ratio: 0.849	Plasticity Index: 22

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-2362	RING		D1549
Mass Container, gm	8.23	109.22	109.22	8.42
Mass Container + Wet Soil, gm	117.95	252.26	237.05	135.33
Mass Container + Dry Soil, gm	83.83	207.12	207.12	105.62
Mass Dry Soil, gm	75.6	97.905	97.905	97.2
Water Content, %	45.13	46.10	30.57	30.57
Void Ratio	---	1.28	0.85	---
Degree of Saturation, %	---	99.84	100.00	---
Dry Unit Weight, pcf	---	75.982	93.805	---

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: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		

One-Dimensional Consolidation by ASTM D2435 - Method B

Log of Time Coefficients

[illegible]

	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

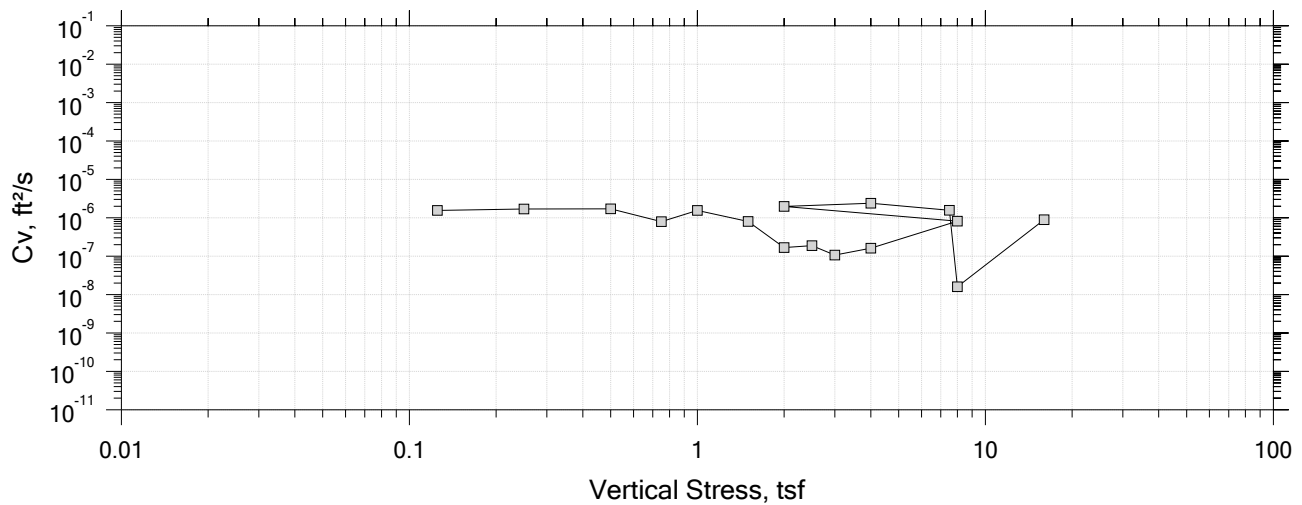
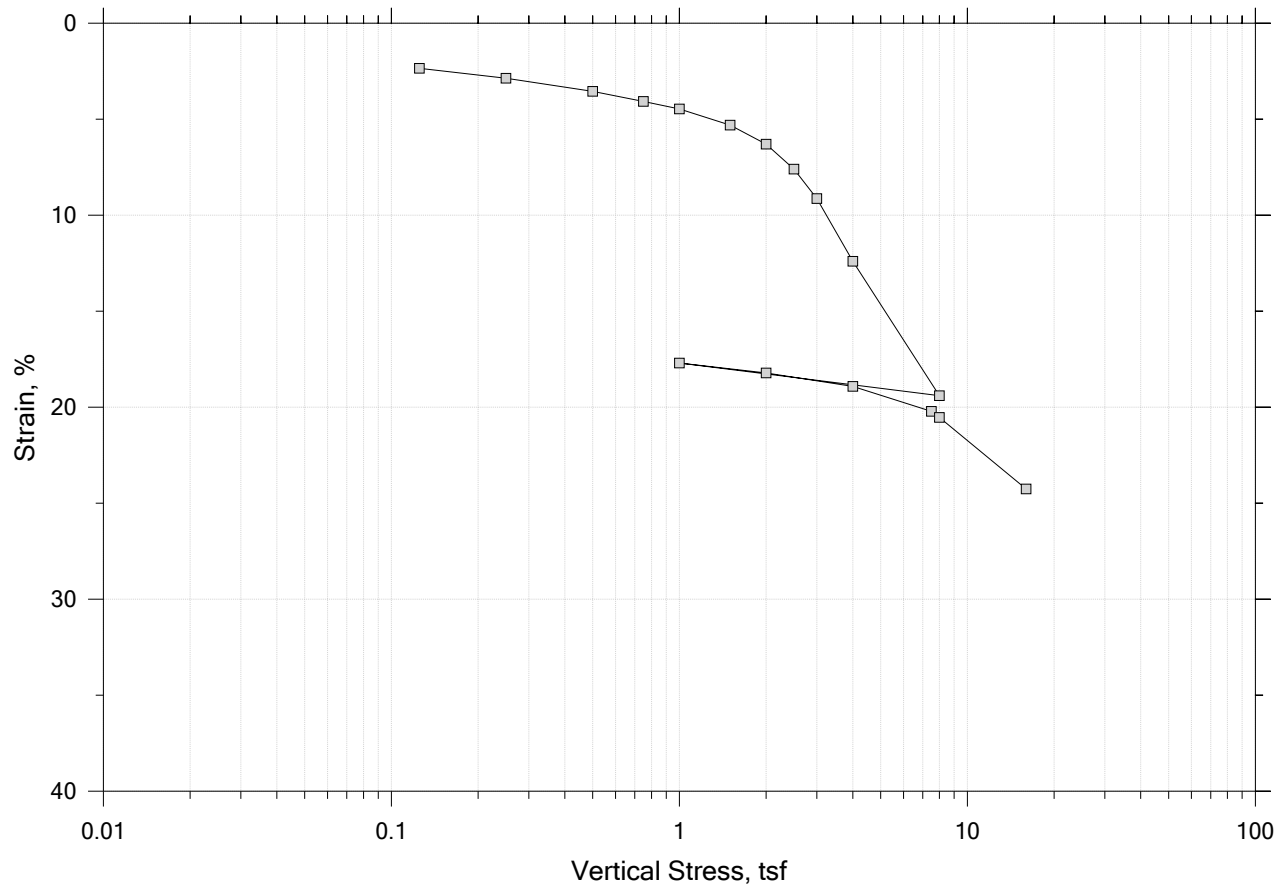
Square Root of Time Coefficients

[illegible]

	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 09/26/18	Depth: 25-27 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, dark gray clay		
	Remarks: System II		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report



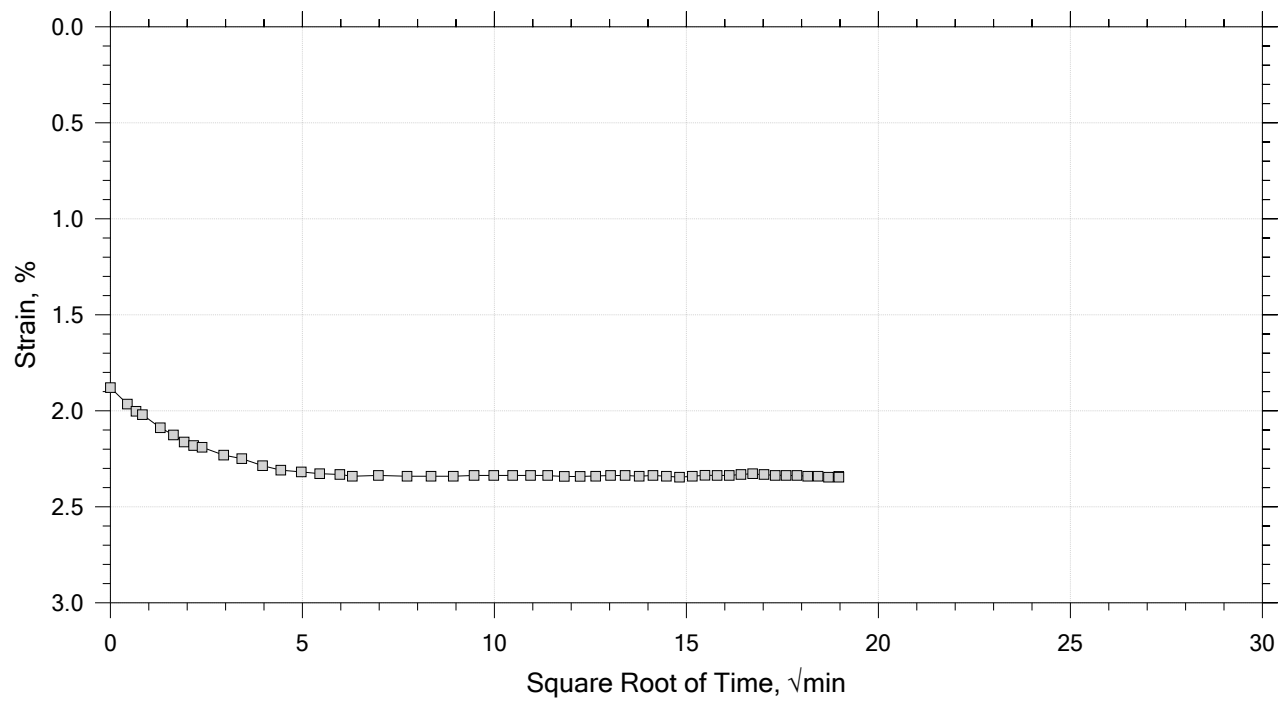
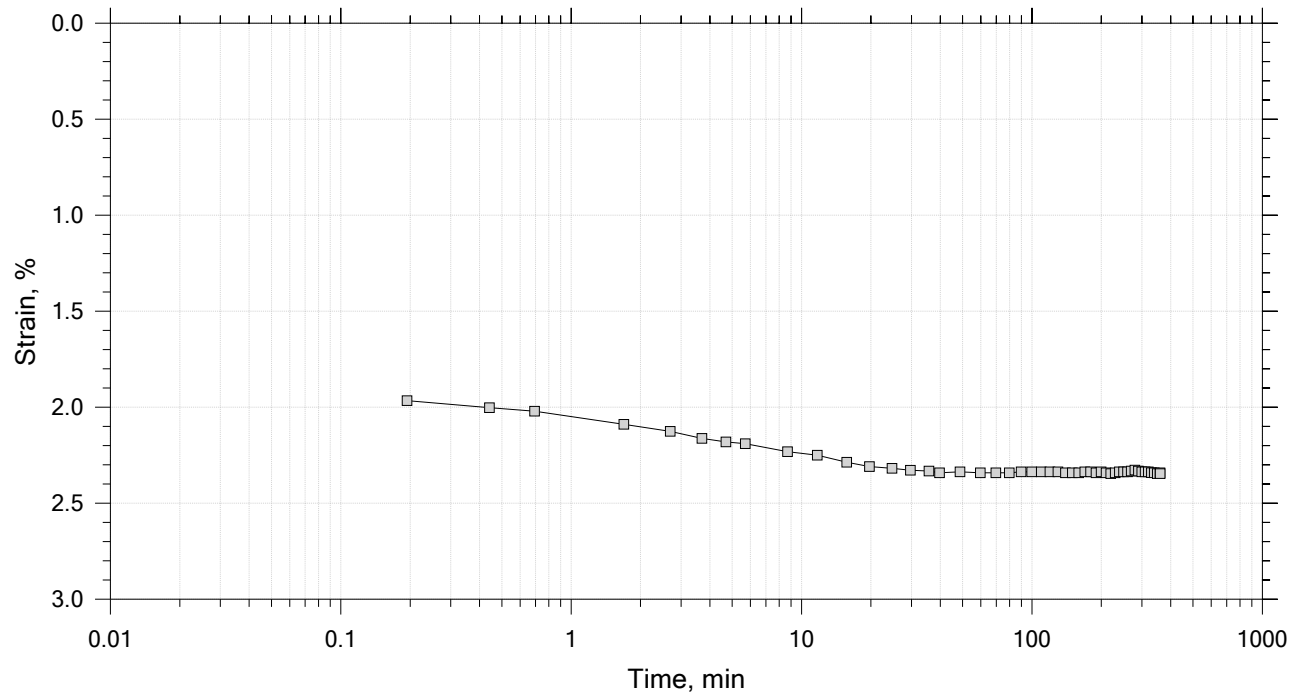
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 17

Constant Load Step

Stress: 0.125 tsf



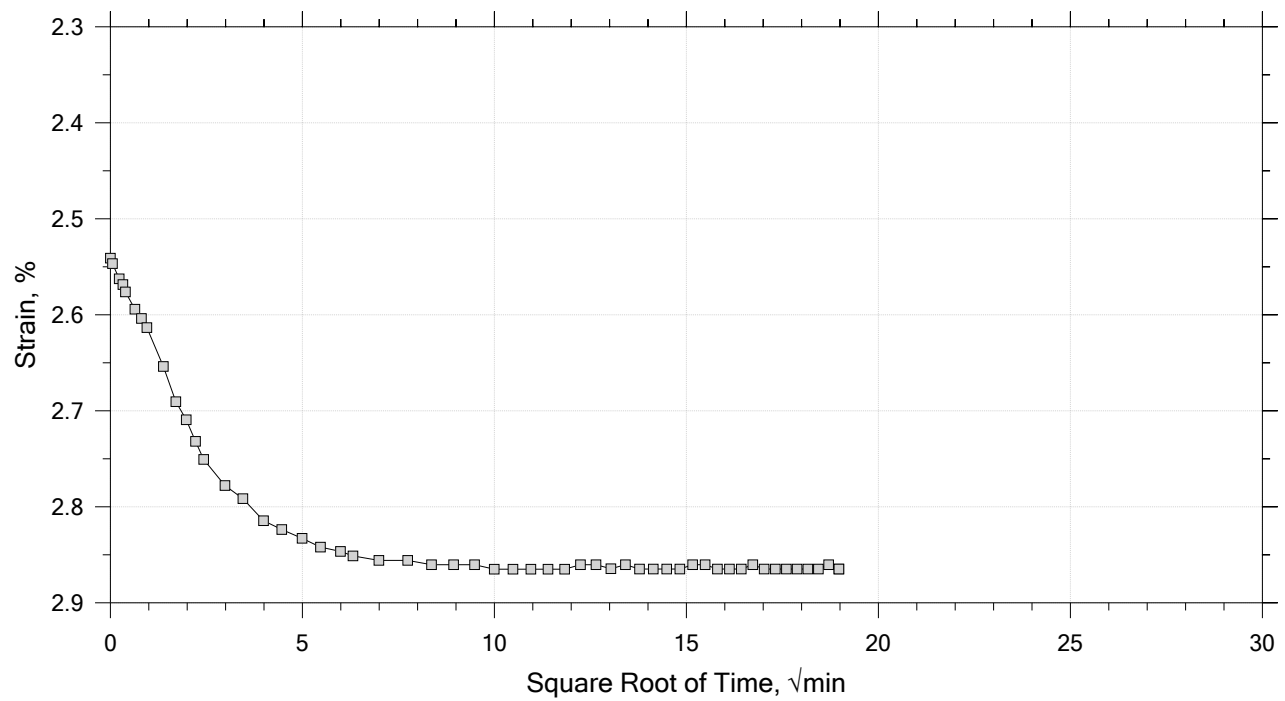
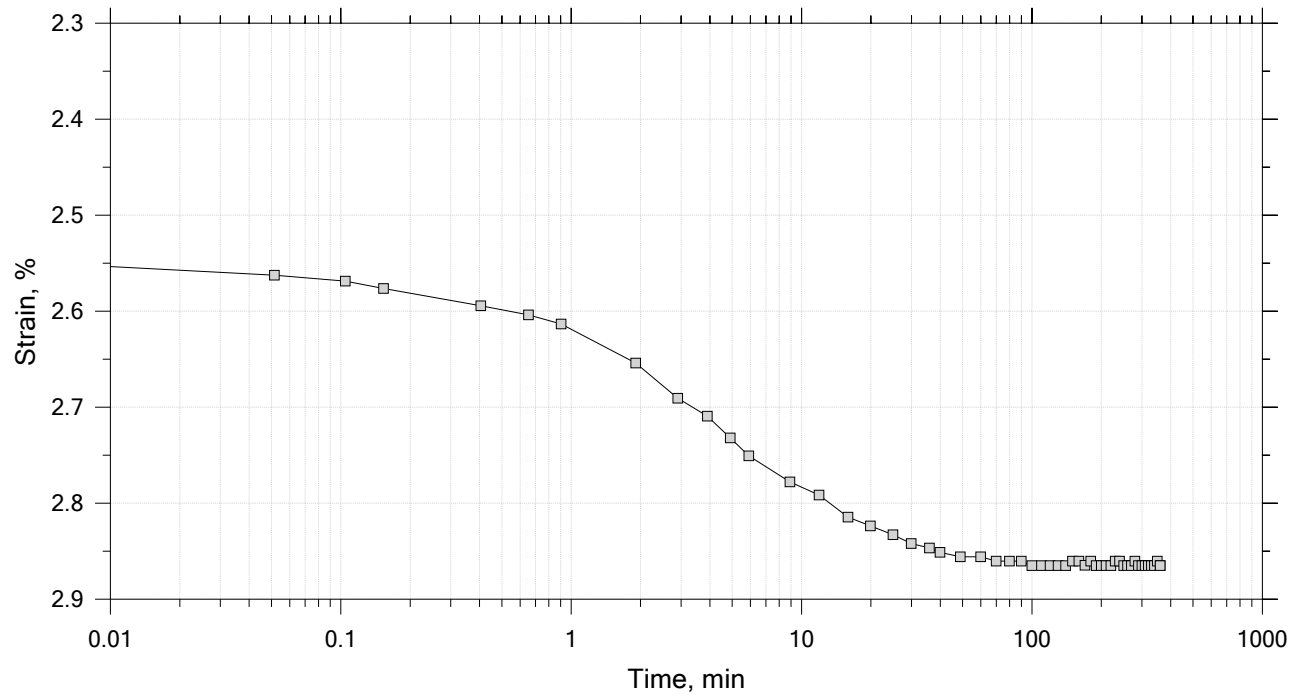
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 17

Constant Load Step

Stress: 0.25 tsf



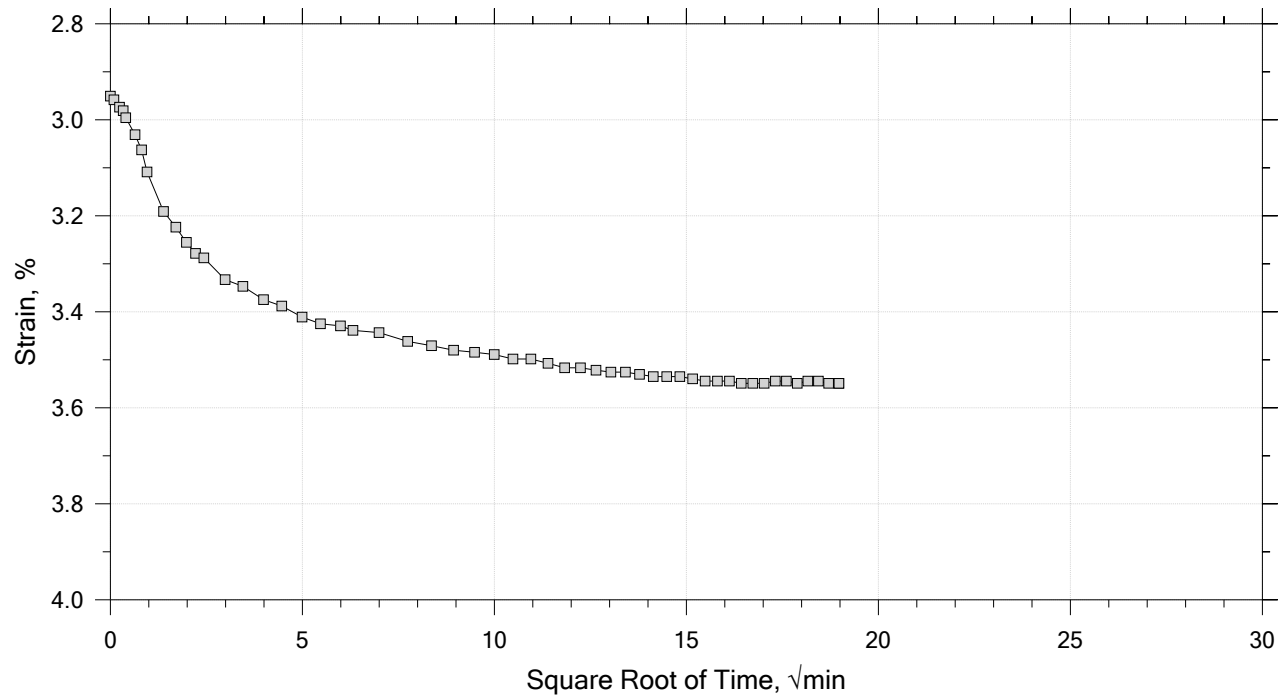
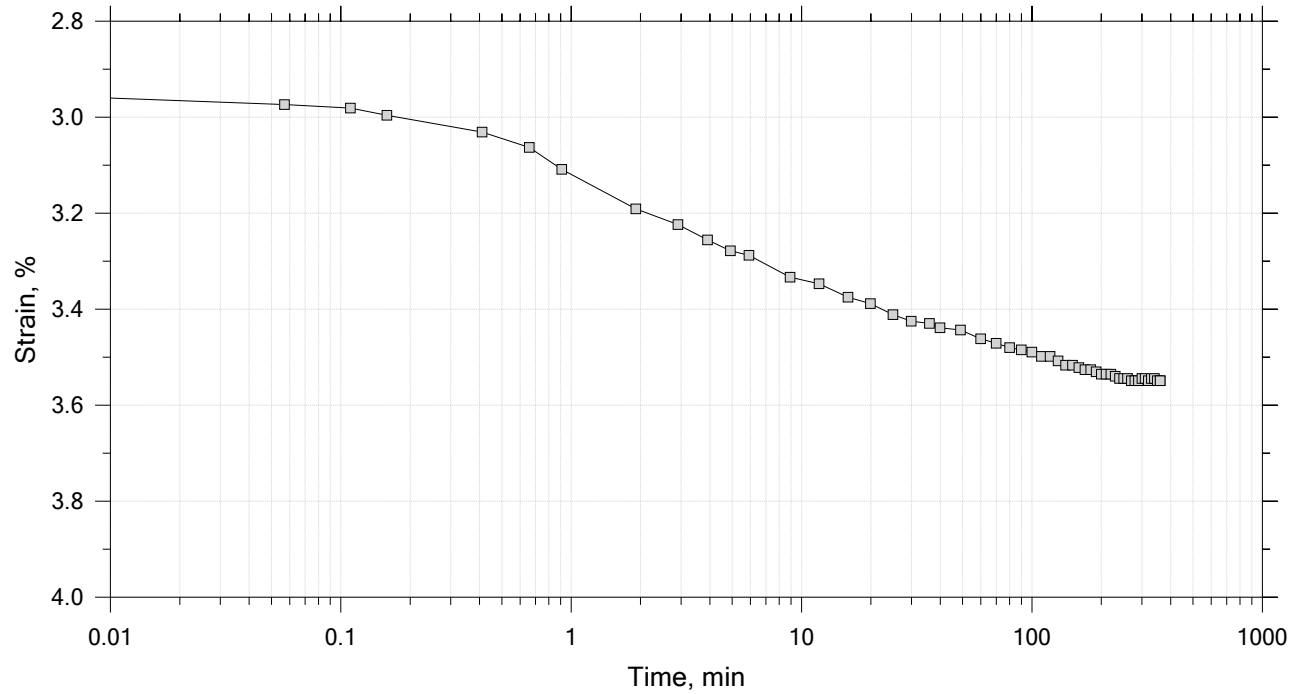
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 17

Constant Load Step

Stress: 0.5 tsf



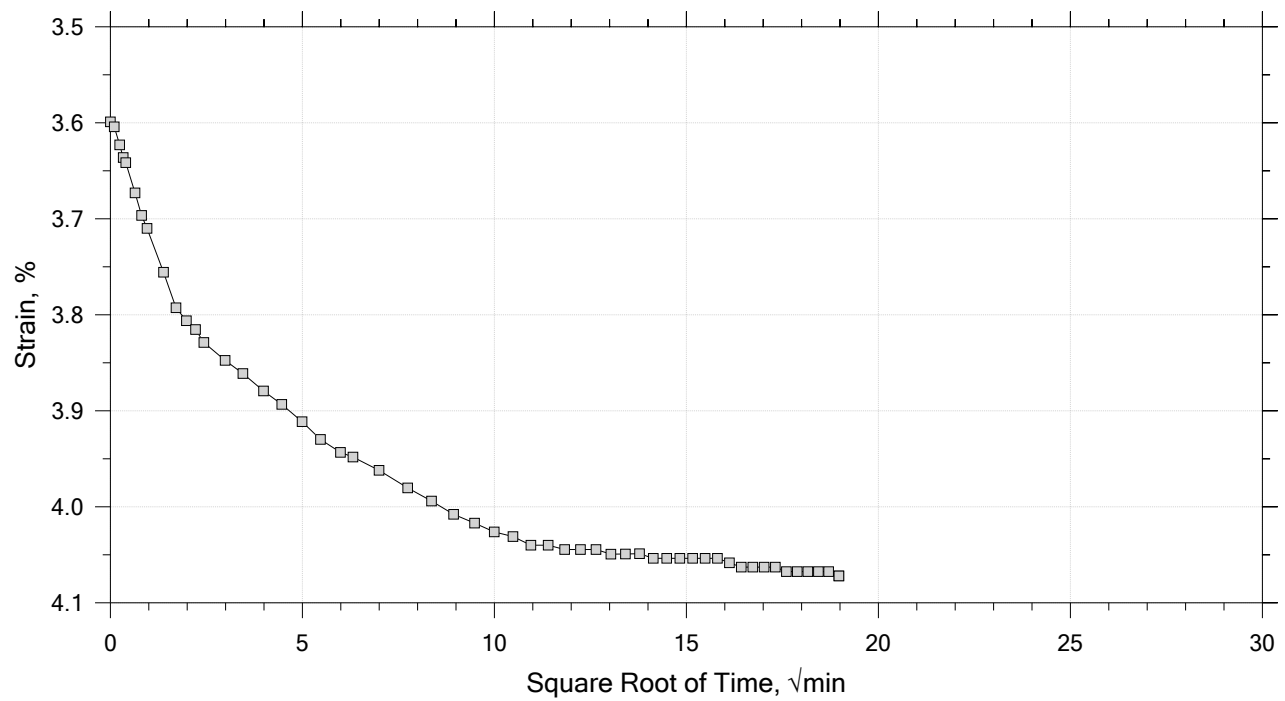
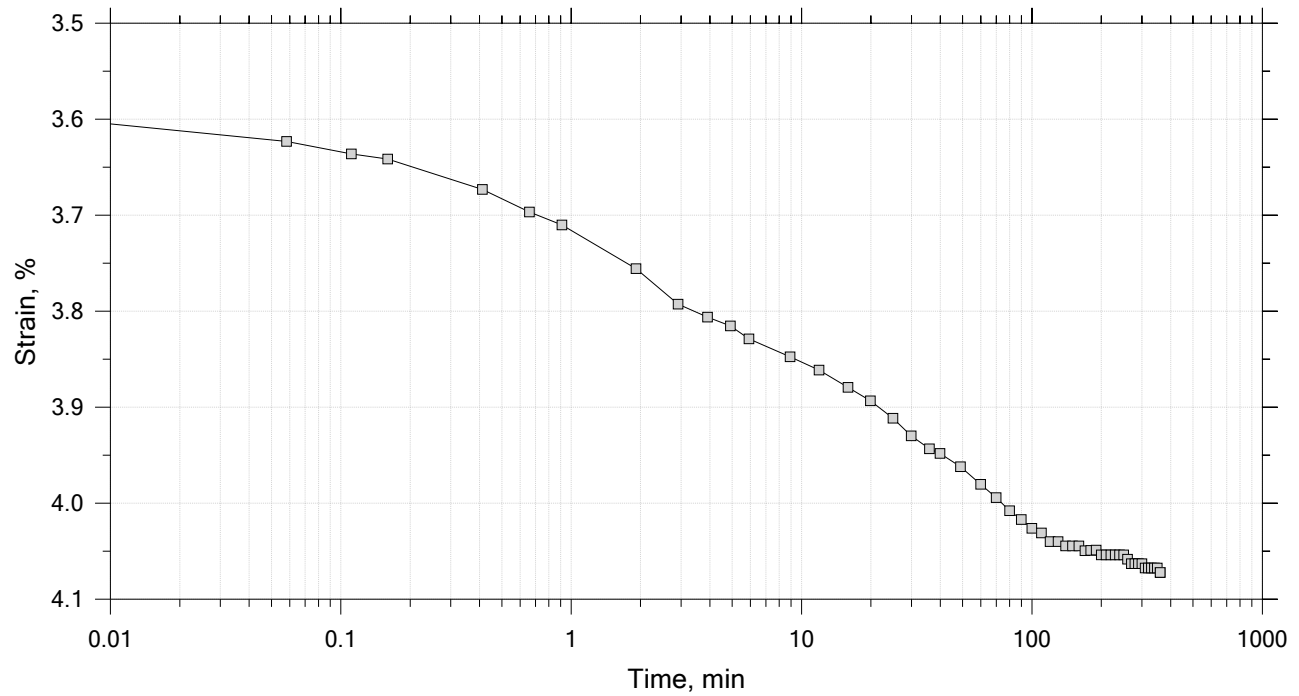
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 17

Constant Load Step

Stress: 0.75 tsf



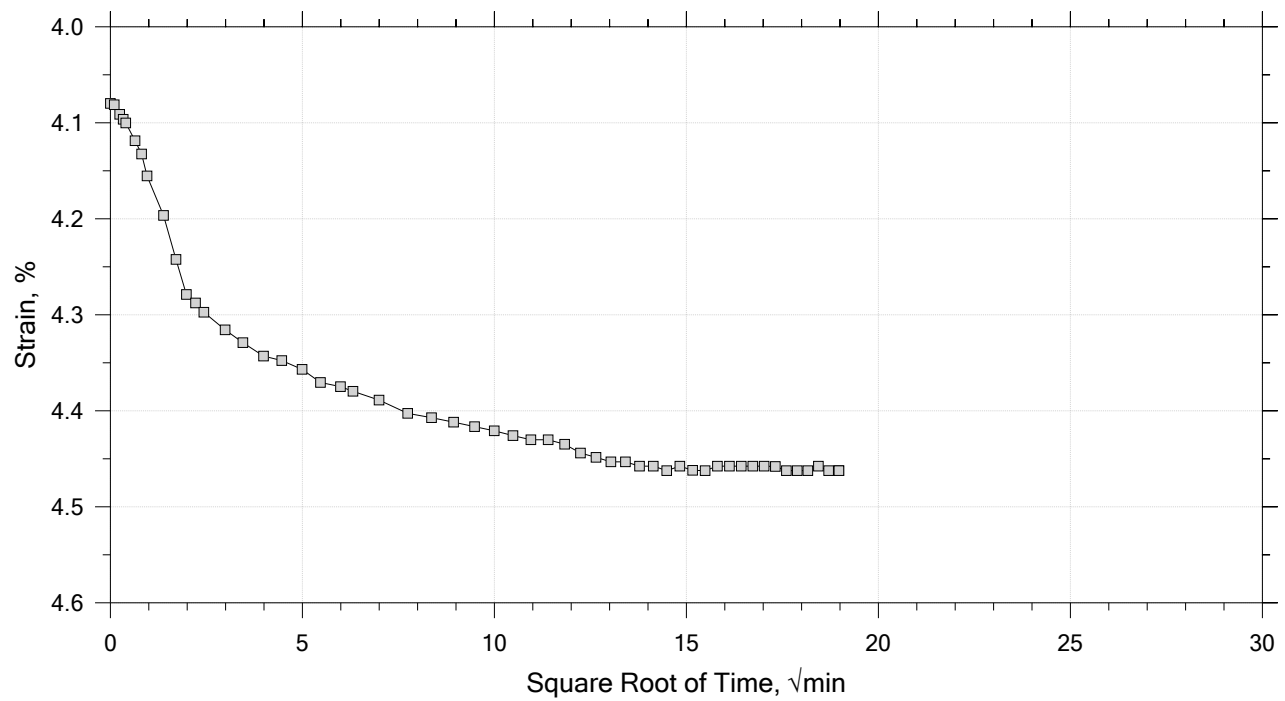
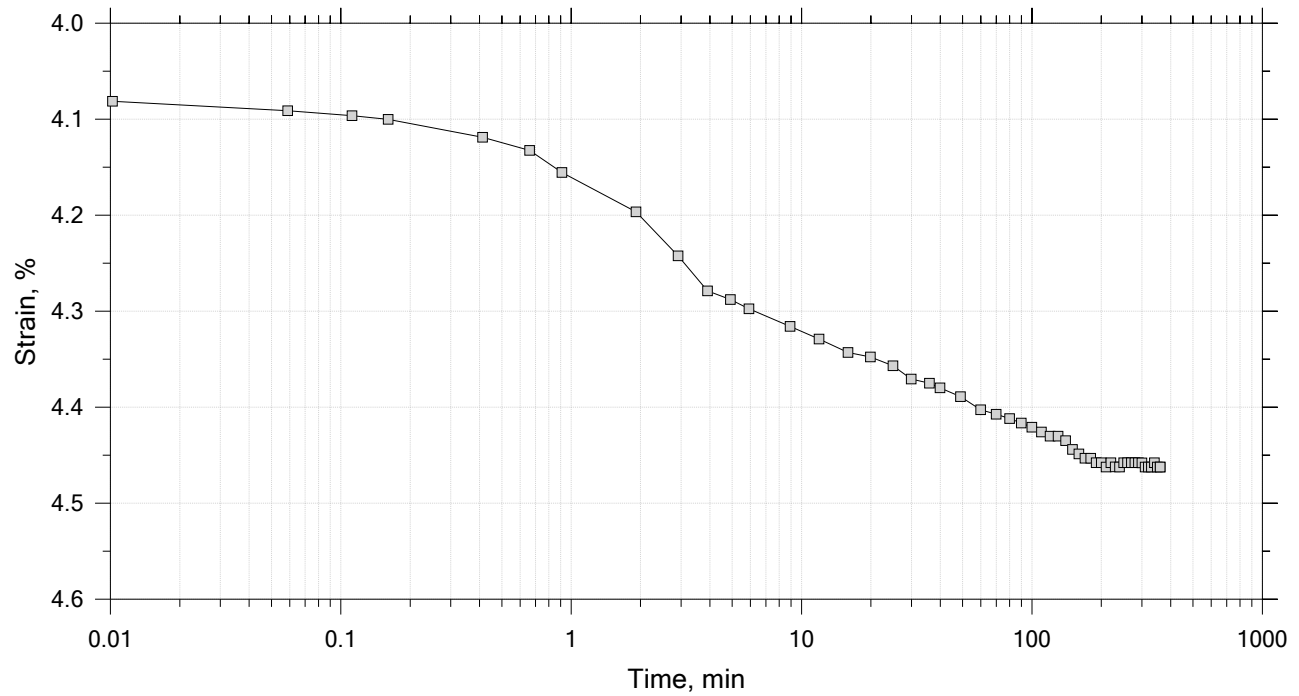
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	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 17

Constant Load Step

Stress: 1 tsf



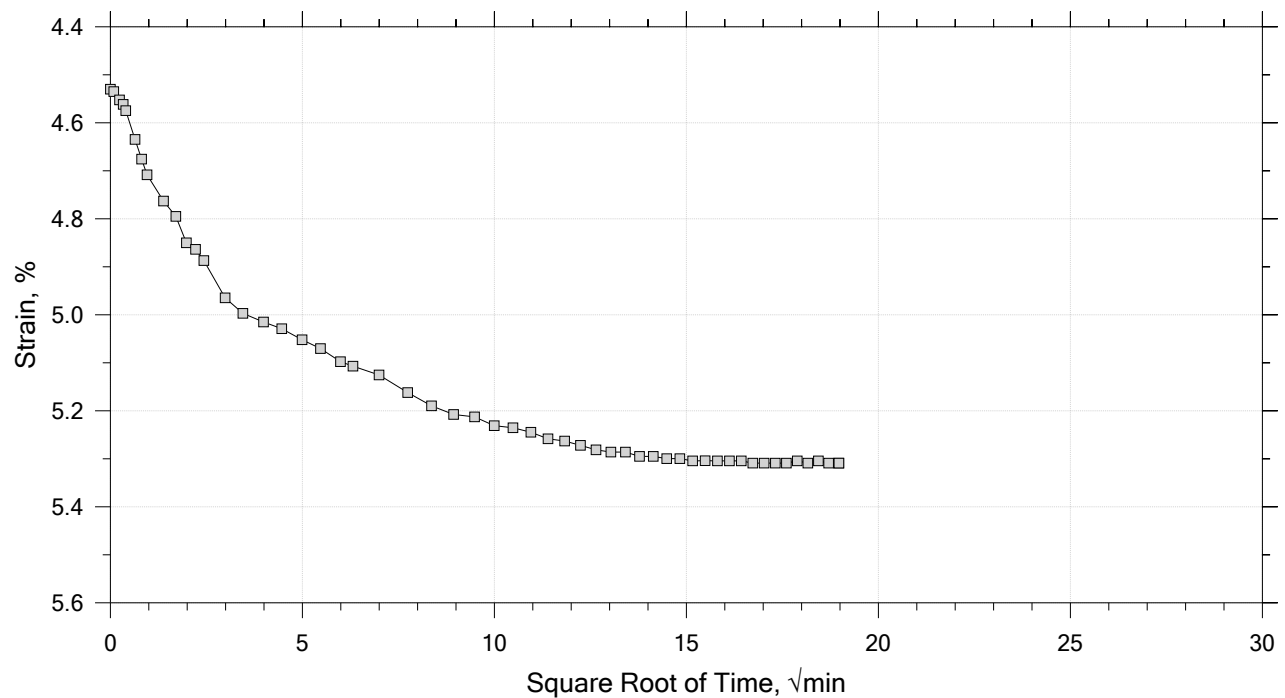
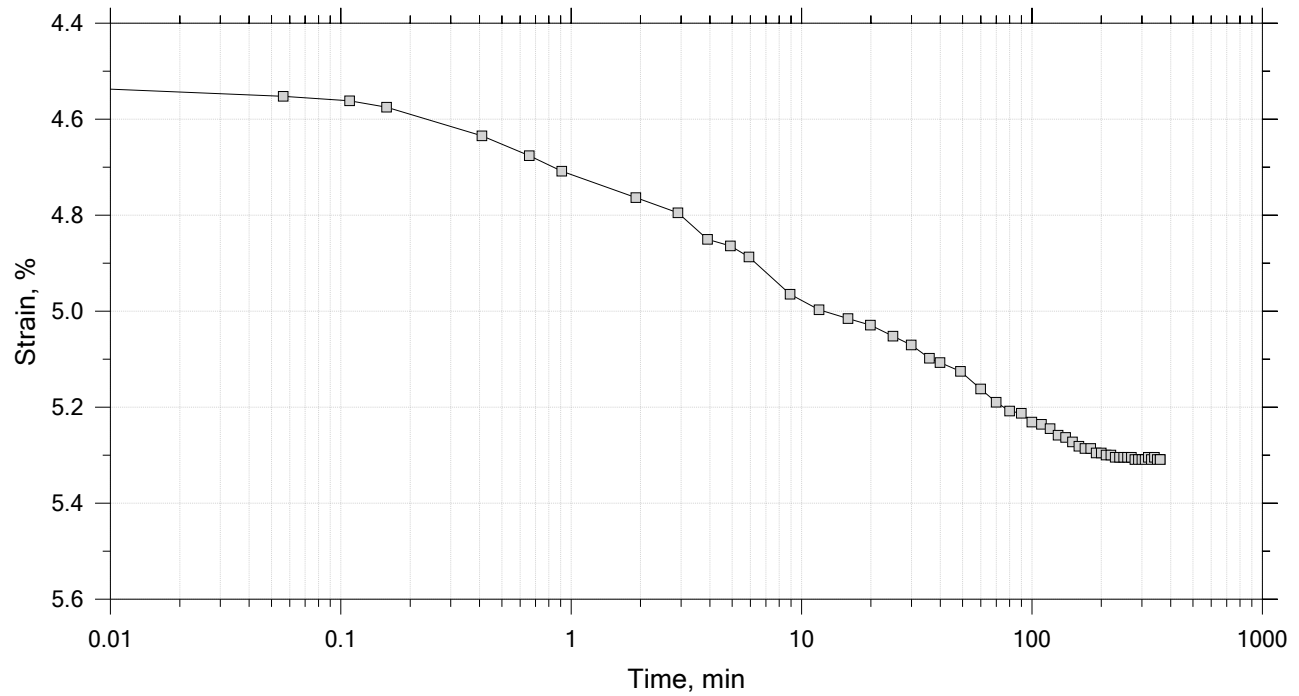
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	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 17

Constant Load Step

Stress: 1.5 tsf



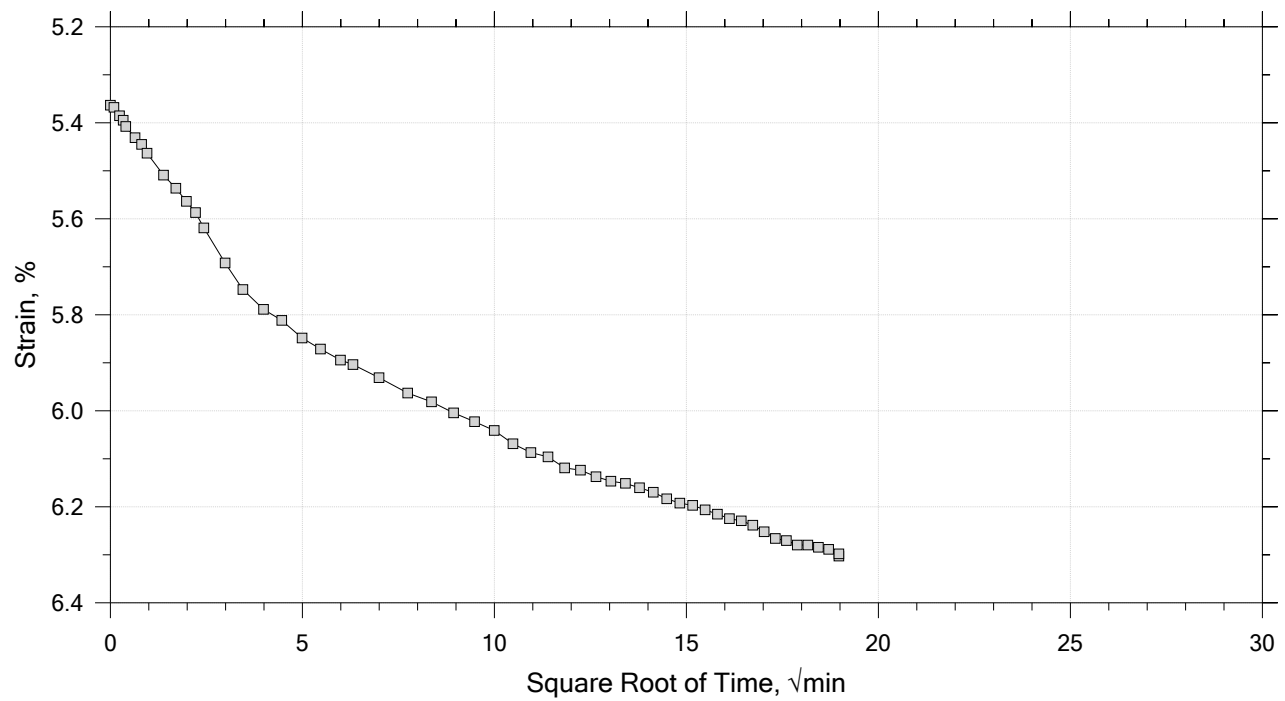
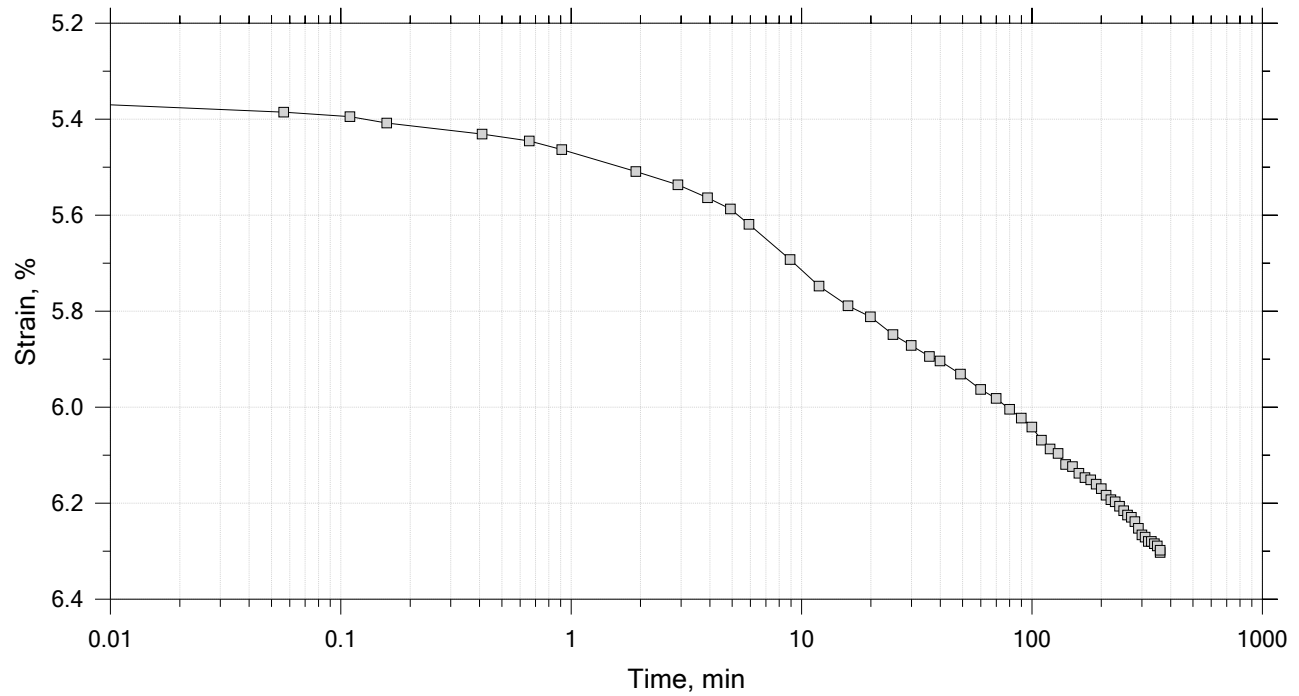
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	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 17

Constant Load Step

Stress: 2 tsf



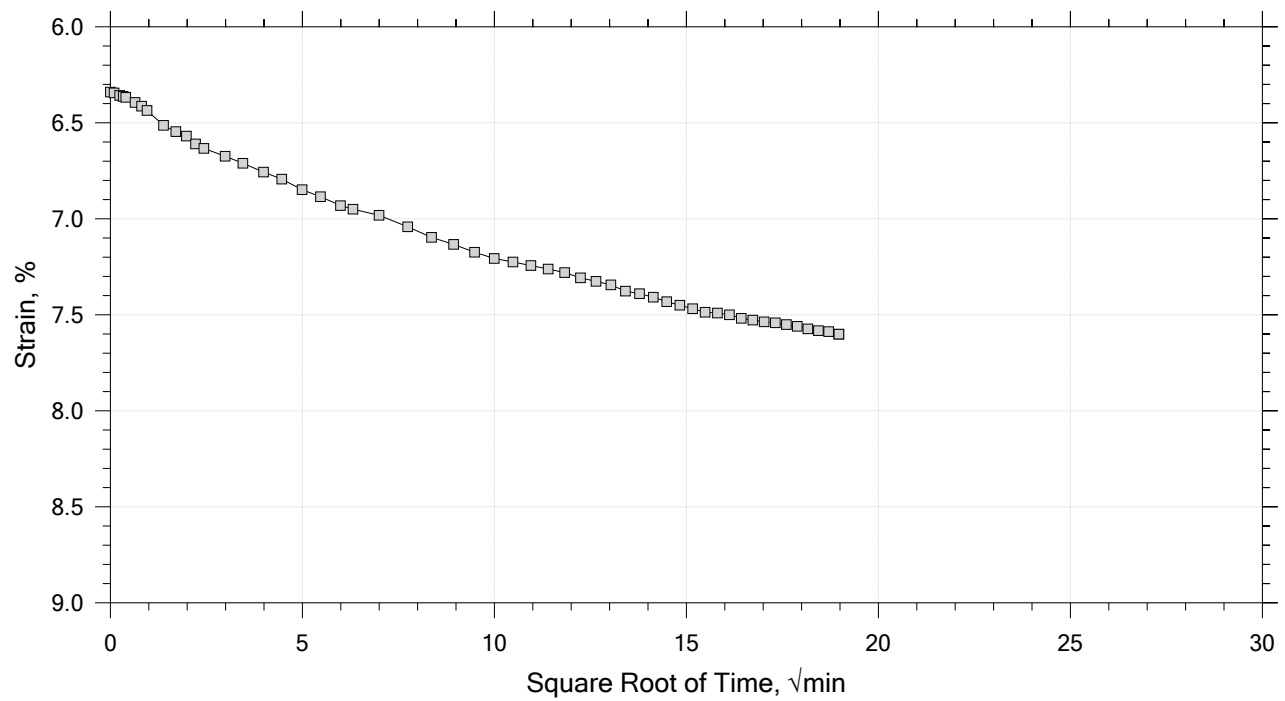
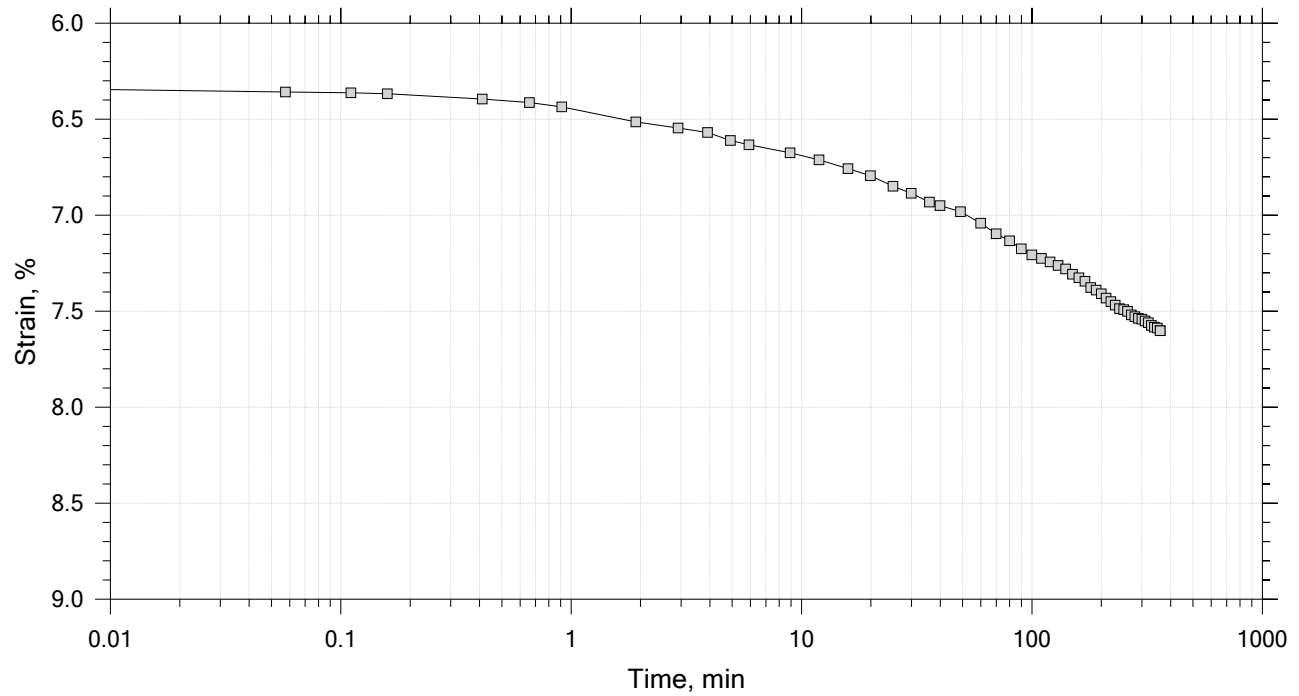
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 17

Constant Load Step

Stress: 2.5 tsf



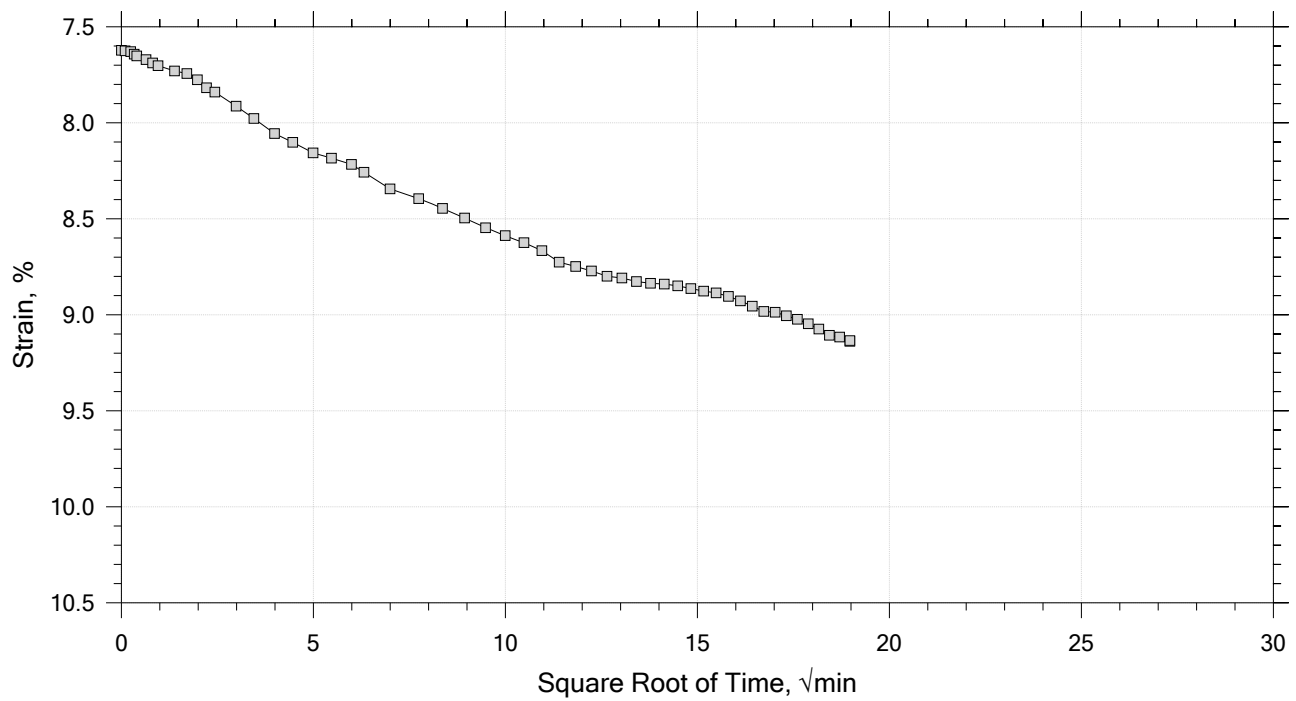
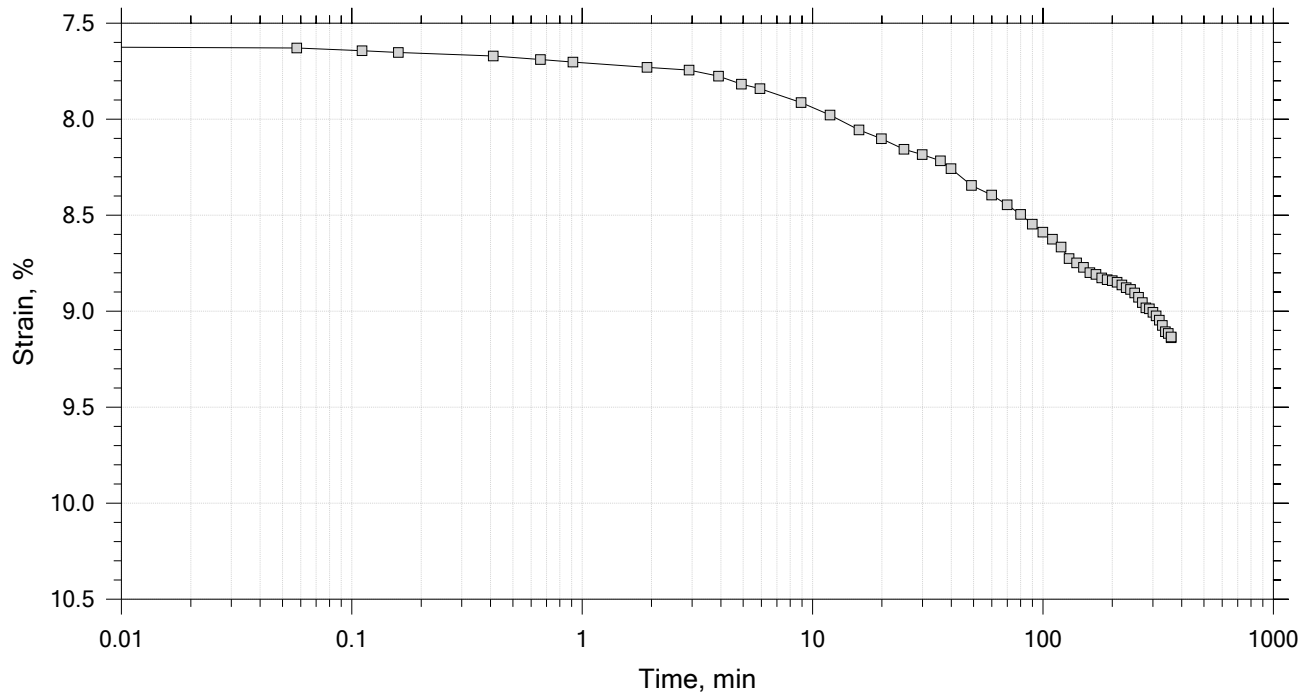
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	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 17

Constant Load Step

Stress: 3 tsf



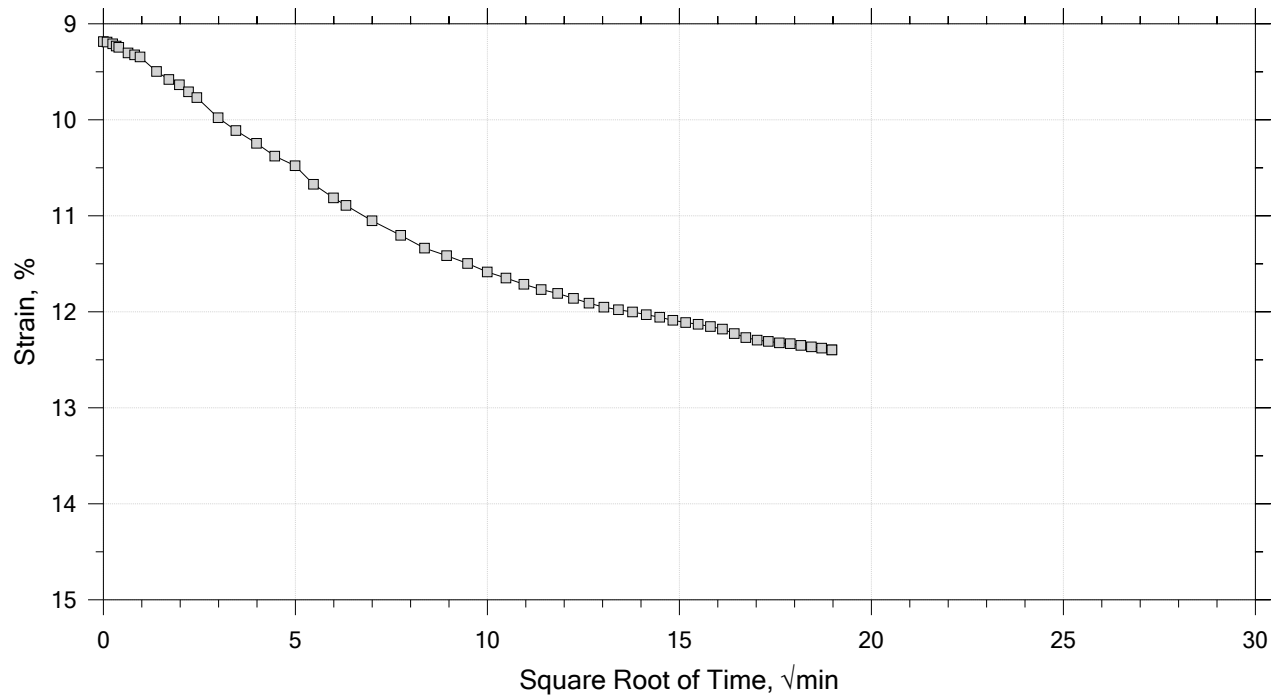
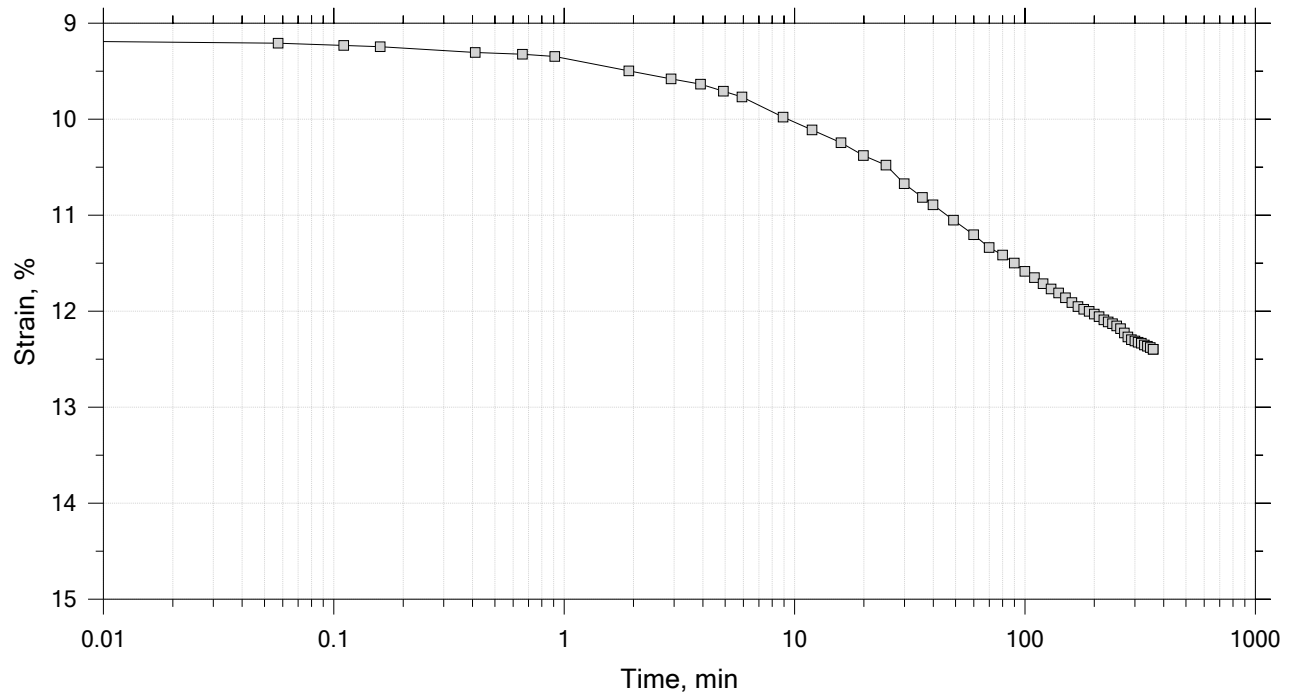
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	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 17

Constant Load Step

Stress: 4 tsf



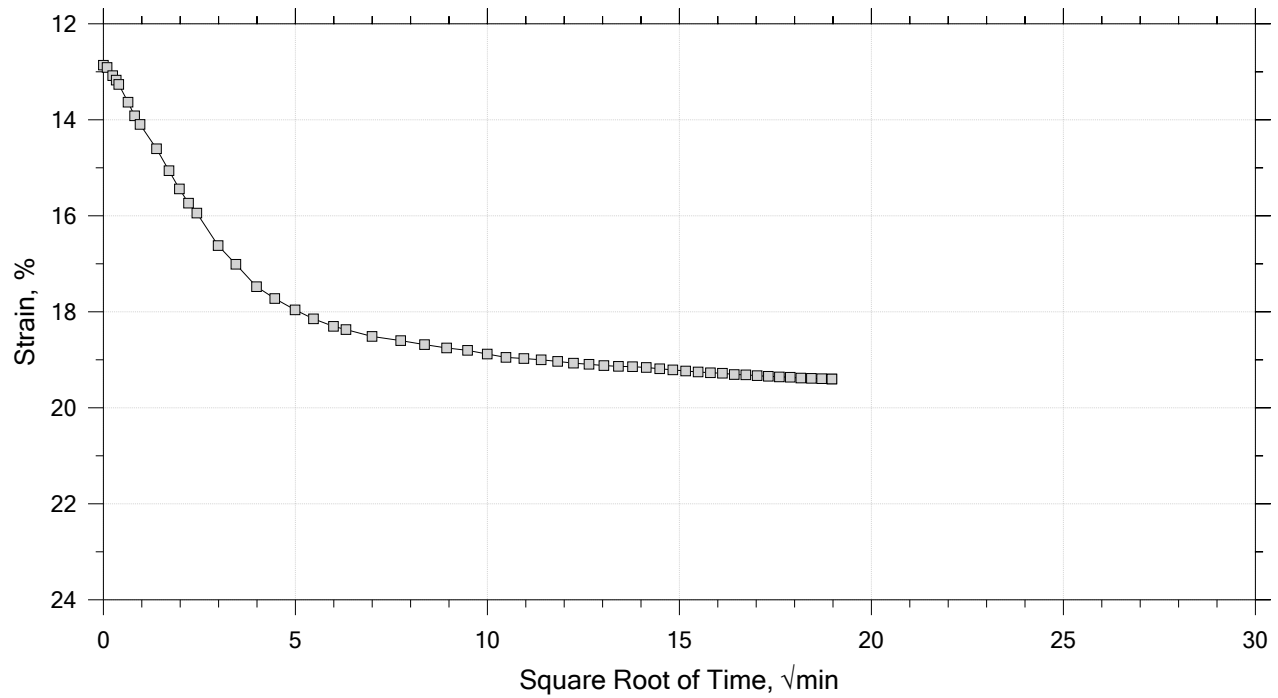
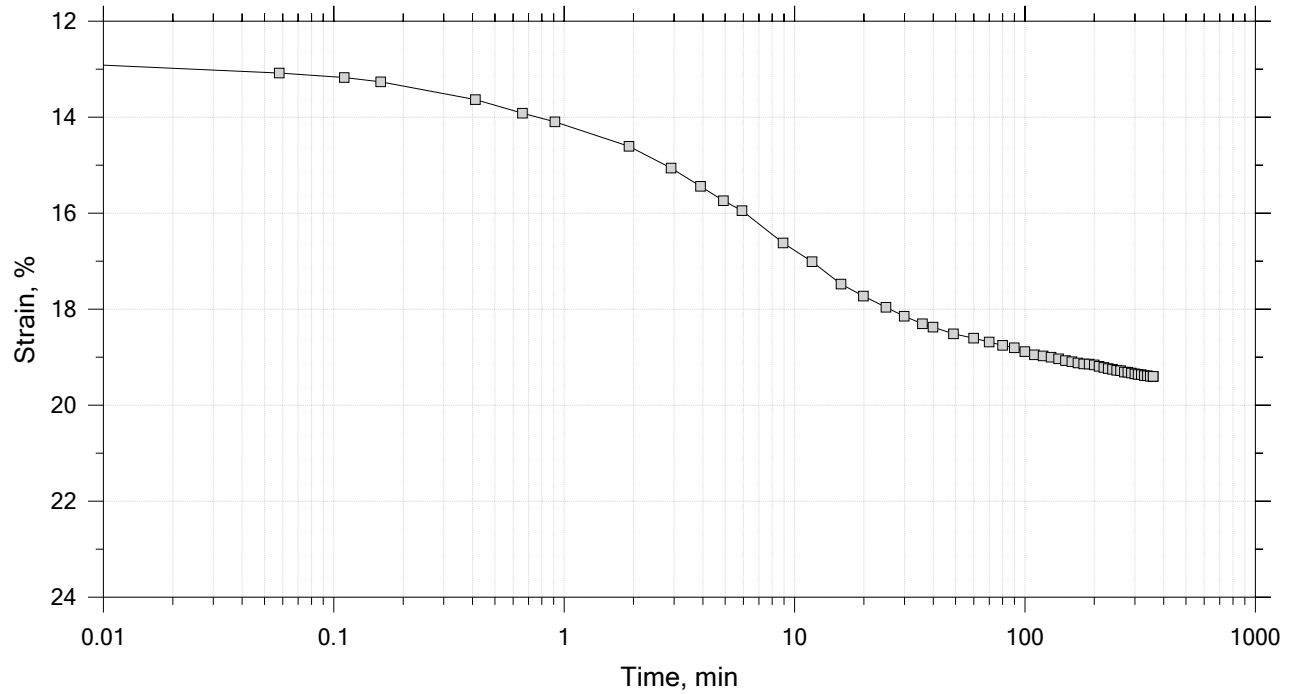
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 17

Constant Load Step

Stress: 8 tsf



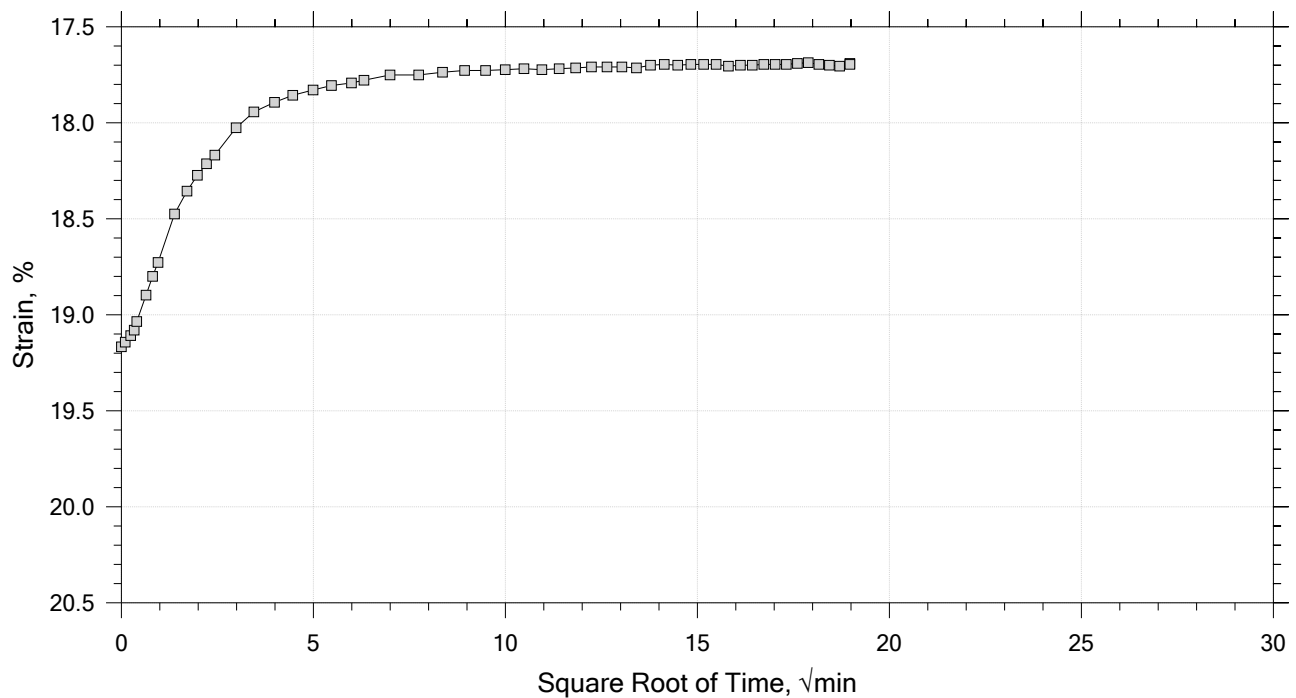
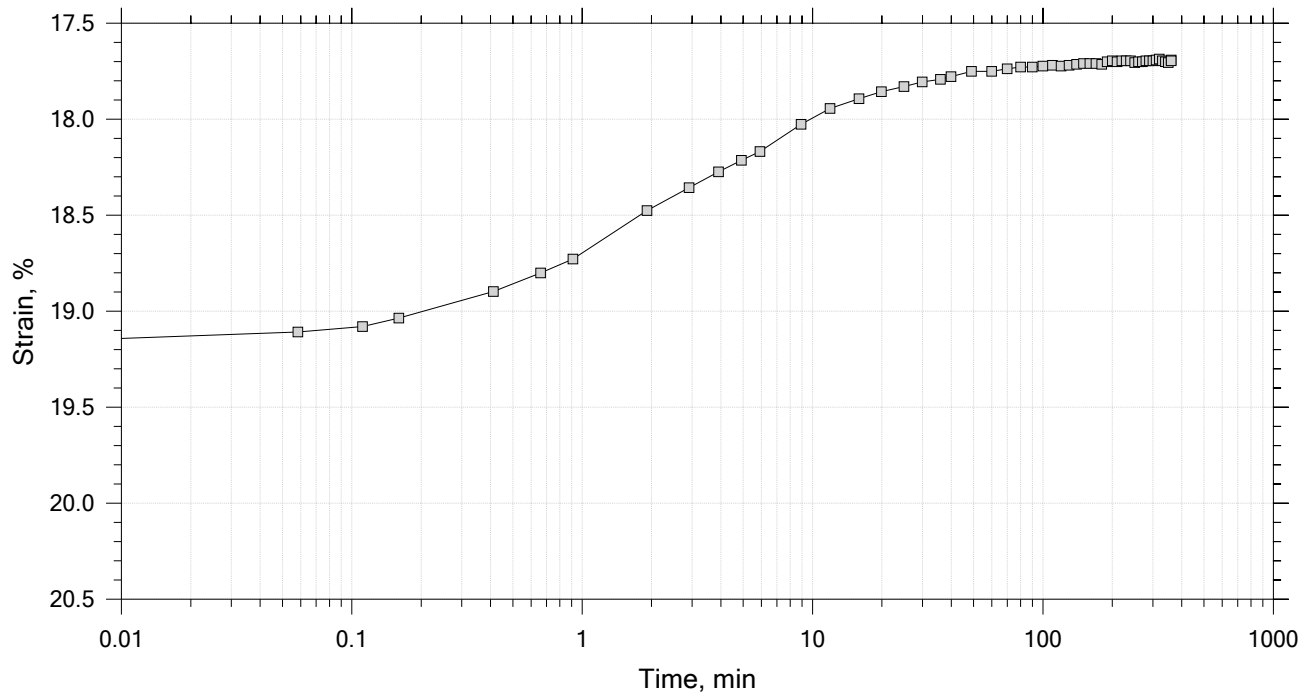
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 17

Constant Load Step

Stress: 1 tsf



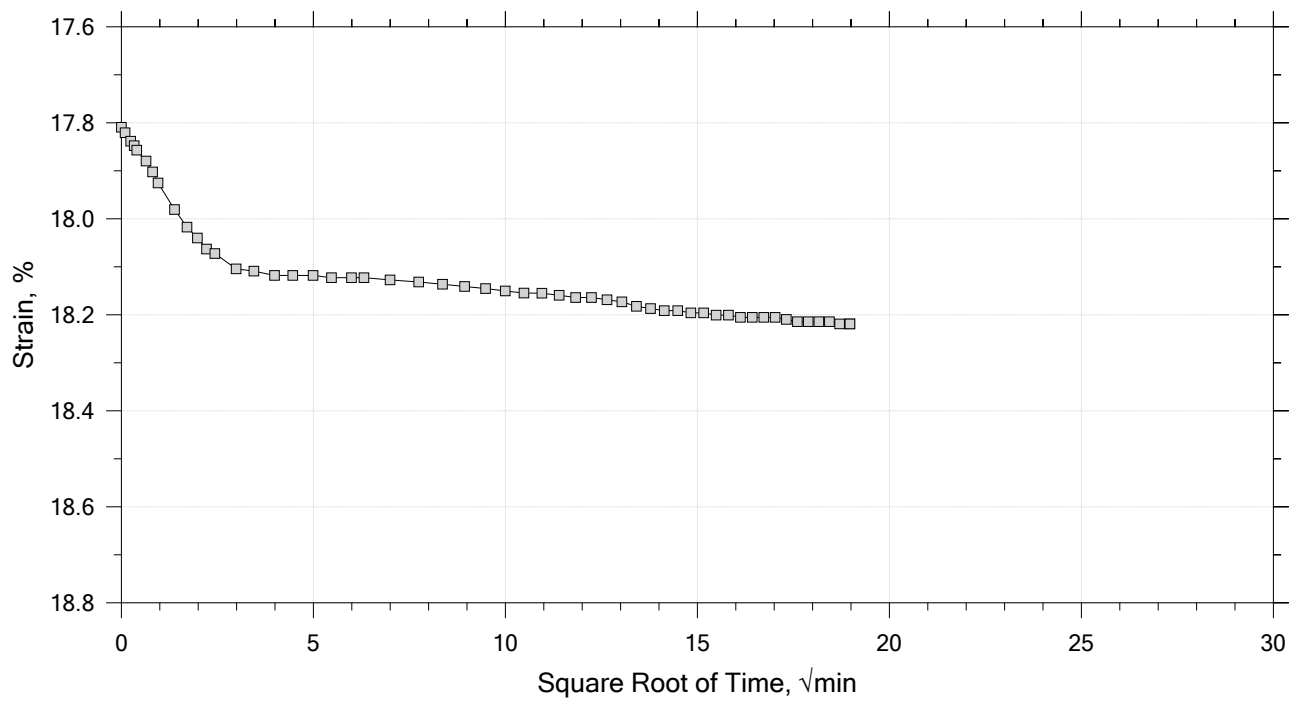
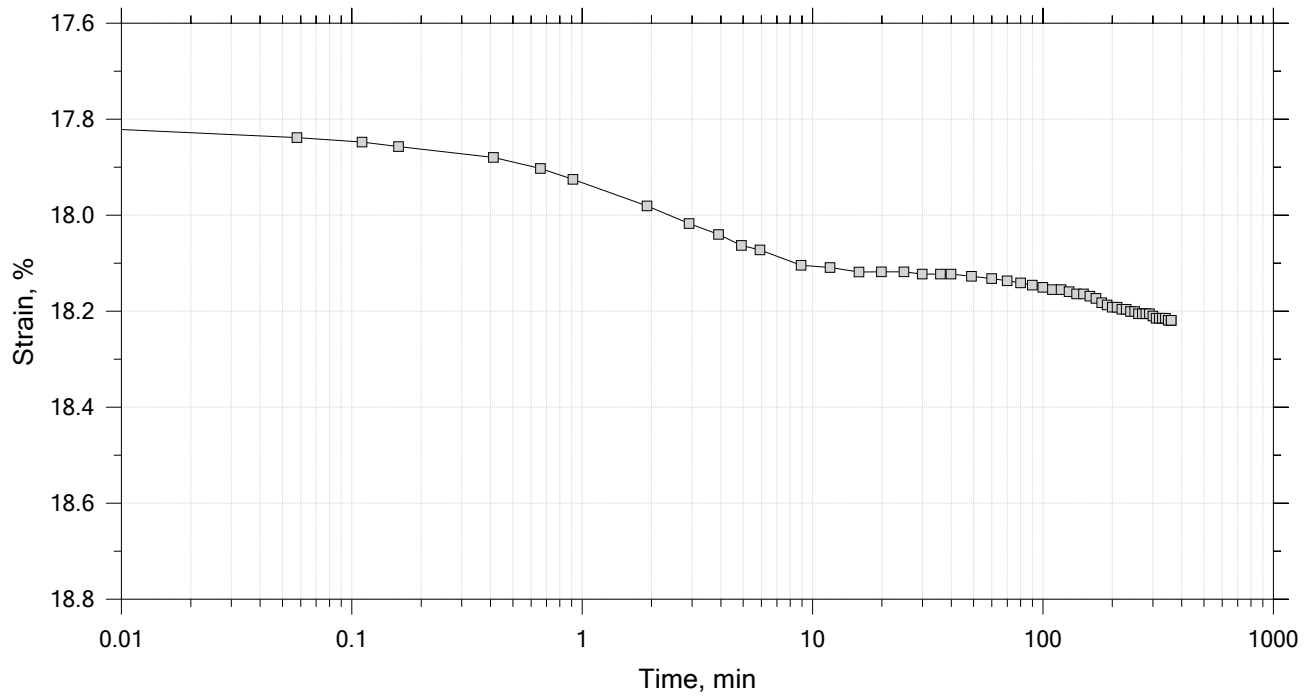
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 17

Constant Load Step

Stress: 2 tsf



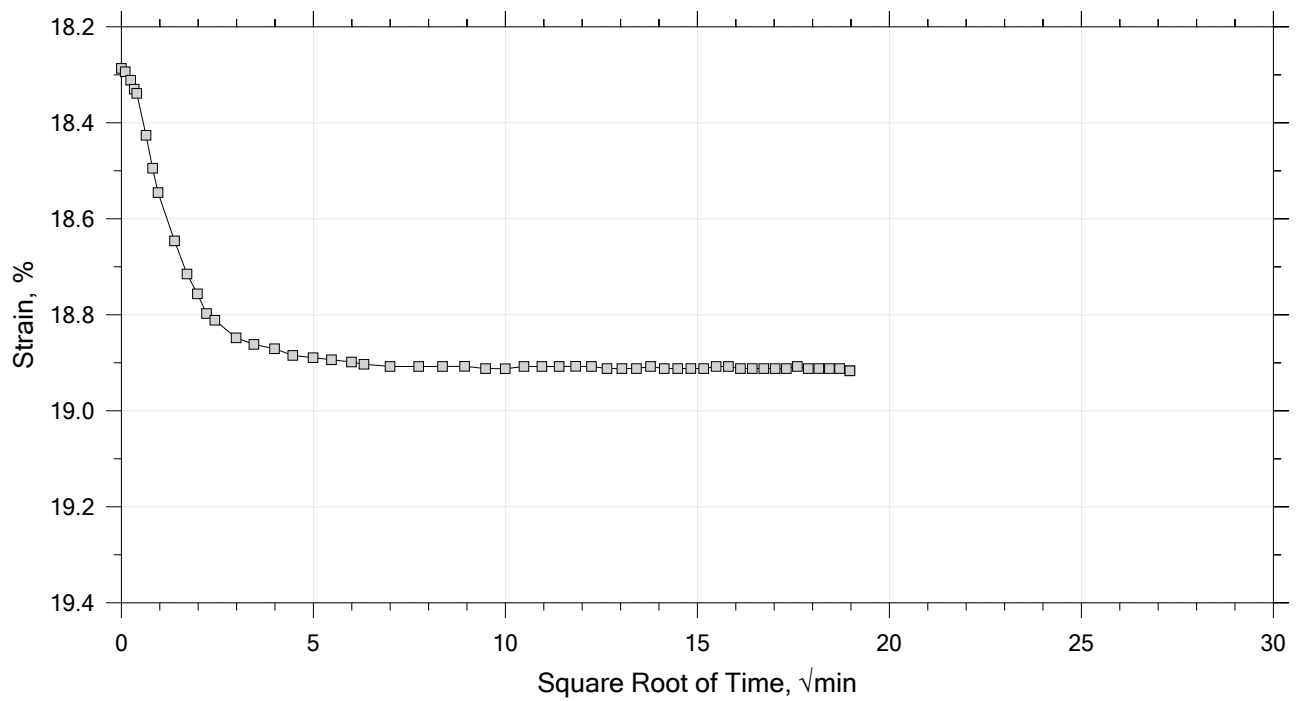
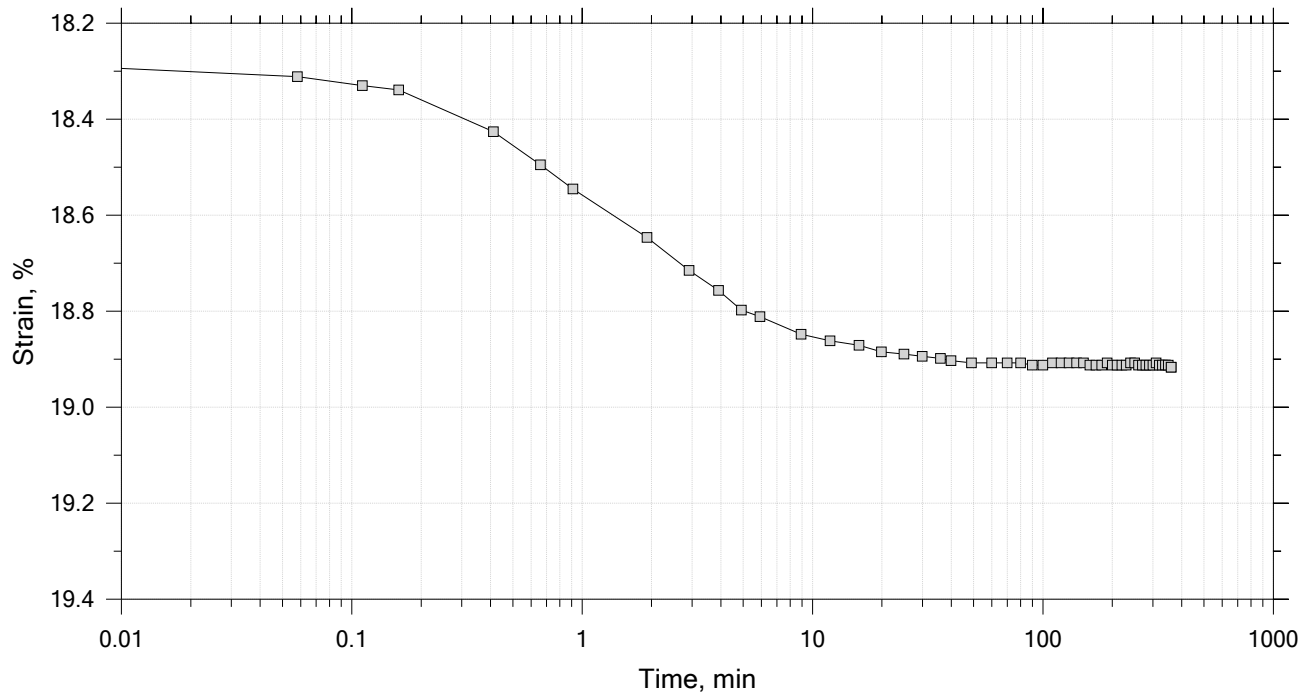
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 17

Constant Load Step

Stress: 4 tsf



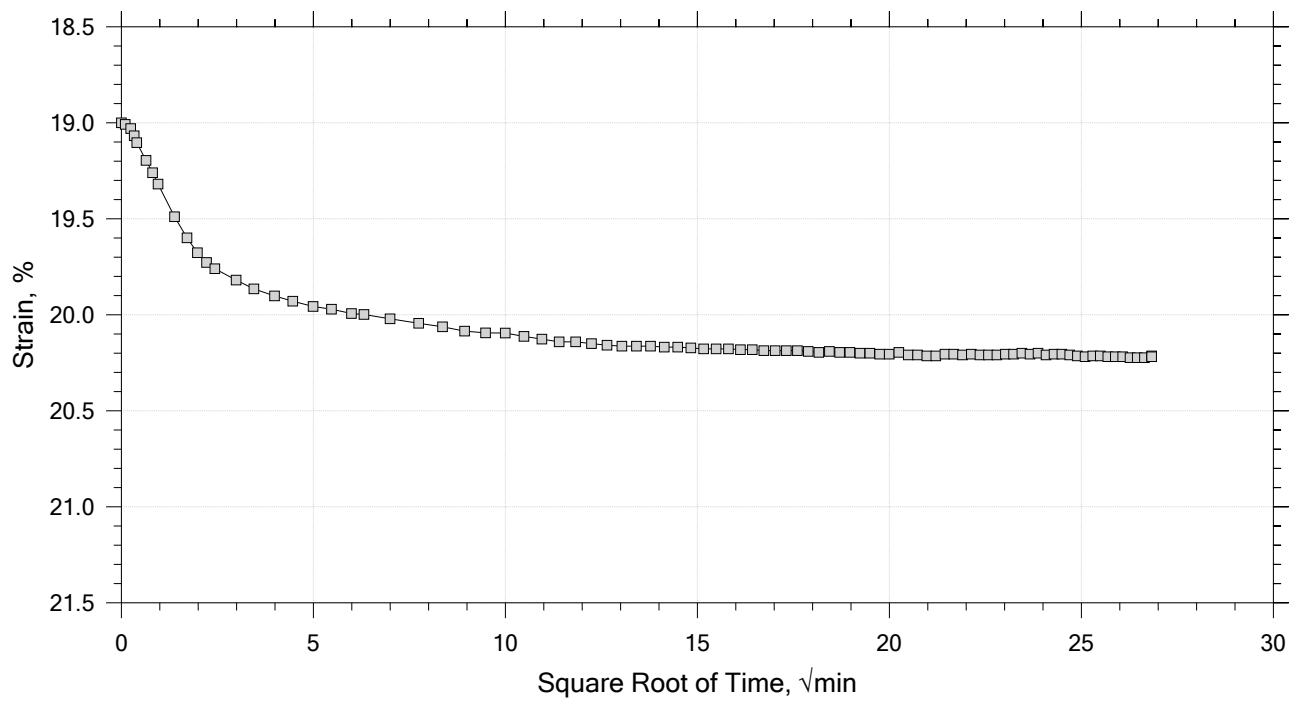
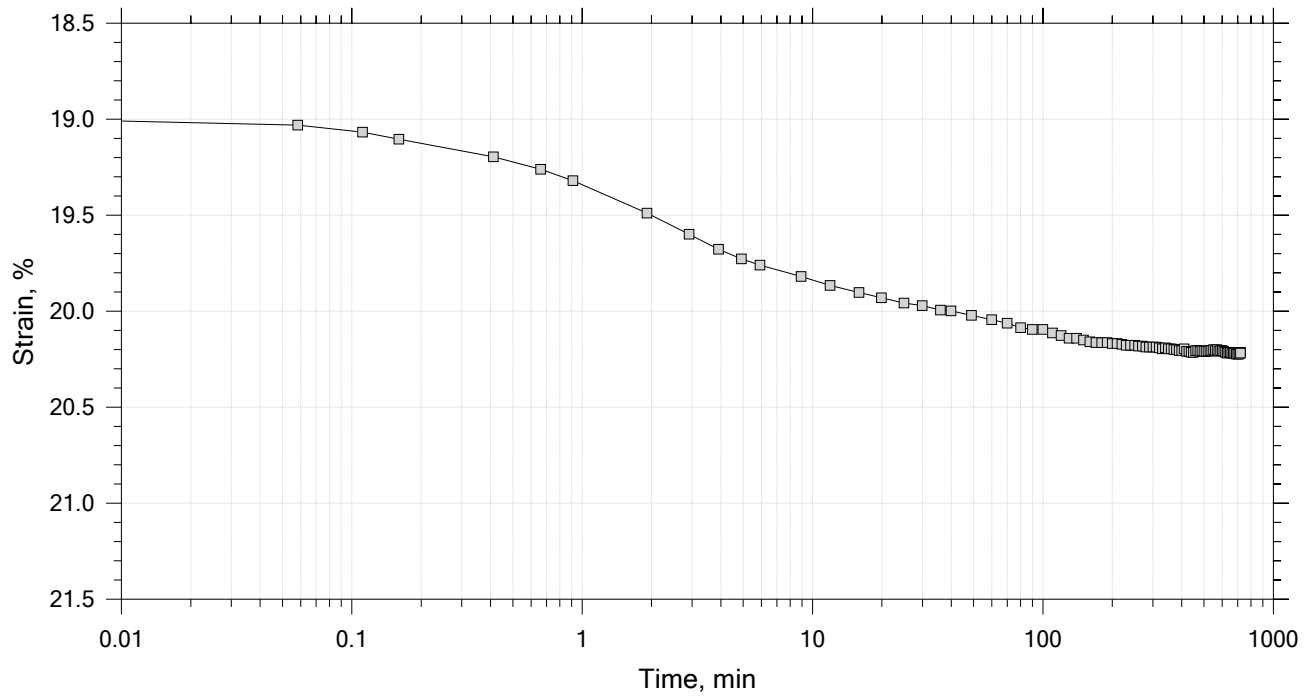
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 17

Constant Load Step

Stress: 7.5 tsf



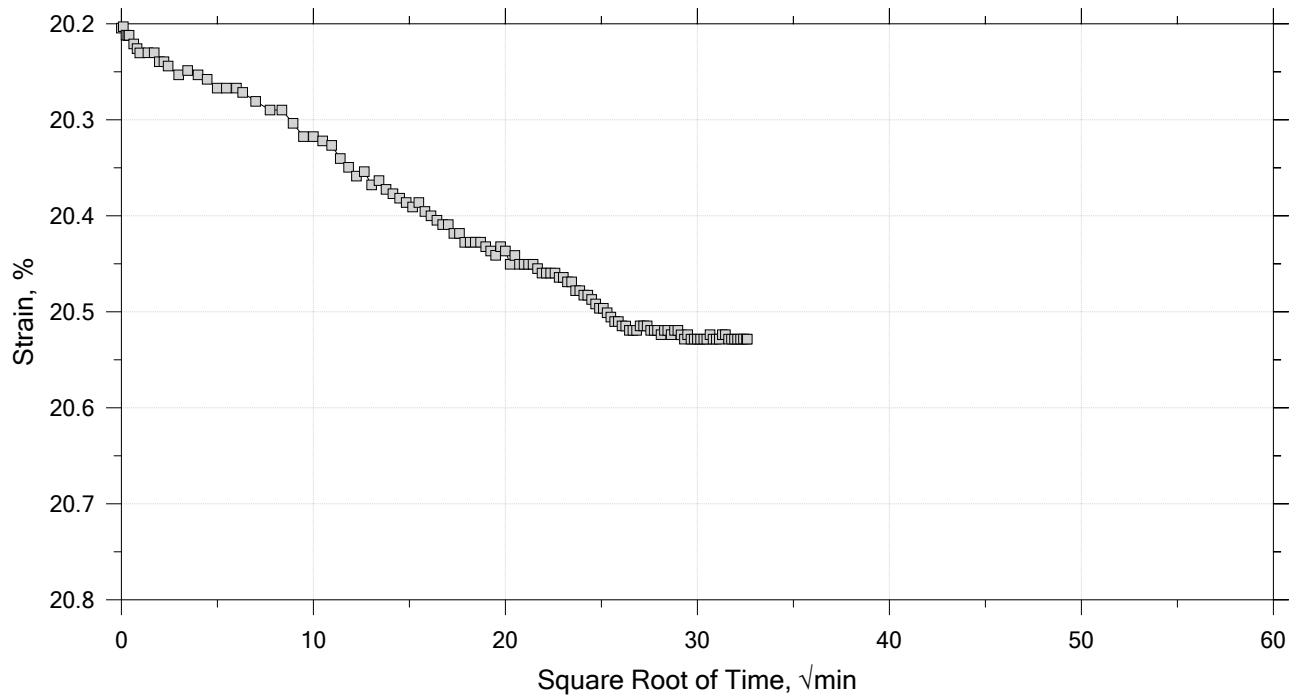
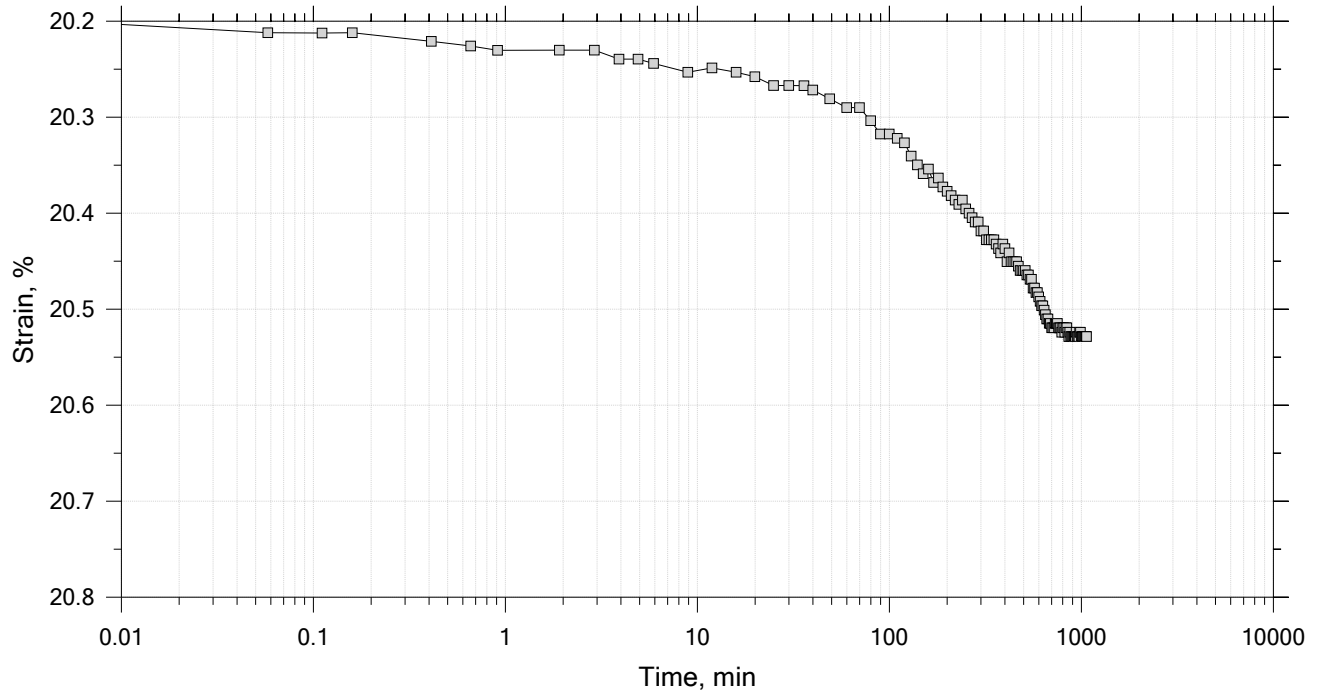
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 16 of 17

Constant Load Step

Stress: 8 tsf



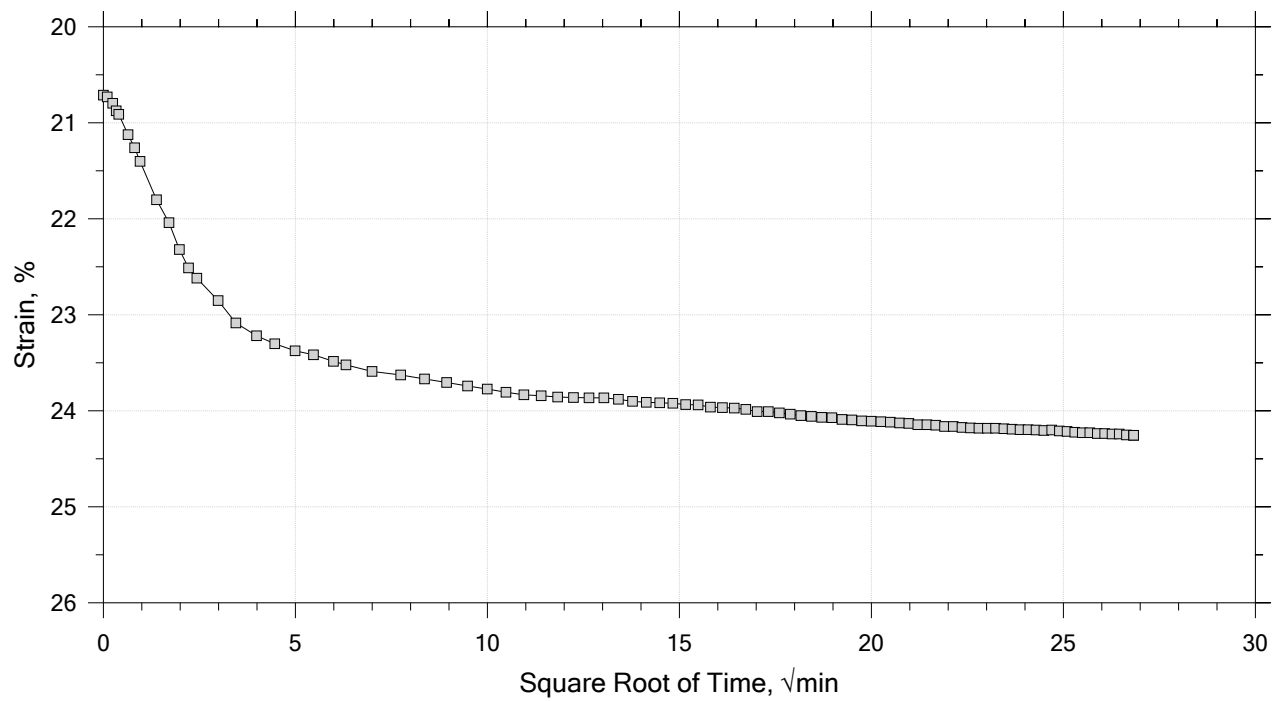
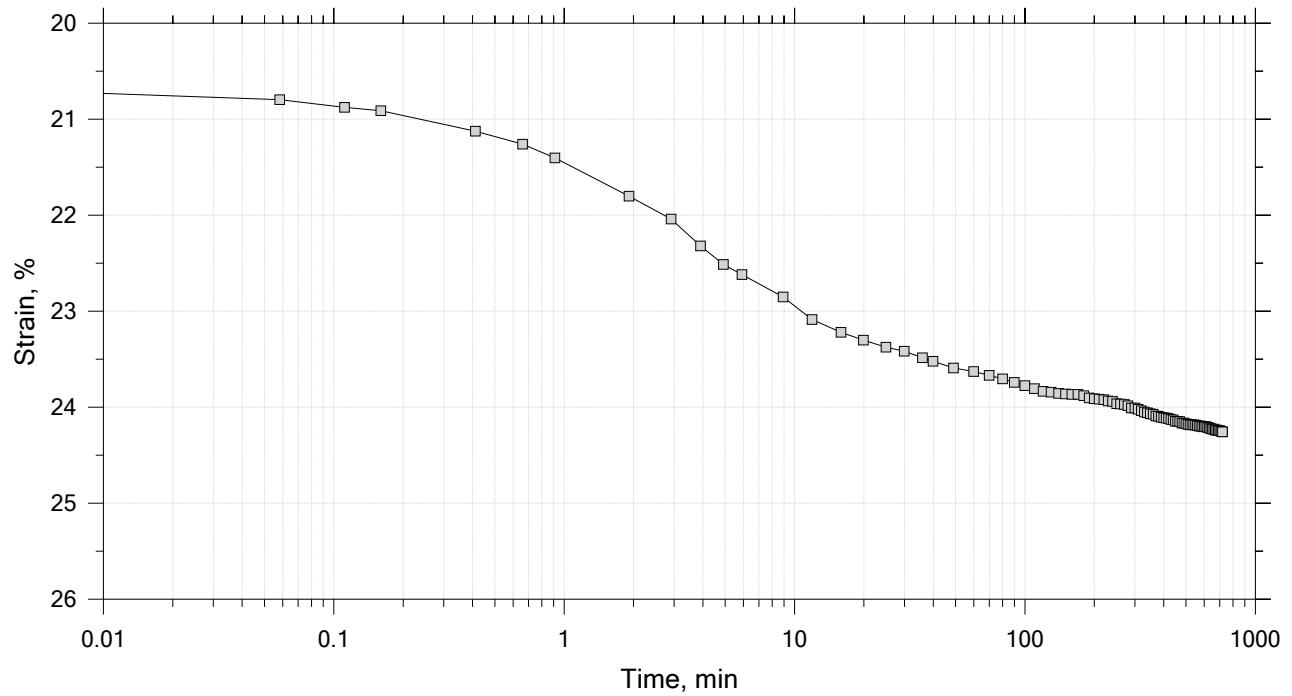
	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 17 of 17

Constant Load Step

Stress: 16 tsf



	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.76	Liquid Limit: 38
Initial Height: 1.00 in	Initial Void Ratio: 1.14	Plastic Limit: 22
Final Height: 0.83 in	Final Void Ratio: 0.773	Plasticity Index: 16

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-2553	RING		A1521
Mass Container, gm	8.2	110.17	110.17	8.34
Mass Container + Wet Soil, gm	110.56	256.75	243.09	138.83
Mass Container + Dry Soil, gm	80.02	213.98	213.98	110.25
Mass Dry Soil, gm	71.82	103.81	103.81	101.91
Water Content, %	42.52	41.20	28.04	28.04
Void Ratio	---	1.14	0.77	---
Degree of Saturation, %	---	99.97	100.00	---
Dry Unit Weight, pcf	---	80.563	97.064	---

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: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		

One-Dimensional Consolidation by ASTM D2435 - Method B

Log of Time Coefficients

[illegible]

	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Square Root of Time Coefficients

[illegible]

	Project: Little River Bridge No. 3112	Location: Gorham, ME	Project No.: GTX-308833
	Boring No.: BB-GLR-102	Tested By: md	Checked By: njh
	Sample No.: 2U	Test Date: 09/26/18	Depth: 40-42 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Wet, very dark greenish gray clay		
	Remarks: System S		
	Displacement at End of Increment		



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX F – ENGINEERING CALCULATIONS

Global Stability



GZA
GeoEnvironmental, Inc
477 Congress Street, Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

JOB: 09.0025988.01
SUBJECT: Little River Bridge Stability Eval
SHEET: 1 OF 3
CALCULATED BY NVW 9-20-19
CHECKED BY ARB 9-21-19

Objective

Evaluate global stability of the proposed approach embankments at critical cross sections near the proposed abutments and in the longitudinal direction beneath each abutment using the profile and properties shown in the Interpretive Subsurface Profile and Design Profile and Cross Sections for the Little River Bridge replacement.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017 with interims. (AASHTO LRFD)
2. Hynes-Griffin and Franklin (1984), "Rationalizing the seismic coefficient method," Miscellaneous Paper GL-84-13, U.S. Army Corps of Engineers Waterway Experiment Station, Vicksburg, Mississippi.
3. Kramer (1996), "Geotechnical Earthquake Engineering," Prentice-Hall, Inc., Upper Saddle River, NY.

Analysis

Soil Properties

Friction angles were developed for granular materials based on corrected N60 blow counts of existing materials or anticipated/MaineDOT BDG design properties of new fills. Shear strengths and unit weights of marine clay were developed based on laboratory testing, field vane testing, and CPT correlations, and are shown on Figures 3 and 4. Design soil properties are shown on the Slope/W output.

Stability Calculation and Performance Criteria

To consider the longitudinal and cross-section analyses, the minimum factor of safety for global stability is selected as 1.5 and 1.3 respectively. These correspond to approximate resistance factors of 0.65 and 0.75, for a slope that supports structures and for a slope that does not contain or support a structural element, as specified in AASHTO LRFD Article 11.6.2.3.

Pseudostatic Analysis

The minimum factor of safety for global stability is selected as 1.0, which is recommended by Hynes-Griffin and Franklin (1984), referenced in Geotechnical Earthquake Engineering (1996). The pseudostatic analysis includes an additional earthquake load by incorporating the horizontal seismic coefficient, k_h , as defined in AASHTO 11.6.5.2.2 and recommended by Hynes-Griffin and Franklin using the Newmark method as half of the maximum peak ground acceleration at the ground surface (see seismic analysis). This criterion is considered suitable to limit potential for large deformation of the embankments, which is considered suitable for this Extreme Event scenario.

GZA evaluated the stability of the proposed approach embankments statically, using the computer analytical software *Slope/W 2018*, developed by Geo-Slope International, based on the Modified Bishop method.



GZA
GeoEnvironmental, Inc
477 Congress Street, Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

*Engineers and
Scientists*

JOB: 09.0025988.01
SUBJECT: Little River Bridge Stability Eval
SHEET: 2 OF 3
CALCULATED BY NVW 9-20-19
CHECKED BY ARB 9-21-19

Longitudinal Analysis

The analyzed profiles considered the interpreted typical subsurface conditions along the project baseline with the beneficial reinforcing effect of timber piles supporting the existing abutments and seven (7) new HP14x89 piles spaced 5.7 feet oriented for strong axis bending, based on Brom's method for pile resistance shown in the attached calculations. The pile lateral resistance from Broms method was applied in Slope/W as a reinforcing load at the spacing stated above. The proposed embankment at the abutments consist of the travelway with a traffic surcharge placed over the existing fill with a proposed slope of 2H:1V with 3 foot thick rip-rap cover.

The attached Slope-W outputs show that the proposed embankment provides the following minimum global factors of safety in the longitudinal direction:

<u>Abutment</u>	<u>Analysis</u>	<u>Factor of Safety Against Rotation</u>	<u>Required Factor of safety</u>
1	Static	1.5	1.5
1	Pseudostatic	1.1	1.0
2	Static	1.8	1.5

Cross Section Analysis

The highest embankment and/or steepest side slope inclinations in proximity to each abutment were selected for analysis. The analyzed profiles considered the typical subsurface conditions at the respective abutments. A Slope-W model was developed on a 20 foot wide travelway with a traffic surcharge load placed over the existing fill, side slopes with a 3 foot thick rip-rap cover, and the strata thicknesses based the following explorations for each abutment:

Abutment 1 BB-GLR-101, BB-GLR-201, and B-4 - at a height of 28' and side slope 1.75H:1V
Abutment 2 BB-GLR-102 and CPT-GLR-102 - at a height of 16' and a side slope of 2H:1V

The attached Slope-W outputs show that the proposed embankment provides the following minimum global factors of safety :

<u>Abutment</u>	<u>Analysis</u>	<u>Factor of Safety Against Rotation</u>	<u>Required Factor of safety</u>
1	Static	1.0	1.3
2	Static	1.6	1.3

Based on the results, global stability is not suitable at Abutment 1, but is suitable for Abutment 2. Therefore the Abutment 1 area was evaluated for remedial alternatives to improve global stability. A four (4) pile configuration spaced 12 feet on center was selected for HP14x89 piles oriented for strong-axis bending. The attached Slope-W output shows that the proposed embankment with slope mitigation provides the following minimum global factors of safety:

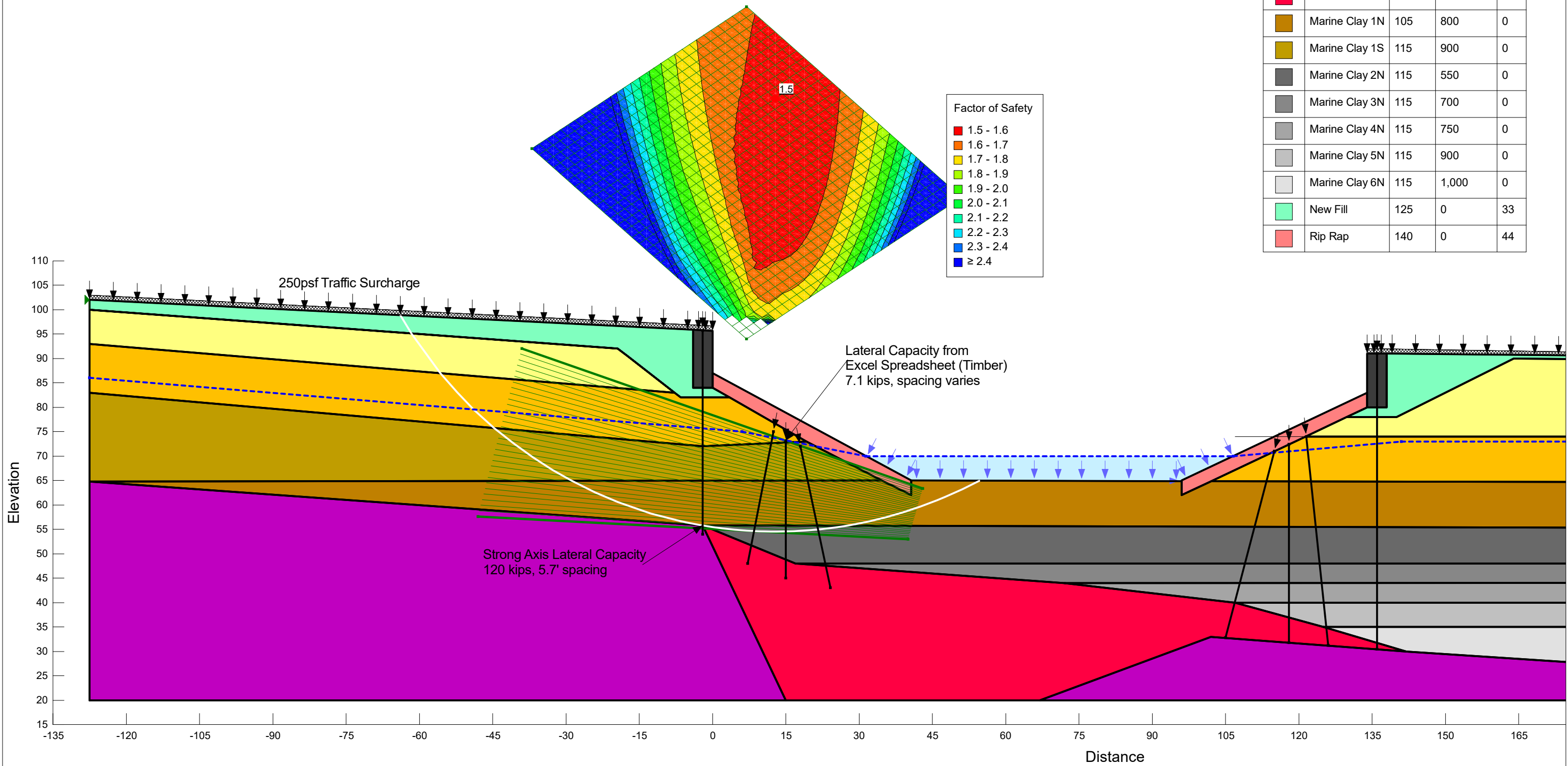
<u>Abutment</u>	<u>Analysis</u>	<u>Factor of Safety Against Rotation</u>	<u>Required Factor of safety</u>
1	Static	1.3	1.3
1	Pseudostatic	1.0	1.0

Conclusion

The global stability meets the minimum required criteria for both static and pseudostatic conditions, under the condition that the global stability mitigation is installed at Abutment 1.

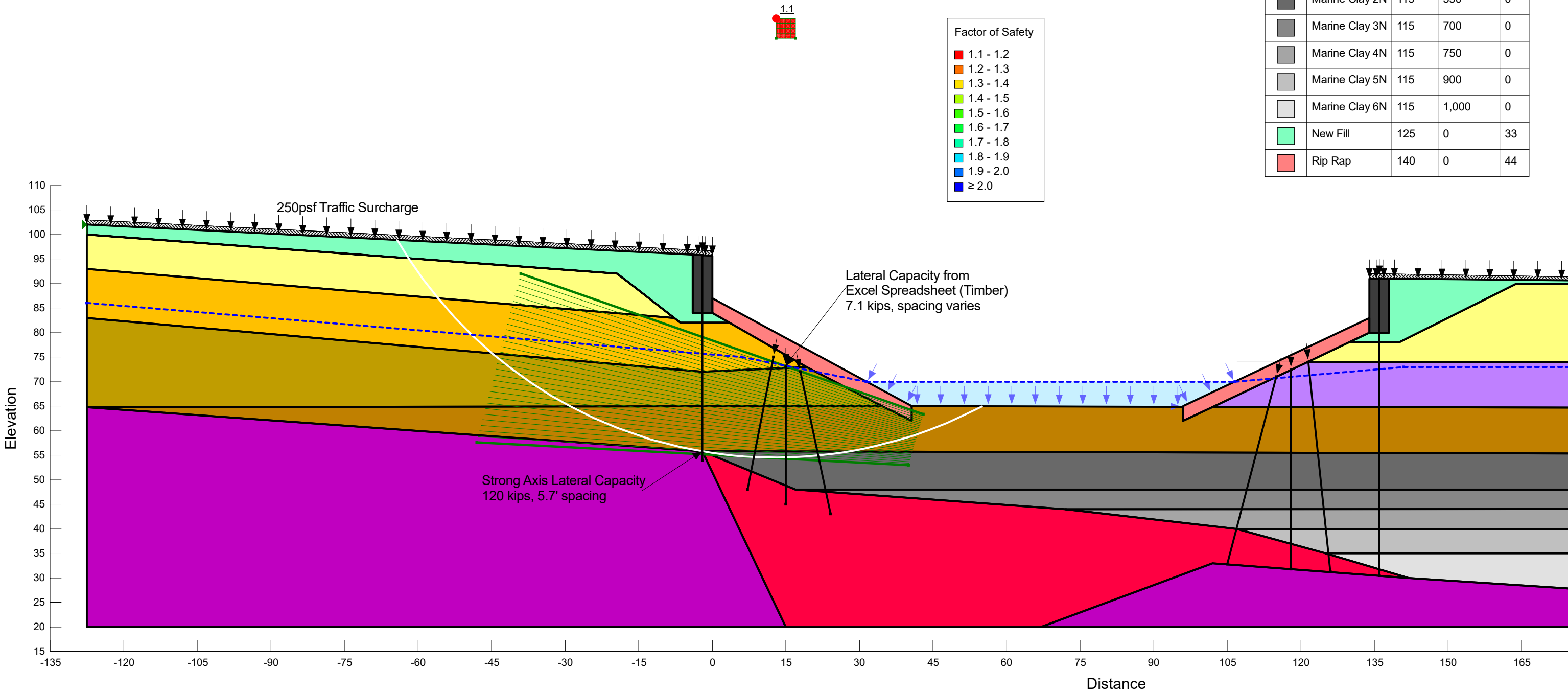
Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
South Approach, South to North Profile Evaluation with HP 14X89 Piles spaced 5.7
GZA File No. 09.0025988.01

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Abutment	150		
	Bedrock			
	Ex Fill (Sand)	125	0	34
	Ex Fill (Sandy Clay, Lean Clay)	115	400	0
	Lower Sand	125	0	34
	Marine Clay 1N	105	800	0
	Marine Clay 1S	115	900	0
	Marine Clay 2N	115	550	0
	Marine Clay 3N	115	700	0
	Marine Clay 4N	115	750	0
	Marine Clay 5N	115	900	0
	Marine Clay 6N	115	1,000	0
	New Fill	125	0	33
	Rip Rap	140	0	44



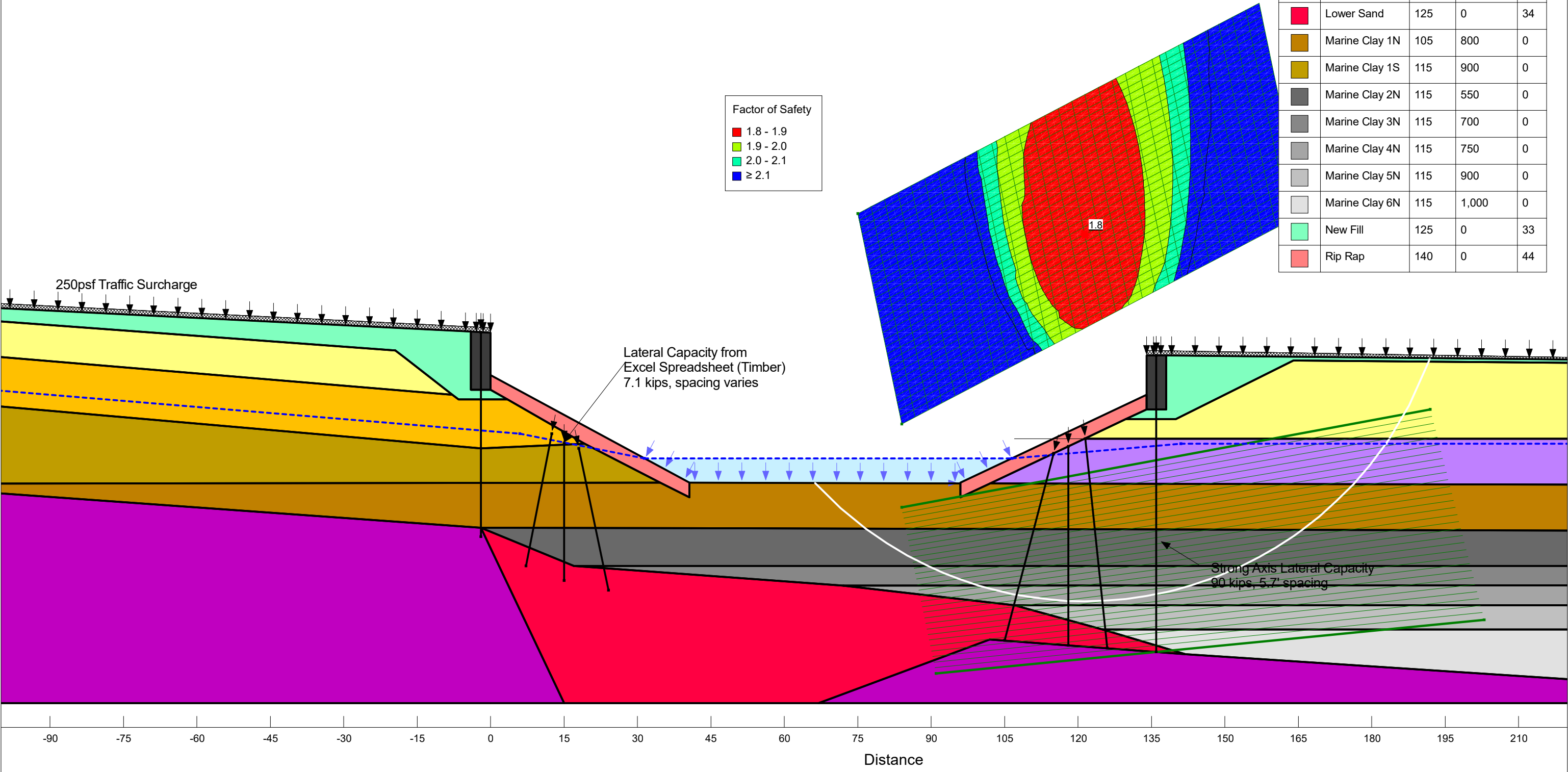
Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
Pseudostatic Analysis
South Approach, South to North Profile Evaluation with HP 14X89 Piles spaced 5.7
GZA File No. 09.0025988.01

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Abutment	150		
	Bedrock			
	Ex Fill (Clayey Sand)	110	0	28
	Ex Fill (Sand)	125	0	34
	Ex Fill (Sandy Clay, Lean Clay)	115	400	0
	Lower Sand	125	0	34
	Marine Clay 1N	105	800	0
	Marine Clay 1S	115	900	0
	Marine Clay 2N	115	550	0
	Marine Clay 3N	115	700	0
	Marine Clay 4N	115	750	0
	Marine Clay 5N	115	900	0
	Marine Clay 6N	115	1,000	0
	New Fill	125	0	33
	Rip Rap	140	0	44



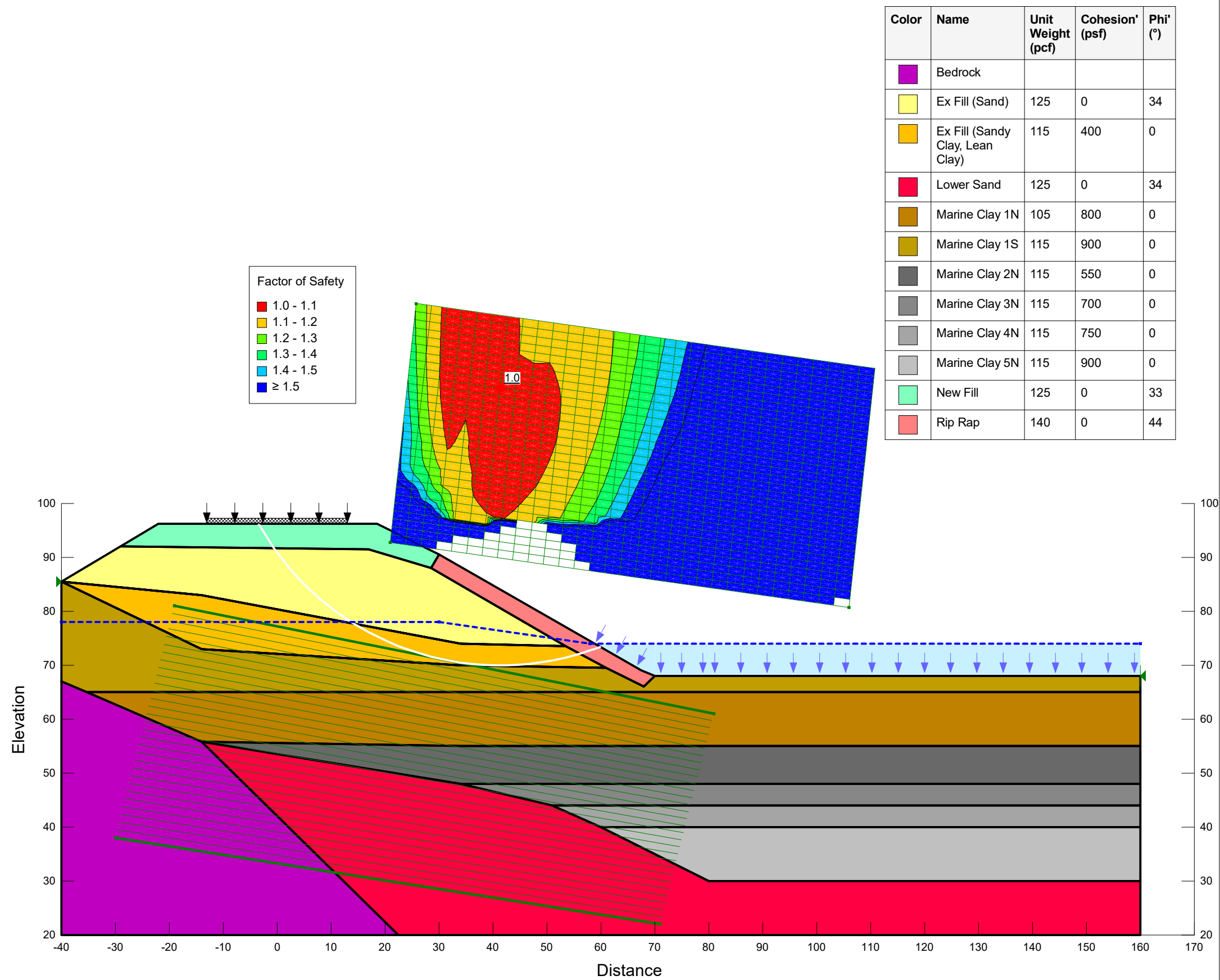
Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
North Approach Evaluation with HP 14X89 Piles spaced 5.7'
GZA File No. 09.0025988.01

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Abutment	150		
	Bedrock			
	Ex Fill (Sand)	125	0	34
	Ex Fill (Sandy Clay - Abut 2)	110	0	28
	Ex Fill (Sandy Clay, Lean Clay)	115	400	0
	Lower Sand	125	0	34
	Marine Clay 1N	105	800	0
	Marine Clay 1S	115	900	0
	Marine Clay 2N	115	550	0
	Marine Clay 3N	115	700	0
	Marine Clay 4N	115	750	0
	Marine Clay 5N	115	900	0
	Marine Clay 6N	115	1,000	0
	New Fill	125	0	33
	Rip Rap	140	0	44



Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
South Approach, 1.75H:1V Grade
GZA File No. 09.0025988.01

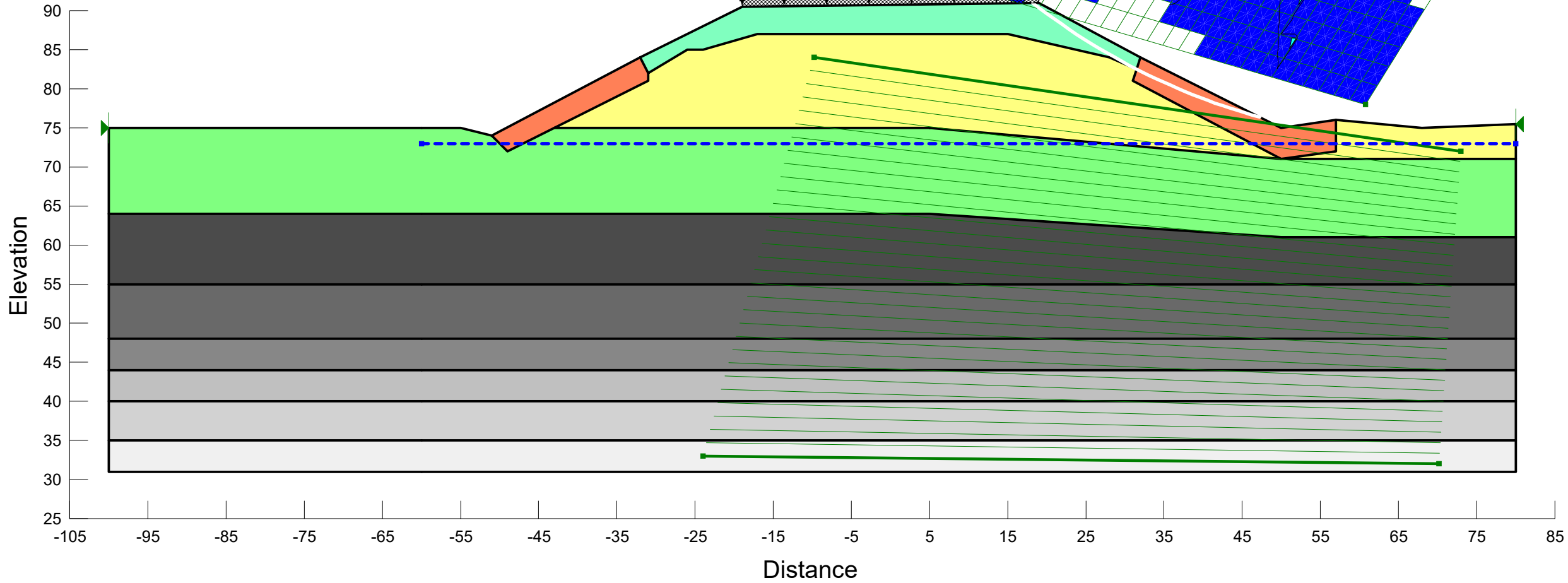
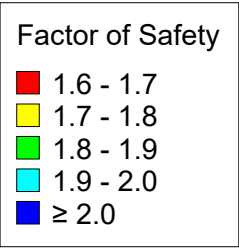
South Approach - Abutment 1 Global Stability



Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
North Approach Cross Section

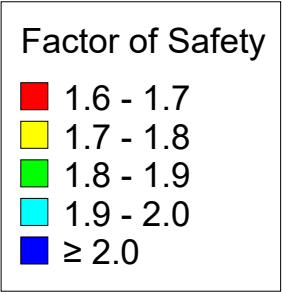
North Approach - Abutment 2 Global Stability
Slip Surface Left to Right

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clay 1N	105	800	0
	Clay 2N	115	550	0
	Clay 3N	115	700	0
	Clay 4N	115	750	0
	Clay 5N	115	900	0
	Clay 6N	115	1,000	0
	Clayey Sand	110	0	28
	Fill (Sand)	125	34	34
	New Fill	125	0	33
	Rip Rap	140	0	44

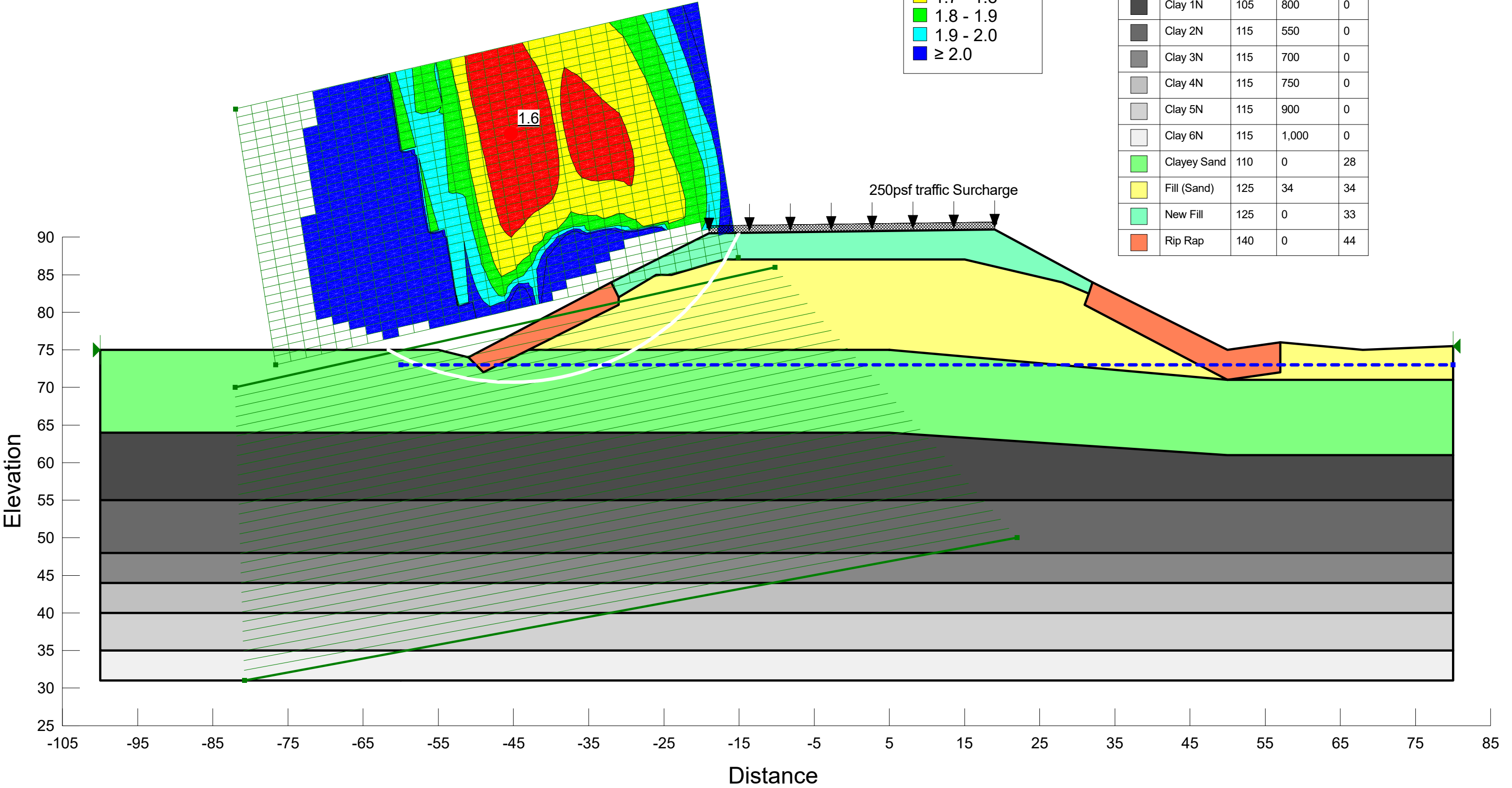


Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
North Approach Cross Section

North Approach - Abutment 2 Global Stability
Slip Surface Right to Left



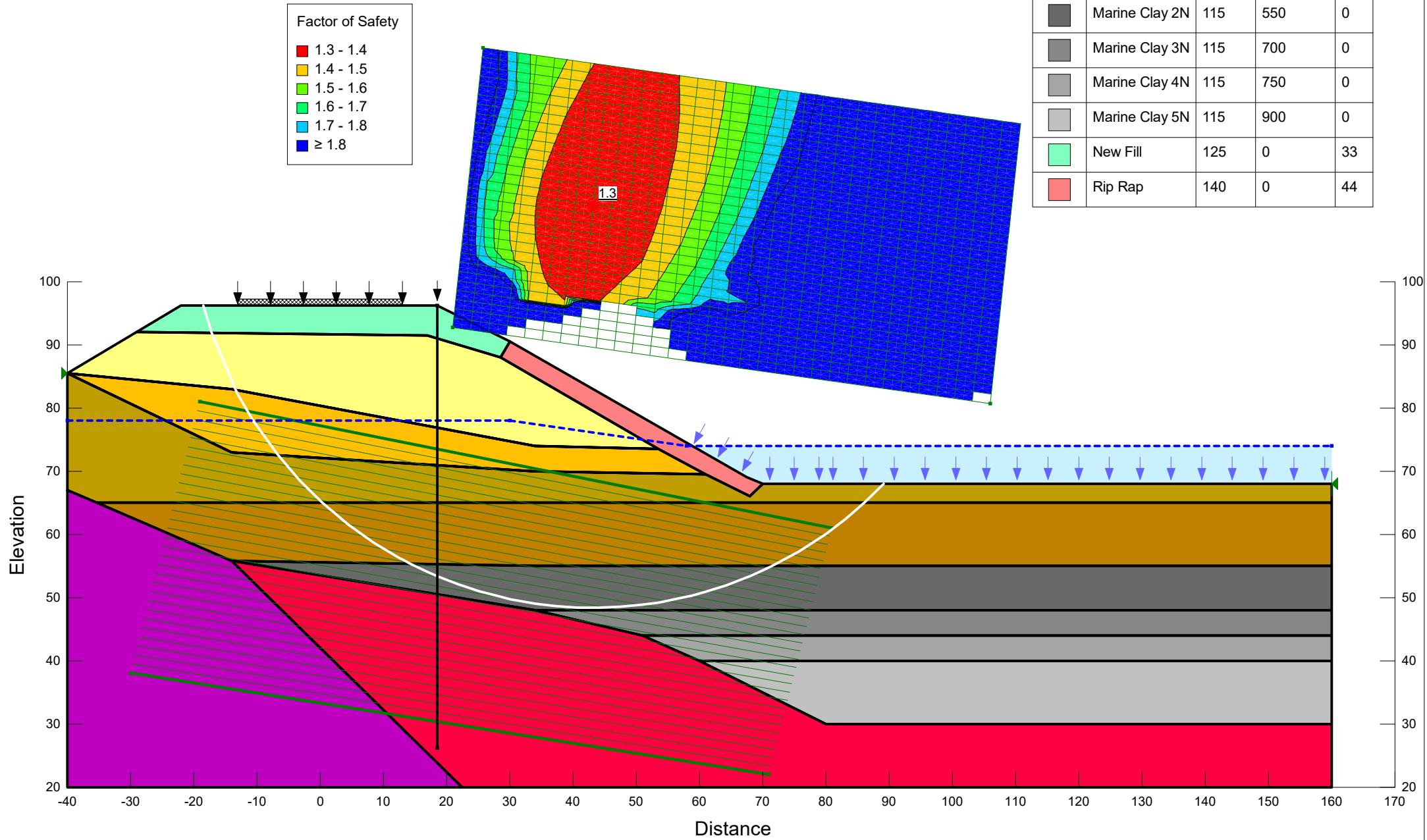
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clay 1N	105	800	0
	Clay 2N	115	550	0
	Clay 3N	115	700	0
	Clay 4N	115	750	0
	Clay 5N	115	900	0
	Clay 6N	115	1,000	0
	Clayey Sand	110	0	28
	Fill (Sand)	125	34	34
	New Fill	125	0	33
	Rip Rap	140	0	44



Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation
South Approach, 1.75H:1V Grade - HP14X89 Reinforcement Spaced 12.25'
GZA File No. 09.0025988.01

South Approach - Abutment 1 Global Stability
Global Stability Mitigation reinforcement - HP14x89 piles

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock			
	Ex Fill (Sand)	125	0	34
	Ex Fill (Sandy Clay, Lean Clay)	115	400	0
	Lower Sand	125	0	34
	Marine Clay 1N	105	800	0
	Marine Clay 1S	115	900	0
	Marine Clay 2N	115	550	0
	Marine Clay 3N	115	700	0
	Marine Clay 4N	115	750	0
	Marine Clay 5N	115	900	0
	New Fill	125	0	33
	Rip Rap	140	0	44

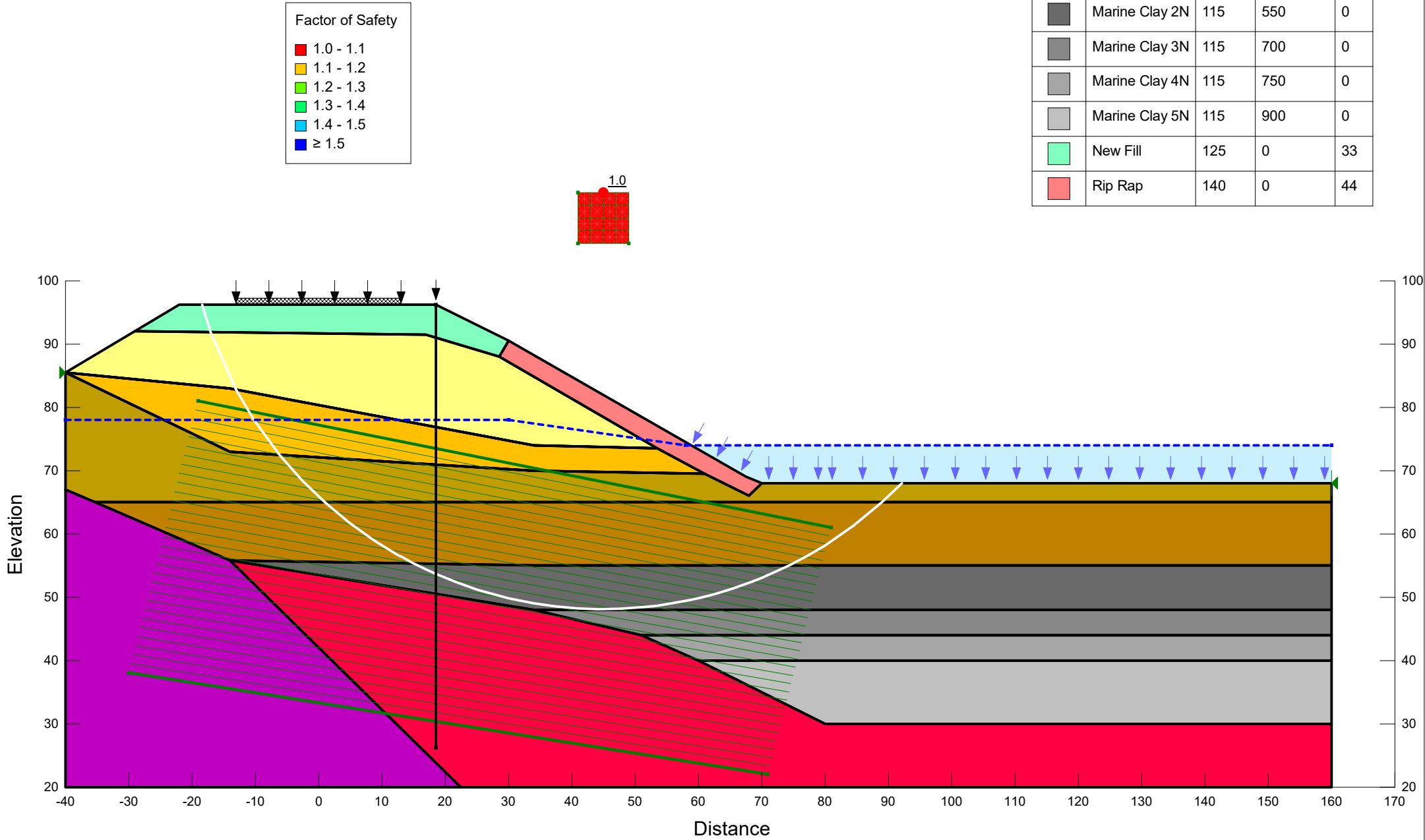


Little River Bridge Replacement, Gorham
Slope/W Global Stability Evaluation - Pseudostatic
South Approach, 1.75H:1V Grade - HP14X89 Reinforcement
GZA File No. 09.0025988.01

Horz Seismic Coef.: 0.114

South Approach - Abutment 1 Global Stability
Global Stability Mitigation reinforcement - HP14x89 piles
Pseudostatic Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock			
	Ex Fill (Sand)	125	0	34
	Ex Fill (Sandy Clay, Lean Clay)	115	400	0
	Lower Sand	125	0	34
	Marine Clay 1N	105	800	0
	Marine Clay 1S	115	900	0
	Marine Clay 2N	115	550	0
	Marine Clay 3N	115	700	0
	Marine Clay 4N	115	750	0
	Marine Clay 5N	115	900	0
	New Fill	125	0	33
	Rip Rap	140	0	44





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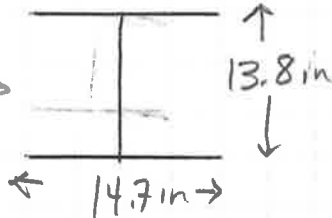
Engineers and
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JOB Gorham Little River Bridge
SHEET NO _____ OF _____
Calculated By EDF Date 7/19/17
Checked By _____ Date _____
Scale _____

Eval Lateral Capacity for New Driven H-piles

HP 14x89

$P_u \rightarrow$



$$\sigma_y = 50 \text{ ksi}$$

$$I = 326 \text{ in}^4 \text{ (weak axis)}$$

$$= 904 \text{ in}^4 \text{ (strong)}$$

$$c = 14.7/2 \text{ (weak)}$$

$$c = 13.8/2 \text{ (strong)}$$

Find Yield Moment

$$M_y = \frac{\sigma_y I}{c} = \frac{50 \text{ ksi} (326 \text{ in}^4)}{14.7/2} = 2217.7 \text{ kip-in}$$

$$= 184.81 \text{ kip-ft (weak)}$$

$$= \frac{50 \text{ ksi} (904 \text{ in}^4)}{(13.8/2)} = 6550.7 \text{ kip-in (strong)}$$

$$= 545.89 \text{ kip-ft (strong)}$$

From Brom's Method Fig 16.8 for Long piles in cohesive soil

Ultimate Resistance Moment $\frac{M_y}{545.89 \text{ kip-ft (strong)}}$

$$= \frac{184.81}{(0.8 \text{ ksf})(\frac{13.8}{12})^3} = 151.89 \text{ weak}$$

From Figure 16.8

$\frac{P_u}{545.89 \text{ kip-ft (strong)}}$

$$= 61 \text{ weak} \quad P_u = 61 (0.8) \left(\frac{13.8}{12}\right)^2 = 64.5 \text{ kips}$$

$\frac{P_u}{120 \text{ kips}}$

$$= 100 \text{ strong} \quad P_u = 100 (0.8) \left(\frac{14.7}{12}\right)^2 = 120 \text{ kips}$$



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SHEET NO _____ OF _____
Calculated By _____ Date _____
Checked By _____ Date _____
Scale _____

For Abutment B

- look @ $S_u = 600$ psf for strong axis

$$M_{y \text{ strong}} = 546 \text{ k-ft}$$

Ultimate Resistance Moment Factor

$$\frac{M_y}{S_u d^3} = \frac{546 \text{ k-ft}}{0.6 \frac{\text{k}}{\text{ft}^2} \left(\frac{14.7}{12} \text{ ft}\right)^3} = 495, \text{ apply to chart}$$

712

Chapter 16

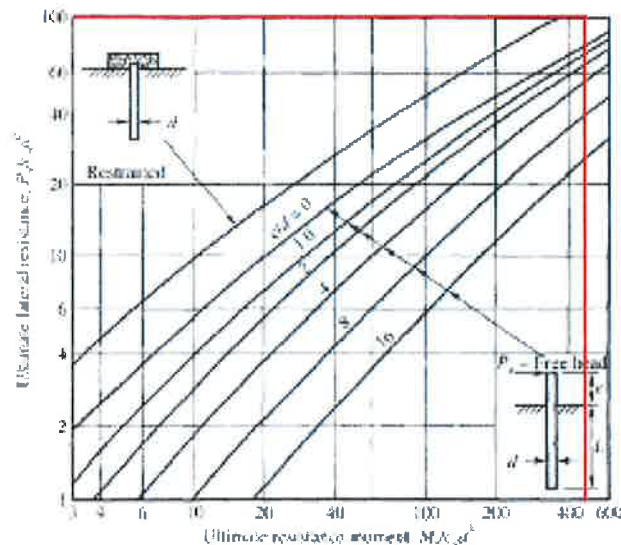


Figure 16.8 Ultimate lateral resistance of a long pile in cohesive soil related to embedded length (after Broms (1964a))

From Figure 16.8

$$\frac{P_u}{S_u d^2} = 100, \quad P_u = 100 \left(0.6 \frac{\text{k}}{\text{ft}^2}\right) \left(\frac{14.7}{12} \text{ ft}\right)^2 = 90 \text{ k}$$

As in Ex. 16.1 $T = 2.35$ m, $M_t = 0.268 \times 2 = 0.536$ MN-m

$$\text{Substituting, } y_g = \frac{2.43 \times 0.268 \times (2.35)^3}{4.35 \times 10^2} + \frac{1.62 \times 0.536 \times (2.35)^2}{4.35 \times 10^2}$$

$$= 0.0194 + 0.0110 = 0.0304 \text{ m} = 3.04 \text{ cm.}$$

Example 16.3

If the pile in Ex. 16.1 is fixed against rotation, calculate the deflection at the ground line.

Solution

Use Eq. (16.16a)

$$y_g = \frac{0.93 P_t T^3}{EI}$$

The values of P_t , T and EI are as given in Ex. 16.1. Substituting these values

$$y_g = \frac{0.93 \times 0.268 \times (2.35)^3}{4.35 \times 10^2} = 0.0075 \text{ m} = 0.75 \text{ cm}$$

16.6 BROMS' SOLUTIONS FOR LATERALLY LOADED PILES

Broms' (1964a, 1964b) solutions for laterally loaded piles deal with the following:

1. Lateral deflections of piles at ground level at working loads
2. Ultimate lateral resistance of piles under lateral loads

Broms' provided solutions for both short and long piles installed in cohesive and cohesionless soils respectively. He considered piles fixed or free to rotate at the head. Lateral deflections at working loads have been calculated using the concept of subgrade reaction. It is assumed that the deflection increases linearly with the applied loads when the loads applied are less than one-half to one-third of the ultimate lateral resistance of the pile.

Lateral Deflections at Working Loads

Lateral deflections at working loads can be obtained from Fig. 16.5 for cohesive soil and Fig. 16.6 for cohesionless soils respectively. For piles in saturated cohesive soils, the plot in Fig. 16.5 gives the relationships between the dimensionless quantity βL and $(y_o k d L)/P_t$ for free-head and restrained piles, where

$$\beta = \left(\frac{k d}{4 E I} \right)^{1/4} \quad (16.17)$$

EI = stiffness of pile section

k = coefficient of horizontal subgrade reaction

d = width or diameter of pile

L = length of pile

A pile is considered long or short on the following conditions

Free-head Pile

Long pile when $\beta L > 2.50$

Short pile when $\beta L < 2.50$



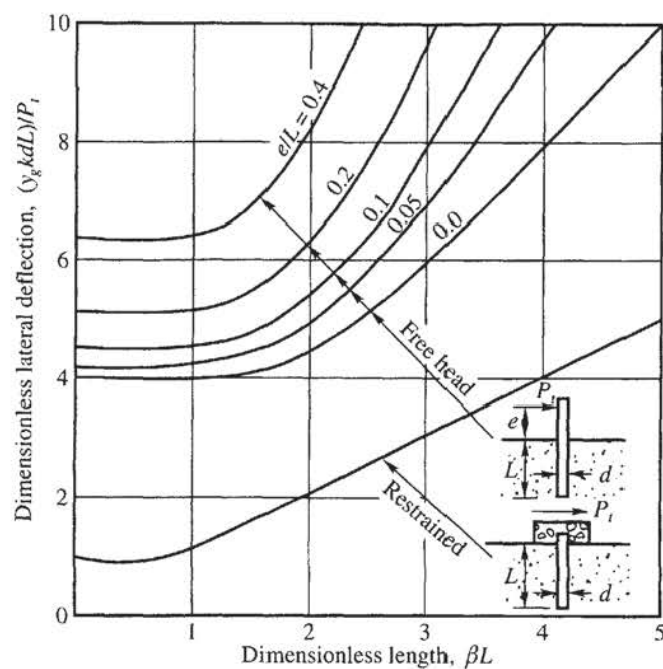


Figure 16.5 Charts for calculating lateral deflection at the ground surface of horizontally loaded pile in cohesive soil (after Broms 1964a)

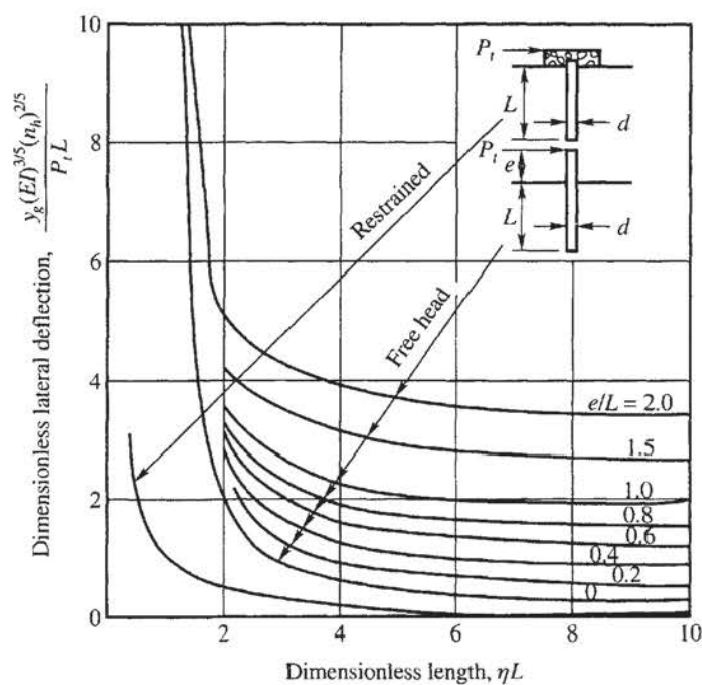


Figure 16.6 Charts for calculating lateral deflection at the ground surface of horizontally loaded piles in cohesionless soil (after Broms 1964b)



Fixed-head Pile

Long pile when $\beta L > 1.5$

Short pile when $\beta L < 1.5$

Tomlinson (1977) suggests that it is sufficiently accurate to take the value of k in Eq. (16.17) as equal to k_1 given in Table 14.1(b).

Lateral deflections at working loads of piles embedded in cohesionless soils may be obtained from Fig. 16.6 Non-dimensional factor $[y_g(EI)^{3/5}(n_h)^{2/5}]/P_t L$ is plotted as a function of ηL for various values of e/L

where y_g = deflection at ground level

$$\eta = \left(\frac{n_h}{EI} \right)^{1/5} \quad (16.18)$$

n_h = coefficient of soil modulus variation

P_t = lateral load applied at or above ground level

L = length of pile

e = eccentricity of load.

Ultimate Lateral Resistance of Piles in Saturated Cohesive Soils

The ultimate soil resistance of piles in cohesive soils increases with depth from $2c_u$ (c_u = undrained shear strength) to 8 to 12 c_u at a depth of three pile diameters ($3d$) below the surface. Broms (1964a) suggests a constant value of $9c_u$ below a depth of $1.5d$ as the ultimate soil resistance. Figure 16.7 gives solutions for short piles and Fig. 16.8 for long piles. The solution for long piles

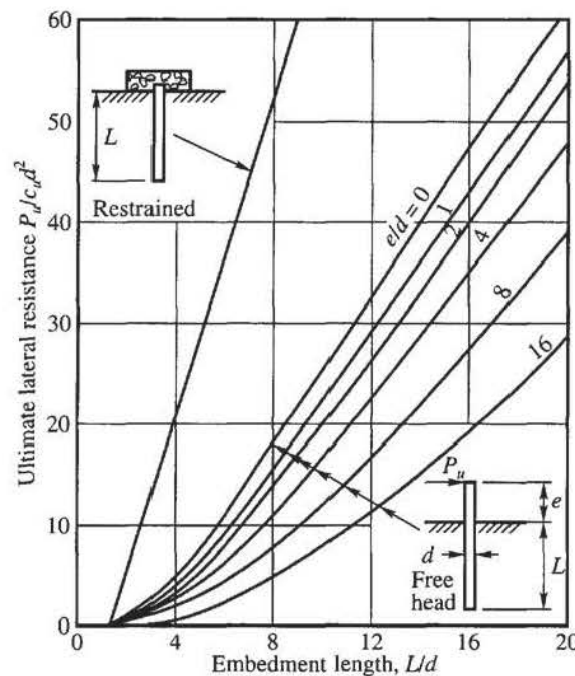


Figure 16.7 Ultimate lateral resistance of a short pile in cohesive soil related to embedded length (after Broms (1964a))



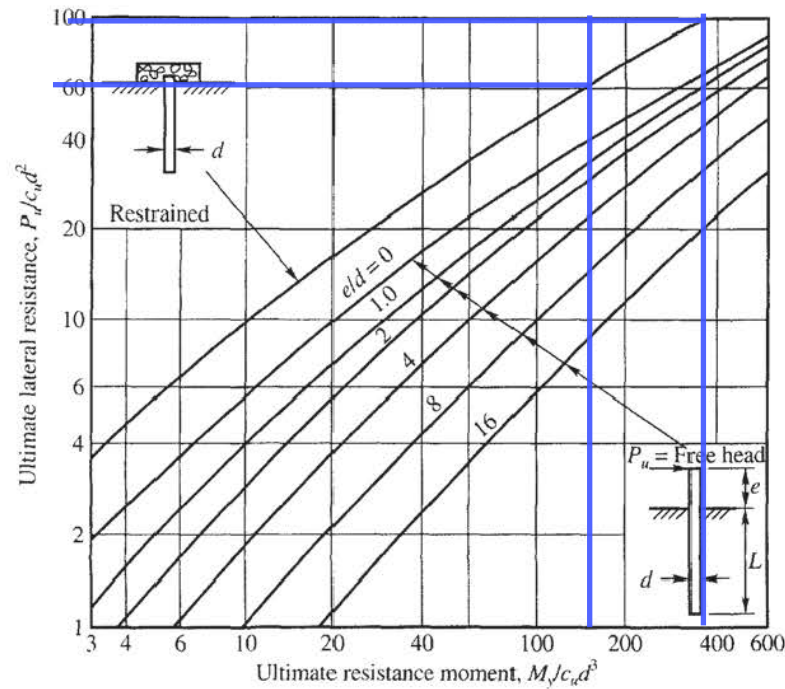


Figure 16.8 Ultimate lateral resistance of a long pile in cohesive soil related to embedded length (after Broms (1964a))

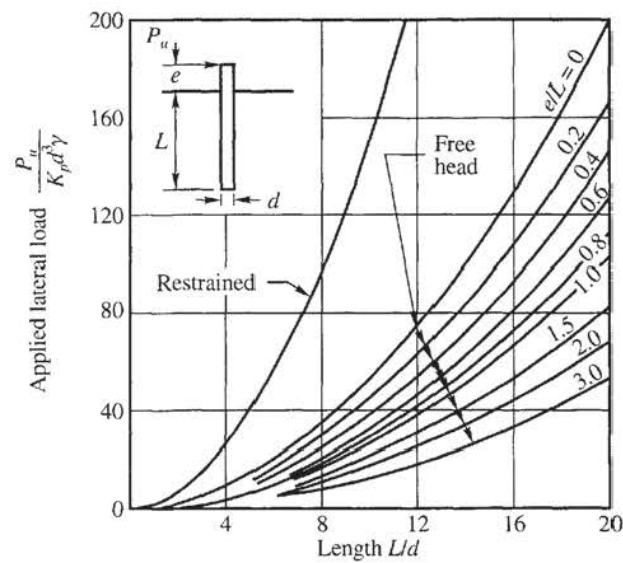


Figure 16.9 Ultimate lateral resistance of a short pile in cohesionless soil related to embedded length (after Broms (1964b))

involves the yield moment M_y for the pile section. The equations suggested by Broms for computing M_y are as follows:



Calculation: Ultimate Soil Resistance for Lateral Load on a 8" OD Timber Pile - Existing piles to be cut 5' below proposed GS

References: 1) Timber Pile Design and Construction Manual Chapter 7

Parameter Description	Parameter Symbol	Marine Clay	Reference
Pile Diameter	b (ft)	0.7	
Unconfined Compressive strength	q_u (ksf)	1.6	From lab, pocket penetrometer, and field data
Soil Density		--	
Empirical Coefficient from Table 7-1	n_1	0.36	TPDCM Table 7-2
Empirical Coefficient for Timber	n_2	1.3	TPDCM 7.2
Lateral Subgrade Modulus	K_h (kcf)	1,786	TPDCM Eq. 7-1/ 7-2
Adjustment for static loading	--	0.33	TPDCM 7.2 step 3
Adjusted Lateral Subgrade modulus	K_h (kcf)	589.5	
Modulus of Elasticity	E (ksi)	1,500	TPDCM Table 3-1
Moment of Inertia	I (ft ⁴)	0.01179	TPDCM Table 3-1
Section Modulus	S (ft ³)	0.03367	
Allowable Bending Stress of Timber Pile	F_b (ksf)	346	TPDCM Table 3-1 (2400psi)
Eccentricity of applied Load (ground to pile head)	e_c (ft)	0	TPDCM Table 3-1
Resisting Moment of the pile	M_v (K-ft)	12	TPDCM 7.2 step 4
Determine β_h for cohesive Soils	β_h	1.554	TPDCM 7-1 (equation on page 61, step 5)
Determine η for cohesionless Soils	η	--	TPDCM 7-1 (equation on page 61, step 5)
Pile Embedment Length	D (ft)	20	Interp Boring GZ-1,GZ-2, BB-YYR-401
Dimensionless Length Factor (Cohesive)	$\beta_h D$	31	TPDCM 7.2 step 6
Dimensionless Length Factor (Cohesionless)	ηD	--	TPDCM 7.2 step 6
Determine if Long or Short Pile		Long	TPDCM 7.2 step 7
Rankine Passive Earth Pressure	K_p	--	For Friction angle of 35° (see friction angle calculation)
Effective Unit Weight	γ' (pcf)	53	
Undrained Cohesion	c_u (ksf)	0.8	From lab, pocket penetrometer, and field data
	e_c/b	0.0	
	e_c/D	0.0	
Dimensionless Yield Factor (Cohesive)	$M_y/c_u b^3$	42.4	
Dimensionless Load Factor (Cohesive Free Head)	$Q_u/c_u b^2$	18.00	TPDCM Figure 7-2
Ultimate Lateral Load (Cohesive Free Head)	Q_u (kip)	7.1	

Notes:

1. Yellow cell is user input, white is calculated.
2. Free head condition used.

Table 3-1
Allowable Stress Values for Treated Round Timber Piles Graded in Accordance with ASTM D25

Species	Axial Compression (F_c) (psi)	Bending (F_b) (psi)	Shear Perpendicular to the Grain (F_v) (psi)	Compression Perpendicular to the Grain ($F_{c\perp}$) (psi)	Modulus of Elasticity (E) (psi)
Southern Pine ¹	1200	2400	110	250	1,500,000
Douglas Fir ²	1250	2450	115	230	1,500,000
Lodgepole Pine	1150	1700	80	270	1,000,000
Red Oak ³	1100	2450	135	350	1,250,000
Red Pine ⁴	900	1900	85	155	1,280,000

1. Southern Pine design values apply to Loblolly, Longleaf, Shortleaf, and Slash Pines.
2. Pacific Coast Douglas Fir design values apply to this species as defined in ASTM D 1760
3. Red Oak design values apply to Northern and Southern Red Oak
4. Red Pine design values apply to Red Pine grown in the United States

Table 7-1
Coefficient n_1 for Cohesive Soils

Unconfined Compressive Strength (ksf)	n_1
Less than 1	0.32
1 to 4	0.36
More than 4	0.40

Cohesionless Soils: Choose K_h from table 7-2.

Table 7-2
Values of K_h for Cohesionless Soils

Soil Density	K_h (kcf)	
	Above Groundwater	Below Groundwater
Loose	12	7
Medium	52	35
Dense	112	69

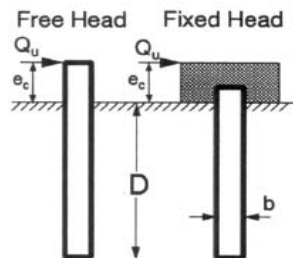
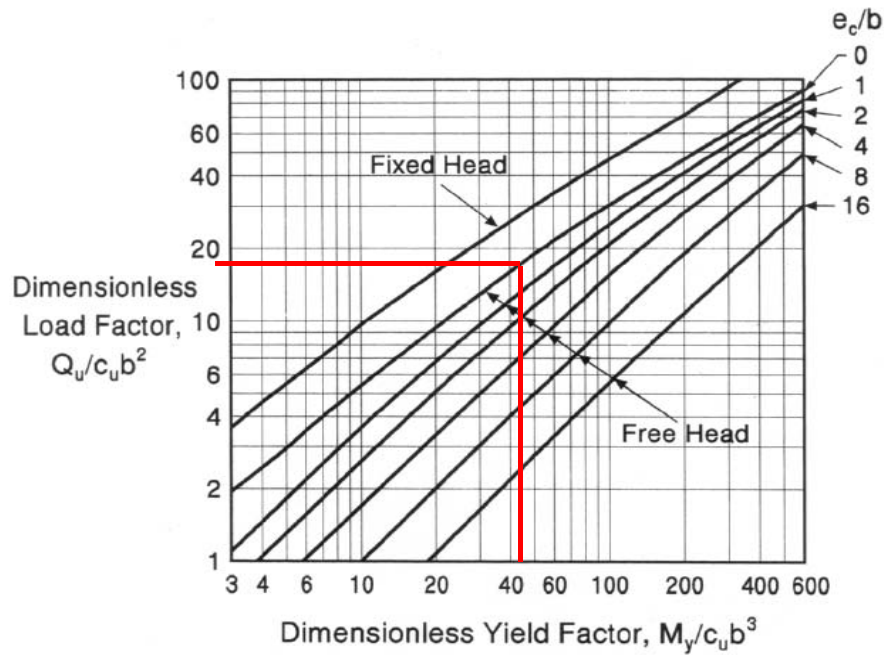


Figure 7-2: Ultimate lateral load capacity of long piles in cohesive soils (FHWA-HI-97-013)

Driveability



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GeoEnvironmental, Inc
477 Congress Street, Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
<http://www.gza.com>

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JOB: 09.0025988.01 Little River Bridge
SUBJECT: Axial Pile Resistance
SHEET: 1 OF 10
CALCULATED BY N. Williams, 9/10/19
REVIEWED BY A. Blaisdell, 9/11/19

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Little River Bridge Replacement in Gorham, ME

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017 with interims. (AASHTO LRFD)

Soil Properties

Consider Little River Bridge Interpretive Subsurface Profile (see Figure 2), subsurface layering and properties relative to pile design are discussed in the Global Stability calculation.

Structural Properties

HP14x89

$P_u := 410 \text{ kip}$ Maximum Factored Axial Load including downdrag

1.5" thermal movement given by Hoyle, Tanner & Associates, Inc. at each abutment.

HP14 x 89, Weak Axis Properties

Steel yield strength

$F_{y50} := 50 \text{ ksi}$

Modulus of elasticity for steel

$E := 29000 \text{ ksi}$

Cross sectional area of pile

$A_g := 26.1 \text{ in}^2$

Radius of gyration

$r_x := 5.88 \text{ in}$



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JOB: 09.0025988.01 Little River Bridge
SUBJECT: Axial Pile Resistance
SHEET: 2 OF 10
CALCULATED BY N. Williams, 9/10/19
REVIEWED BY A. Blaisdell, 9/11/19

1. Nominal Structural Compressive Resistance P_n

Determine the pile unbraced length of the pile by running LPILE software for top translation = 1.5 inches, $P = 330$ kip at the pile head, and Live Load Rotation = 0 (Fixed against rotation). Refer to the Lateral Pile analysis for unbraced lengths.

Maximum Unbraced lengths from LPILE output per abutment

Abutment 1 max unbraced length

$$L_1 := 17 \cdot \text{ft}$$

Abutment 2 max unbraced length (Based on longer pile case at Abutment 1)

$$L_2 := 23 \cdot \text{ft}$$

Calculate the column slenderness factor (λ) for each abutment's max unbraced length.

Upper segment slenderness factor

$$\lambda_1 := \frac{(L_1)}{r_x} = 34.694$$

Lower segment slenderness factor

$$\lambda_2 := \frac{(L_2)}{r_x} = 46.939$$

Determine K values for top and bottom of the pile per LRFD Table C4.6.2.5-1.
Maximum unbraced lengths occur at the second unbraced segment of each pile

Upper segment K value (Type d)

$$K_1 := 1$$

Lower segment K value (Type c)

$$K_2 := K_1$$

Calculate the nominal and factored structural pile resistance P_n , per LRFD 6.9.4.1 using the λ values

Elastic critical buckling resistance, P_e , based on flexural buckling



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GeoEnvironmental, Inc
 477 Congress Street, Suite 700
 Portland, Maine 04101
 207-879-9190
 Fax 207-879-0099
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JOB: 09.0025988.01 Little River Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 3 OF 10
 CALCULATED BY N. Williams, 9/10/19
 REVIEWED BY A. Blaisdell, 9/11/19

Abutment 1

$$P_{e1} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_1 \cdot \lambda_1)^2} = 6206 \cdot \text{kip}$$

Abutment 2

$$P_{e2} := \frac{(\pi^2 \cdot E) \cdot A_g}{(K_2 \cdot \lambda_2)^2} = 3391 \cdot \text{kip}$$

Nominal yield resistance, P_o

$$P_o := F_{y50} \cdot A_g = 1305 \cdot \text{kip}$$

Check that the ratio of P_e to P_o is > 0.44

AASHTO Eq. 6.9.4.1-1, -2

If $P_e/P_o > 0.44$ then use equation 6.9.4.1.1-1

If $P_e/P_o < 0.44$ then use equation 6.9.4.1.1-2

$$\frac{P_{e1}}{P_o} = 4.8 \quad \frac{P_{e2}}{P_o} = 2.6$$

Use eq. 6.9.4.1.1-1: Nominal structural Pile resistance, P_n for both segments

$$P_{n1} := \left[0.658 \left(\frac{P_o}{P_{e1}} \right) \right] \cdot P_o = 1195 \cdot \text{kip}$$

$$P_{n2} := \left[0.658 \left(\frac{P_o}{P_{e2}} \right) \right] \cdot P_o = 1111 \cdot \text{kip}$$

2. Factored Structural Compressive Resistance - Strength Limit State:

Factored Compressive Resistance for Strength Limit State:

Factored structural Pile Resistance, $P_r = \phi_c (P_n)$

$$\phi_c := .7 \quad P_{r1} := \phi_c \cdot P_{n1} = 837 \cdot \text{kip}$$

$$P_{r2} := \phi_c \cdot P_{n2} = 778 \cdot \text{kip}$$

d. Compare the ratio of P_u , the maximum factored axial load, to P_r , the structural resistance in the specified portion of the pile - the pile size should be such that the ratio is not less than 0.20.

$$\frac{P_u}{P_{r1}} = 0.49$$

$$\frac{P_u}{P_{r2}} = 0.527$$

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit
 States:

From Article 10.5.5.1 and 10.5.5.3

$$\phi := 1$$

Factored Compressive Resistance for Service/Extreme Limit State:

$$P_{rt1E} := \phi \cdot P_{n1} = 1195 \cdot \text{kip}$$

AASHTO Eq. 6.9.2.1-1

$$P_{rt2E} := \phi \cdot P_{n2} = 1111 \cdot \text{kip}$$



GZA
GeoEnvironmental, Inc
 477 Congress Street, Suite 700
 Portland, Maine 04101
 207-879-9190
 Fax 207-879-0099
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JOB: 09.0025988.01 Little River Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 4 OF 10
 CALCULATED BY N. Williams, 9/10/19
 REVIEWED BY A. Blaisdell, 9/11/19

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state.

Required nominal resistance of 547 kips for design of Abutments 1 and 2 based on a maximum factored pile load of 330 kips, a factored downdrag load of 80 kips, and a 0.75 resistance factor.

The estimated % skin friction resistance is a range between 5 and 20% at the required nominal pile resistance considering primarily clay soil profile and anticipated strength loss during driving.

5. Geotechnical Axial Resistance - Drivability Analysis

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$$

AASHTO Eq. 10.7.8.1

$$f_y := 50 \text{ ksi}$$

yield Strength of steel

$$\phi_{da} := 1.0$$

AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \sigma_{dr} = 45 \cdot \text{ksi} \quad \text{Driving Stress in pile cannot exceed 45 ksi}$$

Abutment 1 - Drive pile through 26 feet of soil to rock with toe quake representative of hard driving conditions (0.04in). Model pile length as 30 feet (4 foot stickup at end of drive).

Abutment 2 - Drive pile through 53 feet of soil to rock with toe quake representative of moderate driving conditions (0.07 in). Model pile length as 55 feet (2 foot stickup at end of drive).

Drive piles with a Delmag D 30-32 open-ended diesel hammer with a rated energy of 75,250 ft-lb. **The proposed hammer is sized to achieve the required nominal pile resistance; not necessarily the maximum drivability resistance for the pile section and profile.**

Allowable stress in piles during driving is $0.9 f_y = 45 \text{ ksi}$

GRLWEAP Output is attached on Sheets 7 through 10.

Abutment 1: $R_{ndr1} := 547 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=42 ksi, final penetration resistance=11 bpi.

Abutment 2: $R_{ndr2} := 547 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=42 ksi, final penetration resistance=10 bpi.



GZA
GeoEnvironmental, Inc
477 Congress Street, Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
http://www.gza.com

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JOB: 09.0025988.01 Little River Bridge
SUBJECT: Axial Pile Resistance
SHEET: 5 OF 10
CALCULATED BY N. Williams, 9/10/19
REVIEWED BY A. Blaisdell, 9/11/19

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria on all piles.

$$\phi_{\text{dyn}} := 0.75$$

AASHTO Table 10.5.5.2.3-1

Abutment 1: $R_{\text{ndr1_factored}} := R_{\text{ndr1}} \cdot \phi_{\text{dyn}}$

$$R_{\text{ndr1_factored}} = 410 \cdot \text{kip}$$

Abutment 2: $R_{\text{ndr2_factored}} := R_{\text{ndr2}} \cdot \phi_{\text{dyn}}$

$$R_{\text{ndr2_factored}} := 410 \cdot \text{kip}$$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{\text{serv_ext}} := 1$

From Article 10.5.5.1 and 10.5.5.3

$$R_{\text{ndr1_serv_ext}} := R_{\text{ndr1}} \cdot \phi_{\text{serv_ext}}$$

$$R_{\text{ndr1_serv_ext}} = 547 \cdot \text{kip}$$

$$R_{\text{ndr2_serv_ext}} := R_{\text{ndr2}} \cdot \phi_{\text{serv_ext}}$$

$$R_{\text{ndr2_serv_ext}} = 547 \cdot \text{kip}$$

Summary of Results - Axial Loading:

ASTM A572 HP 14x89	Structural Resistance (kips)	Geotechnical Static Resistance (kips)	Required Geotechnical Resistance (kips)	Governing Resistance (kips)
Abutment 1: Strength Limit State, Design	837	N/A	410	410
Abutment 1: Service/Extreme Limit State, Design	1195	N/A	547	547
Abutment 2: Strength Limit State, Design	778	N/A	410	410
Abutment 2: Service/Extreme Limit State, Design	1111	N/A	547	547

Results indicate the selected hammer is suitable to meet the required nominal pile resistance.

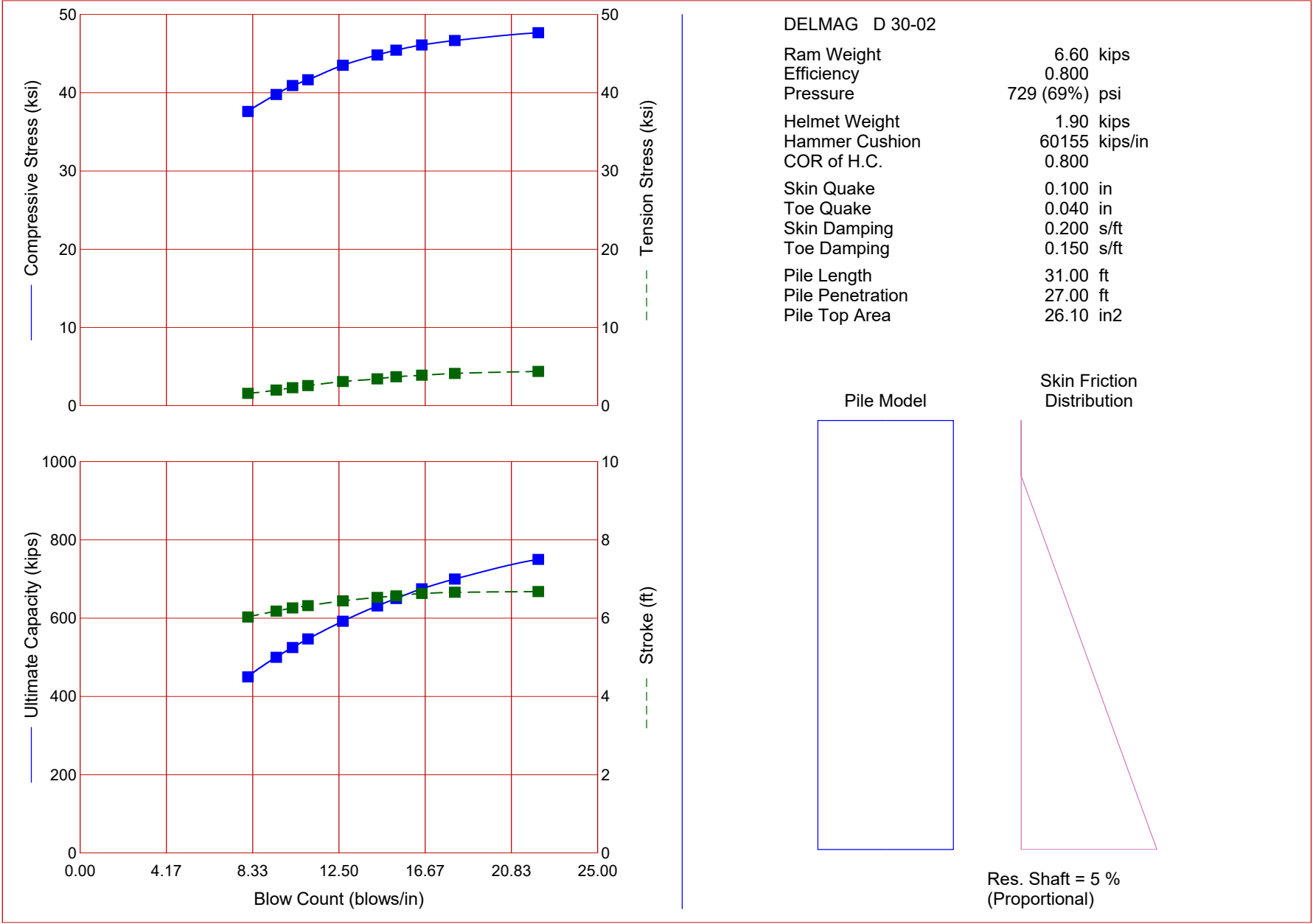
Abutment 1 - 5% Side Friction

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14-Aug-2019
GRLWEAP Version 2010

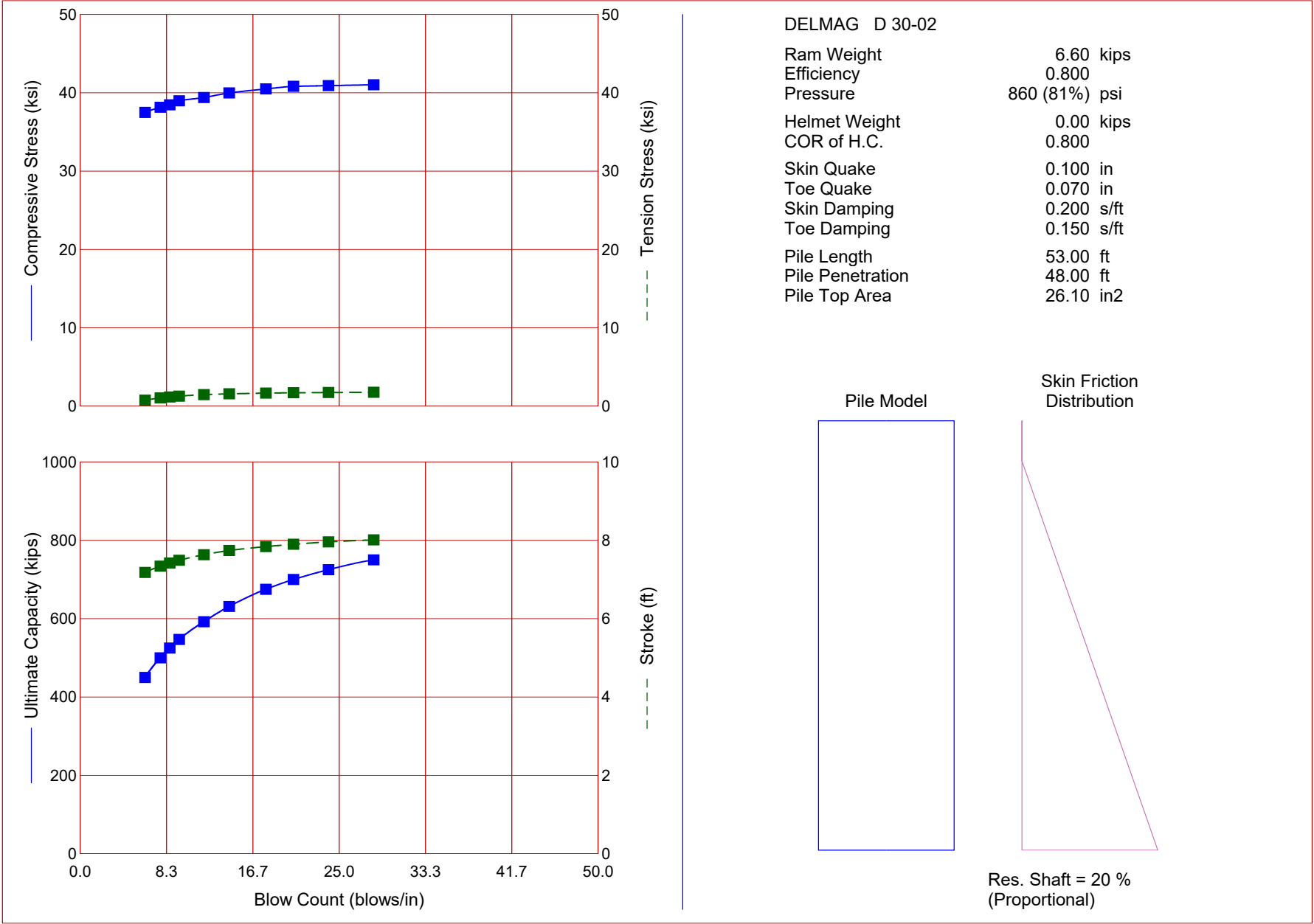
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
450.0	37.60	1.58	8.1	6.03	13.39
500.0	39.76	2.00	9.5	6.18	13.87
525.0	40.90	2.31	10.3	6.26	14.16
547.0	41.64	2.58	11.0	6.32	14.36
592.0	43.50	3.10	12.7	6.44	14.80
631.0	44.80	3.44	14.4	6.53	15.15
650.0	45.43	3.71	15.3	6.57	15.29
675.0	46.09	3.90	16.5	6.63	15.51
700.0	46.66	4.13	18.1	6.66	15.58
750.0	47.66	4.38	22.1	6.68	15.68

Abutment 1 - 5% Side Friction



Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
450.0	37.49	0.76	6.3	7.18	20.91
500.0	38.14	1.04	7.7	7.34	21.59
525.0	38.45	1.16	8.6	7.42	21.92
547.0	38.96	1.27	9.6	7.49	22.24
592.0	39.39	1.46	11.9	7.63	22.80
631.0	39.97	1.57	14.4	7.74	23.24
675.0	40.48	1.66	17.9	7.84	23.72
700.0	40.81	1.70	20.6	7.90	23.97
725.0	40.91	1.74	24.0	7.96	24.18
750.0	41.01	1.76	28.3	8.01	24.39

Abutment 2 - 20% Side Friction



Lateral Pile Analysis



Table 1 - LPile Input Parameters
Bridge Over Little River #3112 Gorham, ME
 GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0025988.01
CALCULATED BY N. Williams, 8/16/19
CHECKED BY A. Blaisdell, 8/16/19

Abutment 1, Pile length = 26' (min. anticipated)						
Stratum	Soil Model	Top of Layer Elevation (ft)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Clay Fill	Matlock Clay	83.0	5	0.02	400	115
Clay Fill	Matlock Clay	78.0	4	0.02	400	53
Marine Deposit	Matlock Clay	74.0	9	0.008	900	53
Marine Deposit	Matlock Clay	65.0	8	0.008	800	43

Abutment 1, Pile length = 34'						
Stratum	Soil Model	Top of Layer Elevation (ft)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Clay Fill	Matlock Clay	83.0	5	0.02	400	115
Clay Fill	Matlock Clay	78.0	4	0.02	400	53
Marine Deposit	Matlock Clay	74.0	9	0.008	900	53
Marine Deposit	Matlock Clay	65.0	9	0.008	800	43
Marine Deposit	Matlock Clay	56.0	7	0.008	550	53
		Total	34			



Table 2 - LPile Output Summary
Bridge Over Little River #3112 Gorham, ME
GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0025988.01
CALCULATED BY N. Williams, 8/21/19
CHECKED BY A. Blaisdell, 8/21/19

Abutment 1, HP 14x89								
Pile Length	Axial Load (kips)	Shear Force for Lateral deflection of 1.5 in. (kips)	Moment at Pile Head (kip-ft)	Depth below Pile Head to Fixity (ft)	Total Stress at Pile Head (ksi)	Bending Stress at Pile Head (ksi)	Axial Stress at Pile Head (ksi)	Deflection of Pile Tip (inches)
26	330	47.9	-379.5	--	47.4	34.8	12.6	-0.232
26*	330	50.3	-393.3	--	48.7	36.0	12.6	-0.059
26**	330	51.3	-399.8	--	49.3	36.6	12.6	-0.003
34	330	51.1	-399.2	33.0	49.2	36.6	12.6	0.002

Notes:

1. Soil layering and properties are presented in Table 1.
2. The axial load is the maximum Factored Axial Load, excluding factored downdrag load, for the 7 Pile configuration.
3. Lpile model included imposed lateral deflection of 1.5 inches and no rotation (fixed pile head).
4. The evaluated pile section is HP14x89, strong axis bending.
5. The bending stress at pile head is calculated as the maximum moment divided by the pile section modulus.
6. * Indicates Pile was evaluated with a shear tip resistance of 10 kips at a deflection of 0.03" to 5" (plastic response beyond 0.3"). The Lpile model converged at 0.06" deflection for this case.
7. ** Indicates Pile was evaluated with 3" embedment into bedrock.
8. Negative deflection of pile tip indicates opposite direction from pile head deflection, positive indicates same direction.

Abutment 1 - 26 foot Pile
No shear resistance at tip

Abutment 1_14x89.lp9o

LPILE for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\\09 Jobs\0025900s\09.0025988.00 - Gorham - Bridge over Little River\09.0025988.01 - Final Design\Work\LPILE\Abutment
1\HP 14x89\26 Foot length - bb-101\

Name of input data file:

Abutment 1_14x89.lp9d

Name of output report file:

Abutment 1_14x89.lp9o

Name of plot output file:

Abutment 1_14x89.lp9p

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Abutment 1_14x89.lp9o

Name of runtime message file:

Abutment 1_14x89.lp9r

Date and Time of Analysis

Date: August 19, 2019

Time: 13:43:12

Problem Title

Project Name: Little River Bridge #3112

Job Number: 09.0025988.00

Client: Hoyle, Tanner & Associates

Engineer: N. Williams

Description: Abutment 1

Program Options and Settings

Computational Options:

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- Use unfactored loads in computations (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
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- 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 = 26.000 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.
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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	26.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Strong H-Pile
Length of section	= 26.000000 ft
Flange Width	= 14.700000 in
Section Depth	= 13.800000 in
Flange Thickness	= 0.615000 in
Web Thickness	= 0.615000 in
Section Area	= 25.811550 sq. in
Moment of Inertia	= 888.177980 in^4
Elastic Modulus	= 29000000. psi

 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 5 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 0.0000 ft
 Distance from top of pile to bottom of layer = 5.000000 ft
 Effective unit weight at top of layer = 115.000000 pcf
 Effective unit weight at bottom of layer = 115.000000 pcf
 Undrained cohesion at top of layer = 400.000000 psf
 Undrained cohesion at bottom of layer = 400.000000 psf
 Epsilon-50 at top of layer = 0.020000
 Epsilon-50 at bottom of layer = 0.020000

Layer 2 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 5.000000 ft
 Distance from top of pile to bottom of layer = 9.000000 ft
 Effective unit weight at top of layer = 53.000000 pcf
 Effective unit weight at bottom of layer = 53.000000 pcf
 Undrained cohesion at top of layer = 400.000000 psf
 Undrained cohesion at bottom of layer = 400.000000 psf
 Epsilon-50 at top of layer = 0.020000
 Epsilon-50 at bottom of layer = 0.020000

Layer 3 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 9.000000 ft
 Distance from top of pile to bottom of layer = 18.000000 ft
 Effective unit weight at top of layer = 53.000000 pcf
 Effective unit weight at bottom of layer = 53.000000 pcf
 Undrained cohesion at top of layer = 900.000000 psf
 Undrained cohesion at bottom of layer = 900.000000 psf
 Epsilon-50 at top of layer = 0.008000
 Epsilon-50 at bottom of layer = 0.008000

Layer 4 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 18.000000 ft
 Distance from top of pile to bottom of layer = 27.000000 ft
 Effective unit weight at top of layer = 43.000000 pcf
 Effective unit weight at bottom of layer = 43.000000 pcf
 Undrained cohesion at top of layer = 800.000000 psf
 Undrained cohesion at bottom of layer = 800.000000 psf
 Epsilon-50 at top of layer = 0.008000
 Epsilon-50 at bottom of layer = 0.008000

Layer 5 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 27.000000 ft
 Distance from top of pile to bottom of layer = 35.000000 ft
 Effective unit weight at top of layer = 53.000000 pcf
 Effective unit weight at bottom of layer = 53.000000 pcf
 Undrained cohesion at top of layer = 550.000000 psf
 Undrained cohesion at bottom of layer = 550.000000 psf
 Epsilon-50 at top of layer = 0.008000
 Epsilon-50 at bottom of layer = 0.008000

(Depth of the lowest soil layer extends 9.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	E50 or krm
1	Soft	0.00	115.0000	400.0000	0.02000
	Clay	5.0000	115.0000	400.0000	0.02000
2	Soft	5.0000	53.0000	400.0000	0.02000

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3	Clay	9.0000	53.0000	400.0000	0.02000
	Soft	9.0000	53.0000	900.0000	0.00800
4	Clay	18.0000	53.0000	900.0000	0.00800
	Soft	18.0000	43.0000	800.0000	0.00800
5	Clay	27.0000	43.0000	800.0000	0.00800
	Soft	27.0000	53.0000	550.0000	0.00800
	Clay	35.0000	53.0000	550.0000	0.00800

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
1	5	y = 1.500000 in	S = 0.0000 in/in	330000.	N.A.

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

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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	11611.
2	5.0000	5.0000	Yes	No	11611.	14844.
3	9.0000	9.0000	Yes	No	26455.	86736.
4	18.0000	18.0000	Yes	No	113191.	70575.
5	27.0000	27.0000	No	No	183766.	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

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Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 1.50000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 330000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.5009	-4553807.	47913.	0.00	50469.	2.58E+10	-77.7065	80.7684	0.00
0.2600	1.5000	-4404444.	47653.	-5.43E-04	49233.	2.58E+10	-82.3757	171.3414	0.00
0.5200	1.4975	-4255333.	47389.	-0.00107	47999.	2.58E+10	-87.0109	181.2879	0.00
0.7800	1.4933	-4106538.	47110.	-0.00157	46768.	2.58E+10	-91.6079	191.3941	0.00
1.0400	1.4877	-3958123.	46818.	-0.00206	45540.	2.58E+10	-96.1627	201.6781	0.00
1.3000	1.4805	-3810151.	46510.	-0.00253	44315.	2.58E+10	-100.6714	212.1581	0.00
1.5600	1.4719	-3662683.	46189.	-0.00299	43095.	2.58E+10	-105.1302	222.8525	0.00
1.8200	1.4618	-3515782.	45855.	-0.00342	41879.	2.58E+10	-109.5355	233.7799	0.00
2.0800	1.4505	-3369509.	45506.	-0.00384	40669.	2.58E+10	-113.8837	244.9594	0.00
2.3400	1.4379	-3223924.	45144.	-0.00424	39464.	2.58E+10	-118.1713	256.4106	0.00
2.6000	1.4241	-3079087.	44769.	-0.00462	38266.	2.58E+10	-122.3950	268.1537	0.00
2.8600	1.4091	-2935058.	44380.	-0.00498	37074.	2.58E+10	-126.5516	280.2096	0.00
3.1200	1.3930	-2791895.	43979.	-0.00533	35889.	2.58E+10	-130.6377	292.6000	0.00
3.3800	1.3758	-2649655.	43565.	-0.00566	34712.	2.58E+10	-134.6504	305.3477	0.00
3.6400	1.3577	-2508395.	43139.	-0.00597	33543.	2.58E+10	-138.5865	318.4762	0.00
3.9000	1.3386	-2368172.	42701.	-0.00627	32382.	2.58E+10	-142.4430	332.0103	0.00
4.1600	1.3186	-2229039.	42250.	-0.00654	31231.	2.58E+10	-146.2172	345.9761	0.00
4.4200	1.2977	-2091053.	41788.	-0.00681	30089.	2.58E+10	-149.9061	360.4009	0.00
4.6800	1.2761	-1954264.	41315.	-0.00705	28957.	2.58E+10	-153.5069	375.3138	0.00
4.9400	1.2537	-1818726.	40831.	-0.00728	27836.	2.58E+10	-157.0169	390.7451	0.00
5.2000	1.2307	-1684490.	40337.	-0.00749	26725.	2.58E+10	-159.6819	404.8220	0.00
5.4600	1.2070	-1551598.	39835.	-0.00769	25625.	2.58E+10	-162.0365	418.8554	0.00
5.7200	1.1827	-1420090.	39326.	-0.00787	24537.	2.58E+10	-164.3058	433.4405	0.00
5.9800	1.1579	-1290005.	38810.	-0.00803	23460.	2.58E+10	-166.4879	448.6102	0.00
6.2400	1.1326	-1161379.	38287.	-0.00818	22396.	2.58E+10	-168.5811	464.3993	0.00
6.5000	1.1068	-1034249.	37758.	-0.00831	21344.	2.58E+10	-170.5837	480.8454	0.00
6.7600	1.0807	-908651.	37223.	-0.00843	20304.	2.58E+10	-172.4940	497.9883	0.00
7.0200	1.0542	-784618.	36682.	-0.00853	19278.	2.58E+10	-174.3103	515.8708	0.00
7.2800	1.0275	-662185.	36135.	-0.00862	18265.	2.58E+10	-176.0311	534.5388	0.00
7.5400	1.0004	-541382.	35583.	-0.00869	17265.	2.58E+10	-177.6547	554.0415	0.00
7.8000	0.9732	-422242.	35027.	-0.00875	16279.	2.58E+10	-179.1796	574.4316	0.00

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8.0600	0.9458	-304792.	34465.	-0.00880	15307.	2.58E+10	-180.6044	595.7657	0.00
8.3200	0.9183	-189063.	33900.	-0.00883	14350.	2.58E+10	-181.9277	618.1048	0.00
8.5800	0.8907	-75082.	33330.	-0.00884	13406.	2.58E+10	-183.1481	641.5144	0.00
8.8400	0.8631	37126.	32757.	-0.00885	13092.	2.58E+10	-184.2642	666.0649	0.00
9.1000	0.8355	147536.	31699.	-0.00883	14006.	2.58E+10	-494.1747	1845.	0.00
9.3600	0.8080	253117.	30153.	-0.00881	14880.	2.58E+10	-496.4996	1917.	0.00
9.6200	0.7806	353834.	28601.	-0.00877	15713.	2.58E+10	-498.5391	1993.	0.00
9.8800	0.7533	449653.	27043.	-0.00872	16506.	2.58E+10	-500.2902	2072.	0.00
10.1400	0.7261	540546.	25480.	-0.00866	17258.	2.58E+10	-501.7497	2156.	0.00
10.4000	0.6992	626487.	23912.	-0.00859	17969.	2.58E+10	-502.9146	2244.	0.00
10.6600	0.6725	707455.	22342.	-0.00851	18639.	2.58E+10	-503.7815	2337.	0.00
10.9200	0.6461	783430.	20769.	-0.00842	19268.	2.58E+10	-504.3469	2436.	0.00
11.1800	0.6200	854399.	19195.	-0.00832	19855.	2.58E+10	-504.6073	2539.	0.00
11.4400	0.5941	920348.	17621.	-0.00822	20401.	2.58E+10	-504.5586	2650.	0.00
11.7000	0.5687	981272.	16047.	-0.00810	20905.	2.58E+10	-504.1968	2766.	0.00
11.9600	0.5436	1037164.	14475.	-0.00798	21368.	2.58E+10	-503.5173	2890.	0.00
12.2200	0.5189	1088027.	12910.	-0.00785	21789.	2.58E+10	-499.6381	3004.	0.00
12.4800	0.4946	1133889.	11364.	-0.00772	22168.	2.58E+10	-491.7187	3102.	0.00
12.7400	0.4708	1174824.	9842.	-0.00758	22507.	2.58E+10	-483.6824	3206.	0.00
13.0000	0.4474	1210904.	8346.	-0.00743	22806.	2.58E+10	-475.5272	3317.	0.00
13.2600	0.4244	1242203.	6875.	-0.00728	23065.	2.58E+10	-467.2502	3435.	0.00
13.5200	0.4019	1268800.	5430.	-0.00713	23285.	2.58E+10	-458.8478	3562.	0.00
13.7800	0.3799	1290772.	4012.	-0.00698	23467.	2.58E+10	-450.3155	3698.	0.00
14.0400	0.3584	1308199.	2621.	-0.00682	23611.	2.58E+10	-441.6474	3845.	0.00
14.3000	0.3374	1321164.	1256.	-0.00666	23718.	2.58E+10	-432.8366	4003.	0.00
14.5600	0.3168	1329750.	-79.9759	-0.00650	23789.	2.58E+10	-423.8745	4174.	0.00
14.8200	0.2968	1334045.	-1388.	-0.00634	23825.	2.58E+10	-414.7506	4360.	0.00
15.0800	0.2773	1334136.	-2668.	-0.00617	23825.	2.58E+10	-405.4526	4562.	0.00
15.3400	0.2583	1330113.	-3918.	-0.00601	23792.	2.58E+10	-395.9651	4783.	0.00
15.6000	0.2398	1322071.	-5138.	-0.00585	23726.	2.58E+10	-386.2700	5026.	0.00
15.8600	0.2218	1310103.	-6328.	-0.00569	23627.	2.58E+10	-376.3449	5295.	0.00
16.1200	0.2042	1294308.	-7486.	-0.00554	23496.	2.58E+10	-366.1630	5593.	0.00
16.3800	0.1872	1274788.	-8612.	-0.00538	23334.	2.58E+10	-355.6909	5927.	0.00
16.6400	0.1707	1251646.	-9705.	-0.00523	23143.	2.58E+10	-344.8878	6305.	0.00
16.9000	0.1546	1224991.	-10764.	-0.00508	22922.	2.58E+10	-333.7026	6734.	0.00
17.1600	0.1390	1194934.	-11787.	-0.00493	22673.	2.58E+10	-322.0707	7229.	0.00
17.4200	0.1238	1161594.	-12773.	-0.00479	22398.	2.58E+10	-309.9090	7808.	0.00
17.6800	0.1091	1125092.	-13720.	-0.00465	22096.	2.58E+10	-297.1086	8495.	0.00
17.9400	0.09483	1085557.	-14625.	-0.00452	21768.	2.58E+10	-283.5230	9329.	0.00
18.2000	0.08094	1043127.	-15441.	-0.00439	21417.	2.58E+10	-269.0654	9215.	0.00
18.4600	0.06745	998240.	-16165.	-0.00426	21046.	2.58E+10	-224.9689	10406.	0.00

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18.7200	0.05434	951038.	-16842.	-0.00414	20655.	2.58E+10	-209.3280	12018.	0.00
18.9800	0.04159	901680.	-17467.	-0.00403	20247.	2.58E+10	-191.4708	14363.	0.00
19.2400	0.02918	850346.	-18031.	-0.00393	19822.	2.58E+10	-170.1292	18191.	0.00
19.5000	0.01709	797249.	-18519.	-0.00383	19383.	2.58E+10	-142.3274	25983.	0.00
19.7600	0.00530	742668.	-18891.	-0.00373	18931.	2.58E+10	-96.2957	56670.	0.00
20.0200	-0.00621	687056.	-18883.	-0.00365	18471.	2.58E+10	101.6360	51094.	0.00
20.2800	-0.01745	632349.	-18501.	-0.00357	18018.	2.58E+10	143.4009	25633.	0.00
20.5400	-0.02846	578958.	-18014.	-0.00349	17576.	2.58E+10	168.7744	18500.	0.00
20.8000	-0.03925	527138.	-17457.	-0.00343	17147.	2.58E+10	187.8540	14931.	0.00
21.0600	-0.04985	477081.	-16847.	-0.00337	16733.	2.58E+10	203.4185	12733.	0.00
21.3200	-0.06026	428945.	-16191.	-0.00331	16335.	2.58E+10	216.6923	11220.	0.00
21.5800	-0.07051	382864.	-15497.	-0.00326	15953.	2.58E+10	228.3381	10104.	0.00
21.8400	-0.08061	338959.	-14769.	-0.00322	15590.	2.58E+10	238.7610	9241.	0.00
22.1000	-0.09059	297335.	-14009.	-0.00318	15246.	2.58E+10	248.2286	8550.	0.00
22.3600	-0.1004	258091.	-13221.	-0.00315	14921.	2.58E+10	256.9286	7980.	0.00
22.6200	-0.1102	221315.	-12407.	-0.00312	14616.	2.58E+10	264.9980	7502.	0.00
22.8800	-0.1199	187092.	-11568.	-0.00309	14333.	2.58E+10	272.5405	7092.	0.00
23.1400	-0.1295	155498.	-10707.	-0.00307	14072.	2.58E+10	279.6363	6737.	0.00
23.4000	-0.1391	126607.	-9824.	-0.00305	13833.	2.58E+10	286.3488	6424.	0.00
23.6600	-0.1486	100488.	-8920.	-0.00304	13617.	2.58E+10	292.7292	6147.	0.00
23.9200	-0.1580	77205.	-7997.	-0.00303	13424.	2.58E+10	298.8190	5899.	0.00
24.1800	-0.1675	56822.	-7056.	-0.00302	13255.	2.58E+10	304.6525	5676.	0.00
24.4400	-0.1769	39397.	-6097.	-0.00302	13111.	2.58E+10	310.2582	5472.	0.00
24.7000	-0.1863	24988.	-5120.	-0.00301	12992.	2.58E+10	315.6600	5287.	0.00
24.9600	-0.1957	13648.	-4127.	-0.00301	12898.	2.58E+10	320.8778	5116.	0.00
25.2200	-0.2051	5430.	-3118.	-0.00301	12830.	2.58E+10	325.9284	4959.	0.00
25.4800	-0.2145	384.2399	-2094.	-0.00301	12788.	2.58E+10	330.8262	4813.	0.00
25.7400	-0.2238	-1441.	-1054.	-0.00301	12797.	2.58E+10	335.5830	4678.	0.00
26.0000	-0.2332	0.00	0.00	-0.00301	12785.	2.58E+10	340.2089	2276.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.50086051 inches
Computed slope at pile head = 0.000000 radians
Maximum bending moment = -4553807. inch-lbs
Maximum shear force = 47913. lbs
Depth of maximum bending moment = 0.000000 feet below pile head
Depth of maximum shear force = 0.000000 feet below pile head

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Number of iterations = 26
Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

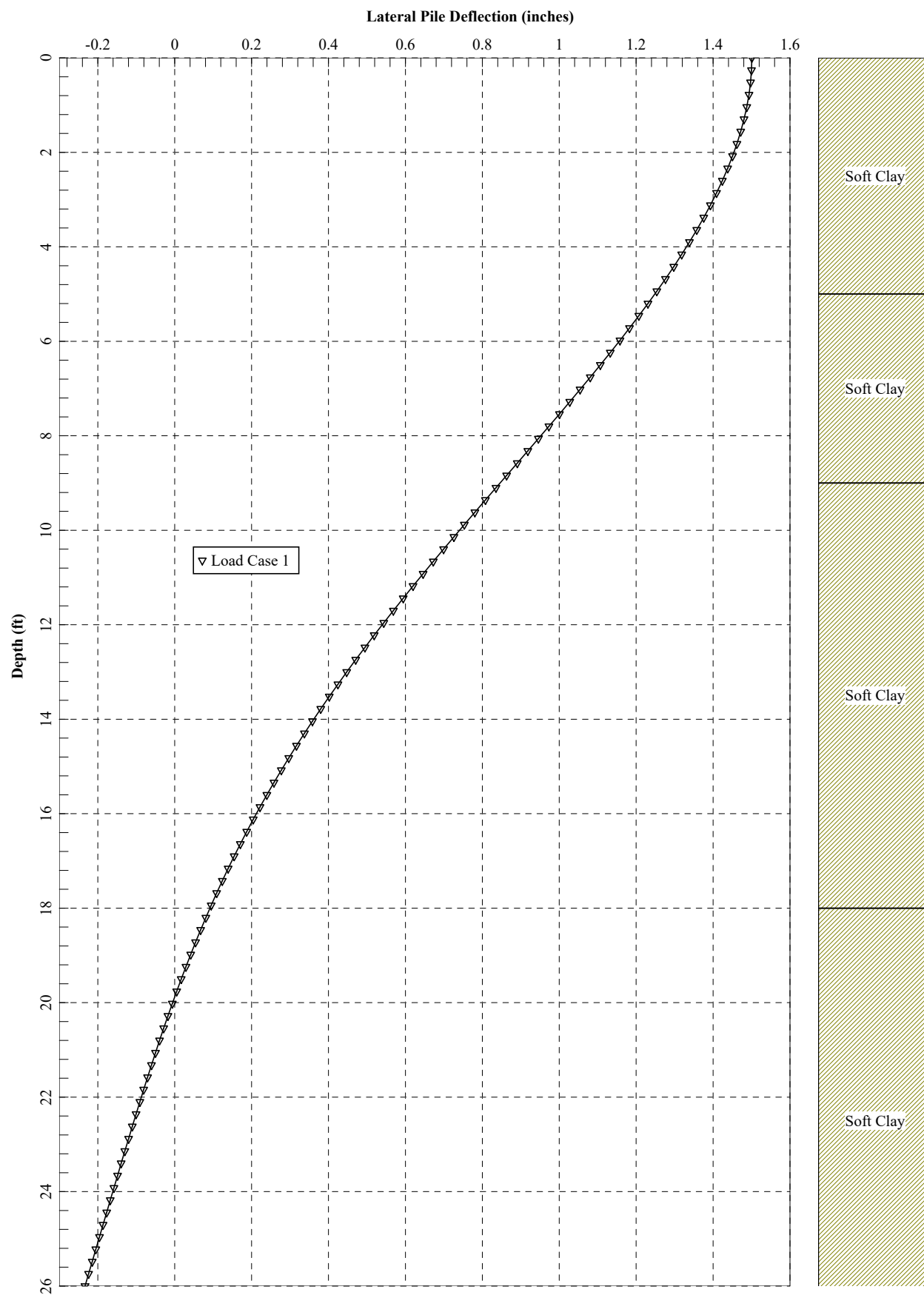
Definitions of Pile-head Loading Conditions:

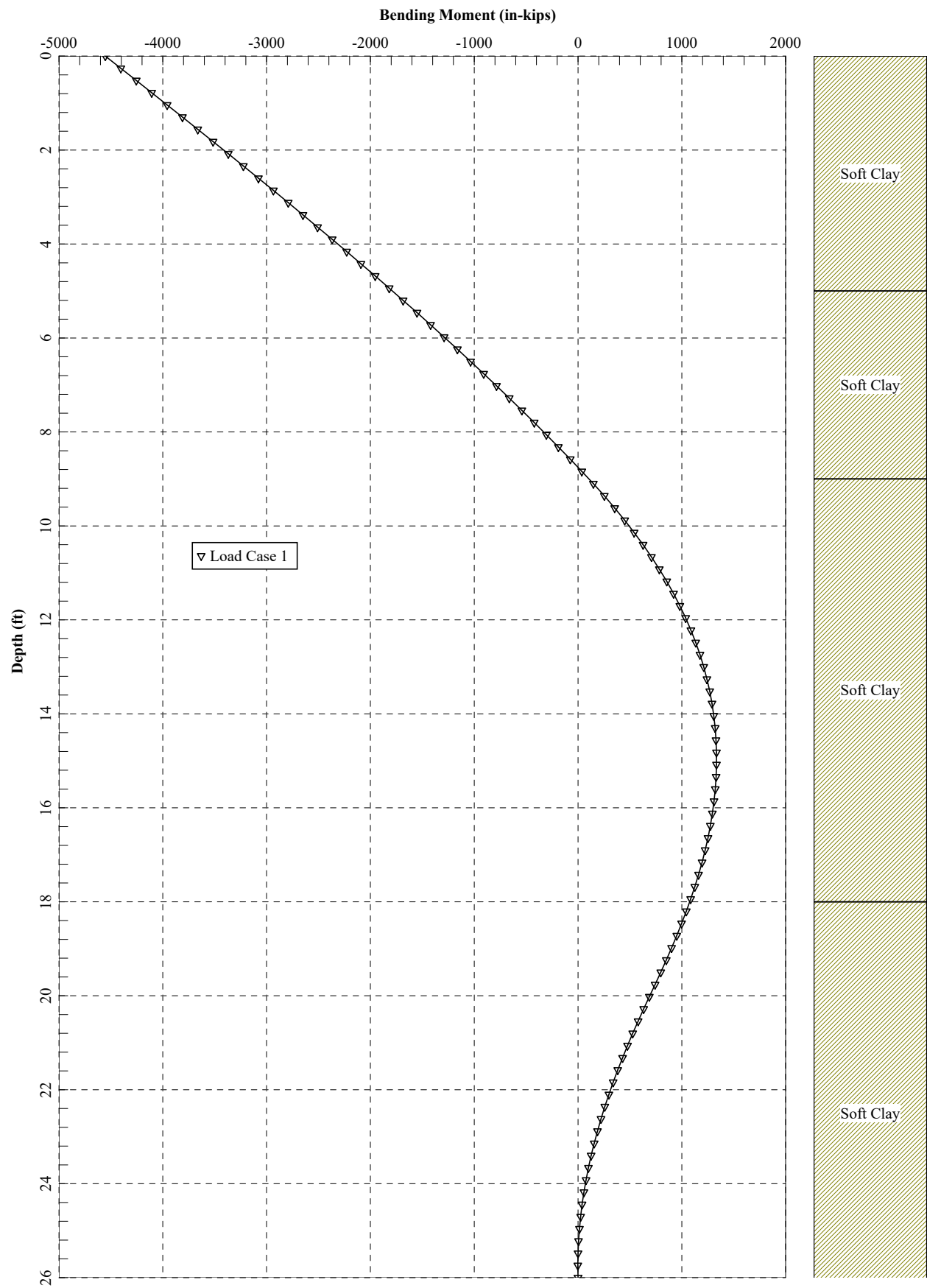
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

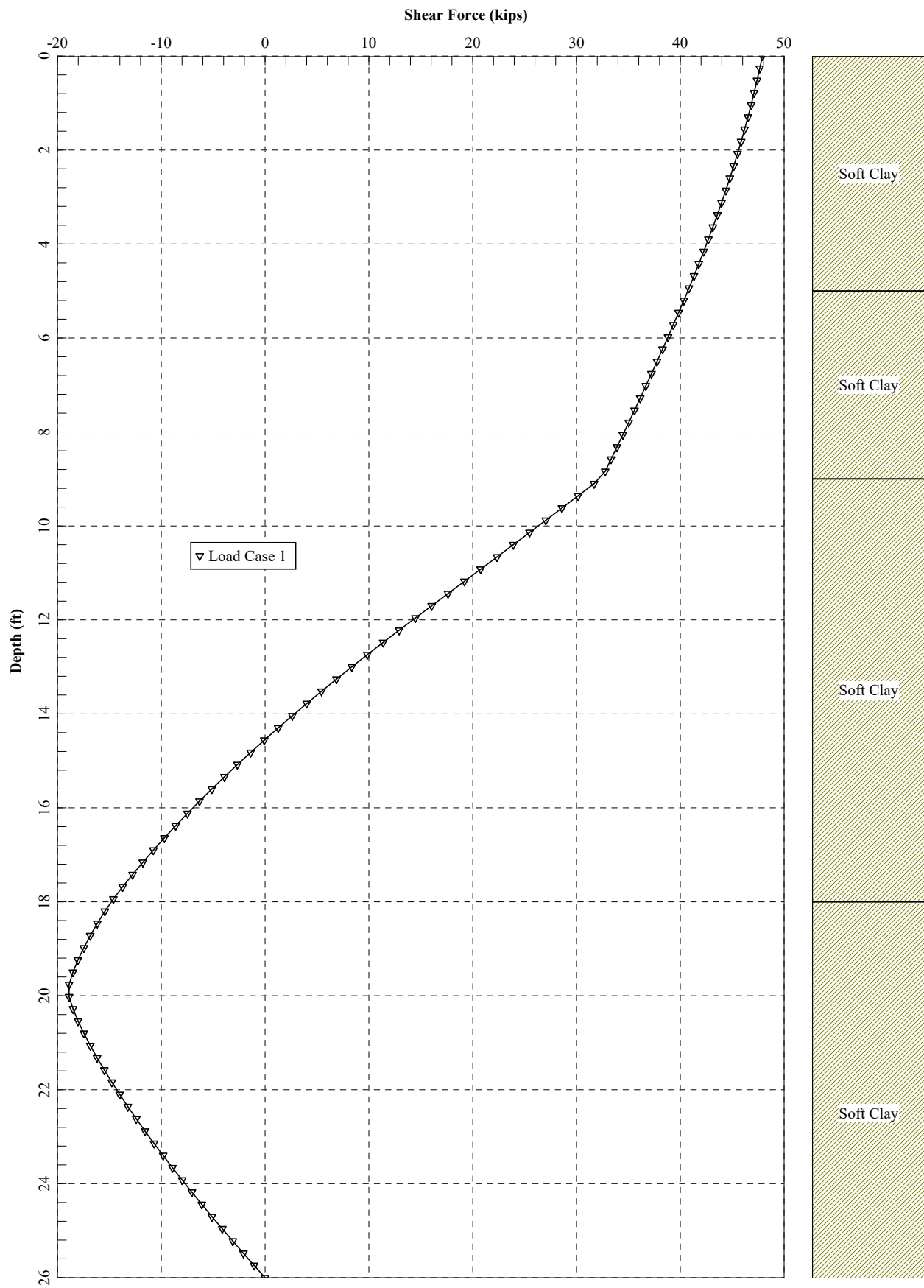
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.5000	S, rad	0.00	330000.	1.5009	0.00	47913.	-4553807.

Maximum pile-head deflection = 1.5008605098 inches
Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.







**Abutment 1 - 26 foot Pile
10 kip shear resistance at tip**

Abutment 1_14x89.lp9o

LPILE for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\\09 Jobs\0025900s\09.0025988.00 - Gorham - Bridge over Little River\09.0025988.01 - Final Design\Work\LPILE\Abutment
1\HP 14x89\26 Foot length - bb-101\ShearResistance at Tip\

Name of input data file:

Abutment 1_14x89.lp9d

Name of output report file:

Abutment 1_14x89.lp9o

Name of plot output file:

Abutment 1_14x89.lp9p

Page 1

Abutment 1_14x89.lp9o

Name of runtime message file:
Abutment 1_14x89.lp9r

Date and Time of Analysis

Date: August 21, 2019

Time: 11:43:24

Problem Title

Project Name: Little River Bridge #3112

Job Number: 09.0025988.00

Client: Hoyle, Tanner & Associates

Engineer: N. Williams

Description: Abutment 1

Program Options and Settings

Computational Options:

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- Use unfactored loads in computations (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 = 1000
- 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
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Analysis includes tip shear resistance for short pile or shaft
- 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 = 26.000 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.
 Page 3

Abutment 1_14x89.lp9o

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	26.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Strong H-Pile
Length of section	= 26.000000 ft
Flange Width	= 14.700000 in
Section Depth	= 13.800000 in
Flange Thickness	= 0.615000 in
Web Thickness	= 0.615000 in
Section Area	= 25.811550 sq. in
Moment of Inertia	= 888.177980 in^4
Elastic Modulus	= 29000000. psi

----- 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 5 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	0.0000	ft
Distance from top of pile to bottom of layer	=	5.000000	ft
Effective unit weight at top of layer	=	115.000000	pcf
Effective unit weight at bottom of layer	=	115.000000	pcf
Undrained cohesion at top of layer	=	400.000000	psf
Undrained cohesion at bottom of layer	=	400.000000	psf
Epsilon-50 at top of layer	=	0.020000	
Epsilon-50 at bottom of layer	=	0.020000	

Layer 2 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	5.000000	ft
Distance from top of pile to bottom of layer	=	9.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	400.000000	psf
Undrained cohesion at bottom of layer	=	400.000000	psf
Epsilon-50 at top of layer	=	0.020000	
Epsilon-50 at bottom of layer	=	0.020000	

Layer 3 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	9.000000	ft
Distance from top of pile to bottom of layer	=	18.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	900.000000	psf
Undrained cohesion at bottom of layer	=	900.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

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Layer 4 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	18.000000	ft
Distance from top of pile to bottom of layer	=	27.000000	ft
Effective unit weight at top of layer	=	43.000000	pcf
Effective unit weight at bottom of layer	=	43.000000	pcf
Undrained cohesion at top of layer	=	800.000000	psf
Undrained cohesion at bottom of layer	=	800.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

Layer 5 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	27.000000	ft
Distance from top of pile to bottom of layer	=	35.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	550.000000	psf
Undrained cohesion at bottom of layer	=	550.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

(Depth of the lowest soil layer extends 9.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	E50 or krm
1	Soft	0.00	115.0000	400.0000	0.02000
	Clay	5.0000	115.0000	400.0000	0.02000
2	Soft	5.0000	53.0000	400.0000	0.02000

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3	Clay	9.0000	53.0000	400.0000	0.02000
	Soft	9.0000	53.0000	900.0000	0.00800
4	Clay	18.0000	53.0000	900.0000	0.00800
	Soft	18.0000	43.0000	800.0000	0.00800
5	Clay	27.0000	43.0000	800.0000	0.00800
	Soft	27.0000	53.0000	550.0000	0.00800
	Clay	35.0000	53.0000	550.0000	0.00800

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
1	5	y = 1.500000 in	S = 0.0000 in/in	330000.	N.A.

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.

Shear Resistance Curve at Pile Tip

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Point No.	Displacement in	Tip Shear Force lbs
1	0.000	0.000
2	0.030	10000.000
3	5.000	10000.000

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:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	11611.
2	5.0000	5.0000	Yes	No	11611.	14844.
3	9.0000	9.0000	Yes	No	26455.	86736.
4	18.0000	18.0000	Yes	No	113191.	70575.
5	27.0000	27.0000	No	No	183766.	N.A.

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Notes: The F0 integral of Layer n+1 equals the sum of the F0 and F1 integrals for Layer n. Layering correction equivalent depths are computed only for soil types with both shallow-depth and deep-depth expressions for peak lateral load transfer. These soil types are soft and stiff clays, non-liquefied sands, and cemented c-phi soil.

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 1

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
Displacement of pile head = 1.500000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 330000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb*2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.5009	-4719319.	50349.	0.00	51839.	2.58E+10	-77.7070	80.7673	0.00
0.2600	1.5000	-4562346.	50089.	-5.62E-04	50540.	2.58E+10	-82.3757	171.3414	0.00
0.5200	1.4974	-4405605.	49825.	-0.00111	49243.	2.58E+10	-87.0091	181.2952	0.00
0.7800	1.4931	-4249162.	49546.	-0.00163	47948.	2.58E+10	-91.6030	191.4145	0.00
1.0400	1.4872	-4093081.	49253.	-0.00213	46657.	2.58E+10	-96.1532	201.7179	0.00
1.3000	1.4798	-3937426.	48946.	-0.00262	45369.	2.58E+10	-100.6557	212.2244	0.00
1.5600	1.4709	-3782259.	48625.	-0.00309	44085.	2.58E+10	-105.1066	222.9529	0.00
1.8200	1.4605	-3627643.	48291.	-0.00354	42805.	2.58E+10	-109.5020	233.9228	0.00
2.0800	1.4488	-3473641.	47942.	-0.00397	41531.	2.58E+10	-113.8385	245.1541	0.00
2.3400	1.4358	-3320314.	47580.	-0.00438	40262.	2.58E+10	-118.1123	256.6671	0.00
2.6000	1.4215	-3167723.	47205.	-0.00477	38999.	2.58E+10	-122.3200	268.4830	0.00
2.8600	1.4060	-3015927.	46817.	-0.00515	37743.	2.58E+10	-126.4582	280.6236	0.00
3.1200	1.3893	-2864986.	46416.	-0.00550	36494.	2.58E+10	-130.5237	293.1116	0.00
3.3800	1.3716	-2714959.	46003.	-0.00584	35252.	2.58E+10	-134.5132	305.9709	0.00
3.6400	1.3529	-2565902.	45577.	-0.00616	34019.	2.58E+10	-138.4236	319.2262	0.00
3.9000	1.3332	-2417873.	45139.	-0.00646	32794.	2.58E+10	-142.2519	332.9035	0.00
4.1600	1.3126	-2270927.	44689.	-0.00675	31578.	2.58E+10	-145.9950	347.0305	0.00

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4.4200	1.2911	-2125118.	44228.	-0.00701	30371.	2.58E+10	-149.6500	361.6358	0.00
4.6800	1.2688	-1980502.	43756.	-0.00726	29174.	2.58E+10	-153.2140	376.7502	0.00
4.9400	1.2458	-1837130.	43272.	-0.00749	27988.	2.58E+10	-156.6844	392.4059	0.00
5.2000	1.2221	-1695054.	42779.	-0.00771	26812.	2.58E+10	-159.3084	406.7228	0.00
5.4600	1.1977	-1554317.	42279.	-0.00790	25648.	2.58E+10	-161.6199	421.0180	0.00
5.7200	1.1728	-1414960.	41771.	-0.00808	24494.	2.58E+10	-163.8434	435.8910	0.00
5.9800	1.1473	-1277022.	41257.	-0.00825	23353.	2.58E+10	-165.9771	451.3767	0.00
6.2400	1.1213	-1140539.	40735.	-0.00839	22223.	2.58E+10	-168.0191	467.5124	0.00
6.5000	1.0949	-1005550.	40208.	-0.00852	21106.	2.58E+10	-169.9678	484.3380	0.00
6.7600	1.0681	-872091.	39675.	-0.00864	20002.	2.58E+10	-171.8214	501.8959	0.00
7.0200	1.0410	-740195.	39136.	-0.00873	18910.	2.58E+10	-173.5784	520.2320	0.00
7.2800	1.0136	-609896.	38592.	-0.00882	17832.	2.58E+10	-175.2372	539.3951	0.00
7.5400	0.9860	-481227.	38043.	-0.00888	16767.	2.58E+10	-176.7961	559.4377	0.00
7.8000	0.9582	-354219.	37489.	-0.00893	15716.	2.58E+10	-178.2539	580.4161	0.00
8.0600	0.9303	-228902.	36931.	-0.00897	14679.	2.58E+10	-179.6090	602.3908	0.00
8.3200	0.9022	-105305.	36368.	-0.00899	13656.	2.58E+10	-180.8601	625.4267	0.00
8.5800	0.8742	16544.	35802.	-0.00899	12922.	2.58E+10	-182.0061	649.5935	0.00
8.8400	0.8461	136620.	35233.	-0.00898	13916.	2.58E+10	-183.0457	674.9663	0.00
9.1000	0.8181	254897.	34182.	-0.00896	14894.	2.58E+10	-490.7154	1871.	0.00
9.3600	0.7902	368365.	32647.	-0.00892	15833.	2.58E+10	-492.8270	1946.	0.00
9.6200	0.7624	476990.	31107.	-0.00887	16732.	2.58E+10	-494.6494	2024.	0.00
9.8800	0.7349	580741.	29561.	-0.00881	17591.	2.58E+10	-496.1804	2107.	0.00
10.1400	0.7075	679589.	28011.	-0.00873	18409.	2.58E+10	-497.4178	2194.	0.00
10.4000	0.6804	773510.	26458.	-0.00864	19186.	2.58E+10	-498.3596	2285.	0.00
10.6600	0.6536	862483.	24902.	-0.00854	19922.	2.58E+10	-499.0038	2382.	0.00
10.9200	0.6271	946492.	23345.	-0.00843	20618.	2.58E+10	-499.3484	2485.	0.00
11.1800	0.6009	1025521.	21787.	-0.00831	21272.	2.58E+10	-499.3916	2593.	0.00
11.4400	0.5752	1099561.	20229.	-0.00819	21884.	2.58E+10	-499.1315	2708.	0.00
11.7000	0.5498	1168606.	18672.	-0.00805	22456.	2.58E+10	-498.5664	2829.	0.00
11.9600	0.5250	1232651.	17118.	-0.00790	22986.	2.58E+10	-497.6943	2958.	0.00
12.2200	0.5005	1291698.	15572.	-0.00775	23474.	2.58E+10	-493.6708	3077.	0.00
12.4800	0.4766	1345778.	14044.	-0.00759	23922.	2.58E+10	-485.6723	3179.	0.00
12.7400	0.4532	1394963.	12541.	-0.00742	24329.	2.58E+10	-477.5801	3288.	0.00
13.0000	0.4303	1439325.	11064.	-0.00725	24696.	2.58E+10	-469.3957	3404.	0.00
13.2600	0.4079	1478938.	9612.	-0.00708	25024.	2.58E+10	-461.1205	3527.	0.00
13.5200	0.3861	1513878.	8187.	-0.00689	25313.	2.58E+10	-452.7557	3659.	0.00
13.7800	0.3649	1544221.	6787.	-0.00671	25564.	2.58E+10	-444.3023	3799.	0.00
14.0400	0.3442	1570047.	5414.	-0.00652	25778.	2.58E+10	-435.7607	3950.	0.00
14.3000	0.3242	1591436.	4068.	-0.00633	25955.	2.58E+10	-427.1312	4111.	0.00
14.5600	0.3047	1608468.	2749.	-0.00614	26096.	2.58E+10	-418.4135	4284.	0.00
14.8200	0.2859	1621226.	1458.	-0.00594	26201.	2.58E+10	-409.6072	4470.	0.00

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15.0800	0.2677	1629795.	193.4768	-0.00574	26272.	2.58E+10	-400.7110	4671.	0.00
15.3400	0.2501	1634260.	-1043.	-0.00555	26309.	2.58E+10	-391.7230	4888.	0.00
15.6000	0.2331	1634708.	-2251.	-0.00535	26313.	2.58E+10	-382.6408	5122.	0.00
15.8600	0.2167	1631227.	-3430.	-0.00515	26284.	2.58E+10	-373.4610	5377.	0.00
16.1200	0.2009	1623908.	-4581.	-0.00495	26223.	2.58E+10	-364.1790	5655.	0.00
16.3800	0.1858	1612841.	-5703.	-0.00476	26132.	2.58E+10	-354.7892	5958.	0.00
16.6400	0.1712	1598119.	-6795.	-0.00456	26010.	2.58E+10	-345.2844	6291.	0.00
16.9000	0.1573	1579836.	-7857.	-0.00437	25859.	2.58E+10	-335.6553	6657.	0.00
17.1600	0.1440	1558090.	-8889.	-0.00418	25679.	2.58E+10	-325.8907	7062.	0.00
17.4200	0.1312	1532976.	-9890.	-0.00399	25471.	2.58E+10	-315.9763	7512.	0.00
17.6800	0.1191	1504596.	-10860.	-0.00381	25236.	2.58E+10	-305.8943	8015.	0.00
17.9400	0.1075	1473050.	-11799.	-0.00363	24975.	2.58E+10	-295.6220	8582.	0.00
18.2000	0.09643	1438442.	-12655.	-0.00345	24689.	2.58E+10	-285.4500	8200.	0.00
18.4600	0.08593	1401189.	-13431.	-0.00328	24380.	2.58E+10	-274.8984	8855.	0.00
18.7200	0.07596	1361386.	-14177.	-0.00311	24051.	2.58E+10	-264.0787	9614.	0.00
18.9800	0.06651	1319135.	-14891.	-0.00295	23701.	2.58E+10	-253.9367	10505.	0.00
19.2400	0.05756	1274539.	-15574.	-0.00279	23332.	2.58E+10	-243.3998	11568.	0.00
19.5000	0.04908	1227707.	-16222.	-0.00264	22945.	2.58E+10	-232.3690	12864.	0.00
19.7600	0.04107	1178752.	-16835.	-0.00250	22540.	2.58E+10	-221.7040	14487.	0.00
20.0200	0.03351	1127794.	-17411.	-0.00236	22118.	2.58E+10	-210.1981	16592.	0.00
20.2800	0.02637	1074960.	-17945.	-0.00222	21681.	2.58E+10	-198.5274	19466.	0.00
20.5400	0.01964	1020391.	-18435.	-0.00210	21229.	2.58E+10	-186.1396	23694.	0.00
20.8000	0.01329	964243.	-18872.	-0.00198	20764.	2.58E+10	-173.9588	30738.	0.00
21.0600	0.00731	906699.	-19243.	-0.00186	20288.	2.58E+10	-161.3243	45799.	0.00
21.3200	0.00167	847998.	-19513.	-0.00176	19802.	2.58E+10	-148.7556	122661.	0.00
21.5800	-0.00365	788551.	-19484.	-0.00166	19311.	2.58E+10	-135.9714	72717.	0.00
21.8400	-0.00867	729833.	-19174.	-0.00156	18825.	2.58E+10	-122.4705	40852.	0.00
22.1000	-0.01341	672128.	-18792.	-0.00148	18347.	2.58E+10	-109.2682	30540.	0.00
22.3600	-0.01790	615617.	-18362.	-0.00140	17879.	2.58E+10	-95.5443	25193.	0.00
22.6200	-0.02216	560437.	-17894.	-0.00133	17423.	2.58E+10	-81.2059	21853.	0.00
22.8800	-0.02620	506697.	-17396.	-0.00127	16978.	2.58E+10	-66.1338	19542.	0.00
23.1400	-0.03006	454492.	-16872.	-0.00121	16546.	2.58E+10	-50.8193	17834.	0.00
23.4000	-0.03374	403903.	-16325.	-0.00116	16127.	2.58E+10	-34.5701	16512.	0.00
23.6600	-0.03727	355002.	-15759.	-0.00111	15723.	2.58E+10	-18.5940	15452.	0.00
23.9200	-0.04067	307853.	-15174.	-0.00107	15333.	2.58E+10	-2.0396	14580.	0.00
24.1800	-0.04395	262516.	-14574.	-0.00104	14957.	2.58E+10	195.0179	13845.	0.00
24.4400	-0.04713	219045.	-13958.	-0.00101	14598.	2.58E+10	199.6141	13215.	0.00
24.7000	-0.05023	177489.	-13329.	-9.82E-04	14254.	2.58E+10	203.8960	12666.	0.00
24.9600	-0.05326	137896.	-12686.	-9.63E-04	13926.	2.58E+10	207.9179	12181.	0.00
25.2200	-0.05623	100310.	-12032.	-9.49E-04	13615.	2.58E+10	211.7245	11747.	0.00
25.4800	-0.05918	64772.	-11365.	-9.39E-04	13321.	2.58E+10	215.3530	11354.	0.00

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Abutment 1_14x89.lp9o									
25.7400	-0.06209	31323.	-10688.	-9.33E-04	13044.	2.58E+10	218.8344	10996.	0.00
26.0000	-0.06500	0.00	-10000.	-9.31E-04	12785.	2.58E+10	222.1949	5333.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.50089179 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -4719319. inch-lbs
 Maximum shear force = 50349. 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 = 23
 Number of zero deflection points = 1

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

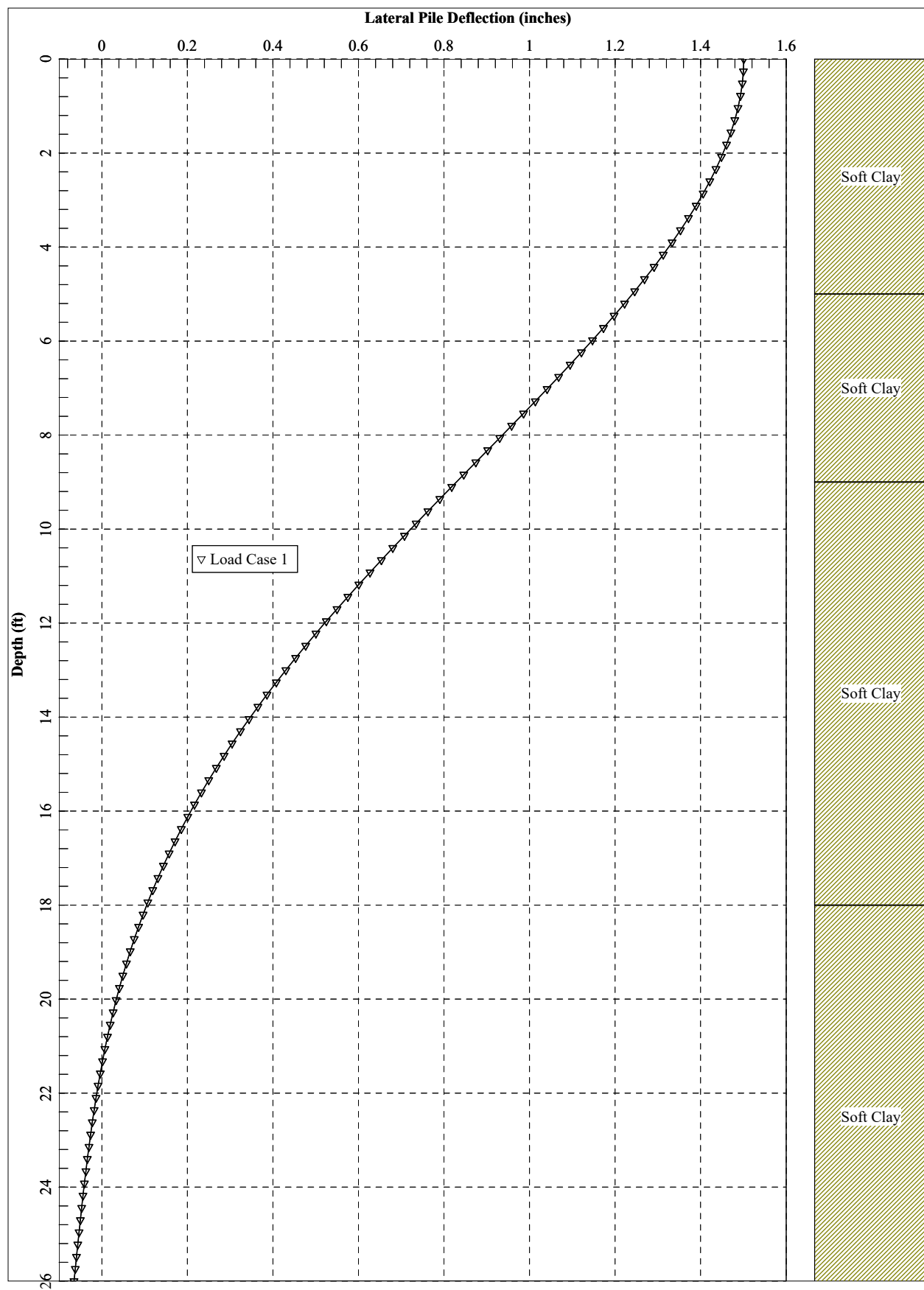
Load Case No.	Load Type 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.5000	S, rad	0.00	330000.	1.5009	0.00	50349. -4719319.

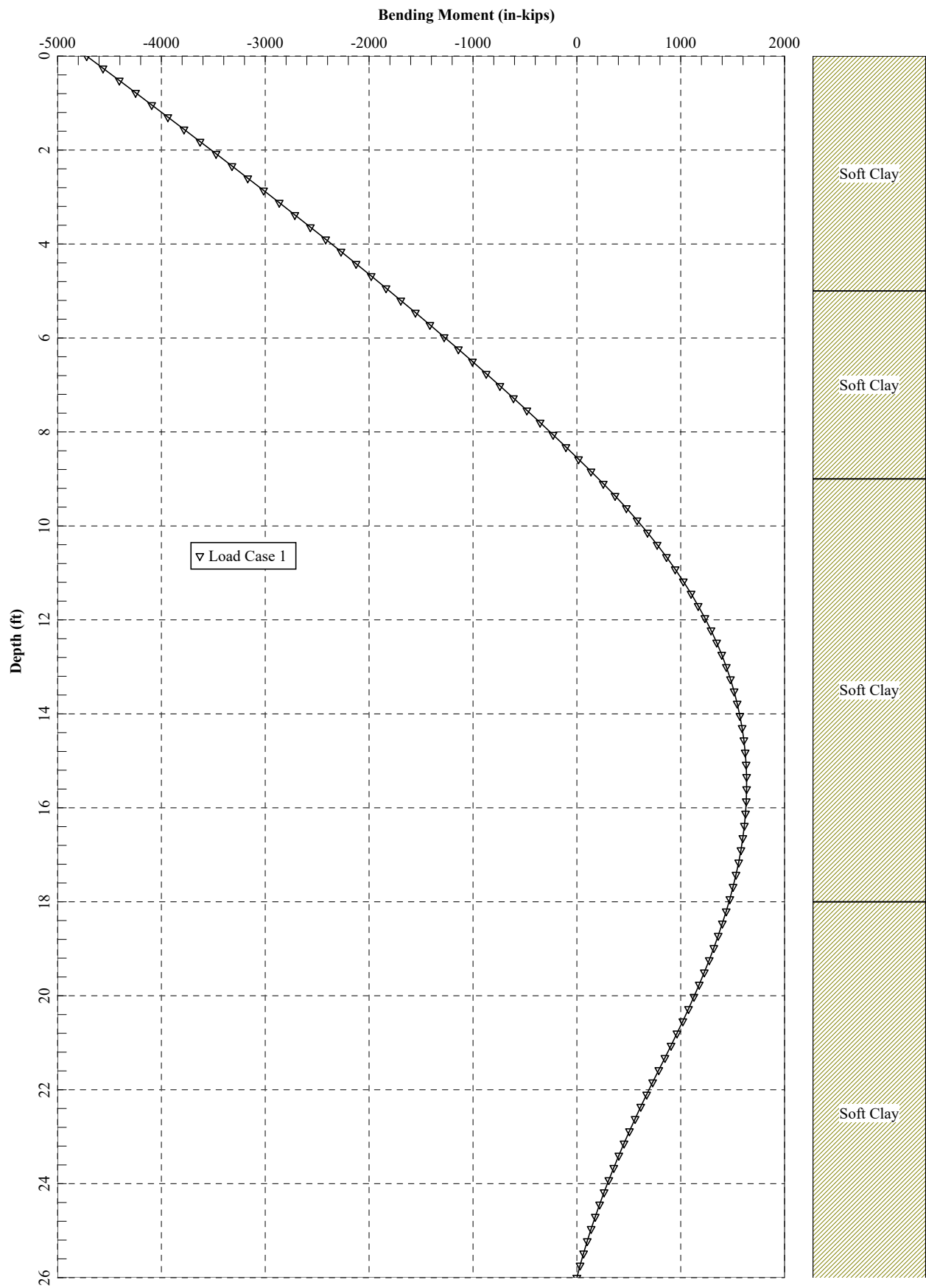
Maximum pile-head deflection = 1.5008917858 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

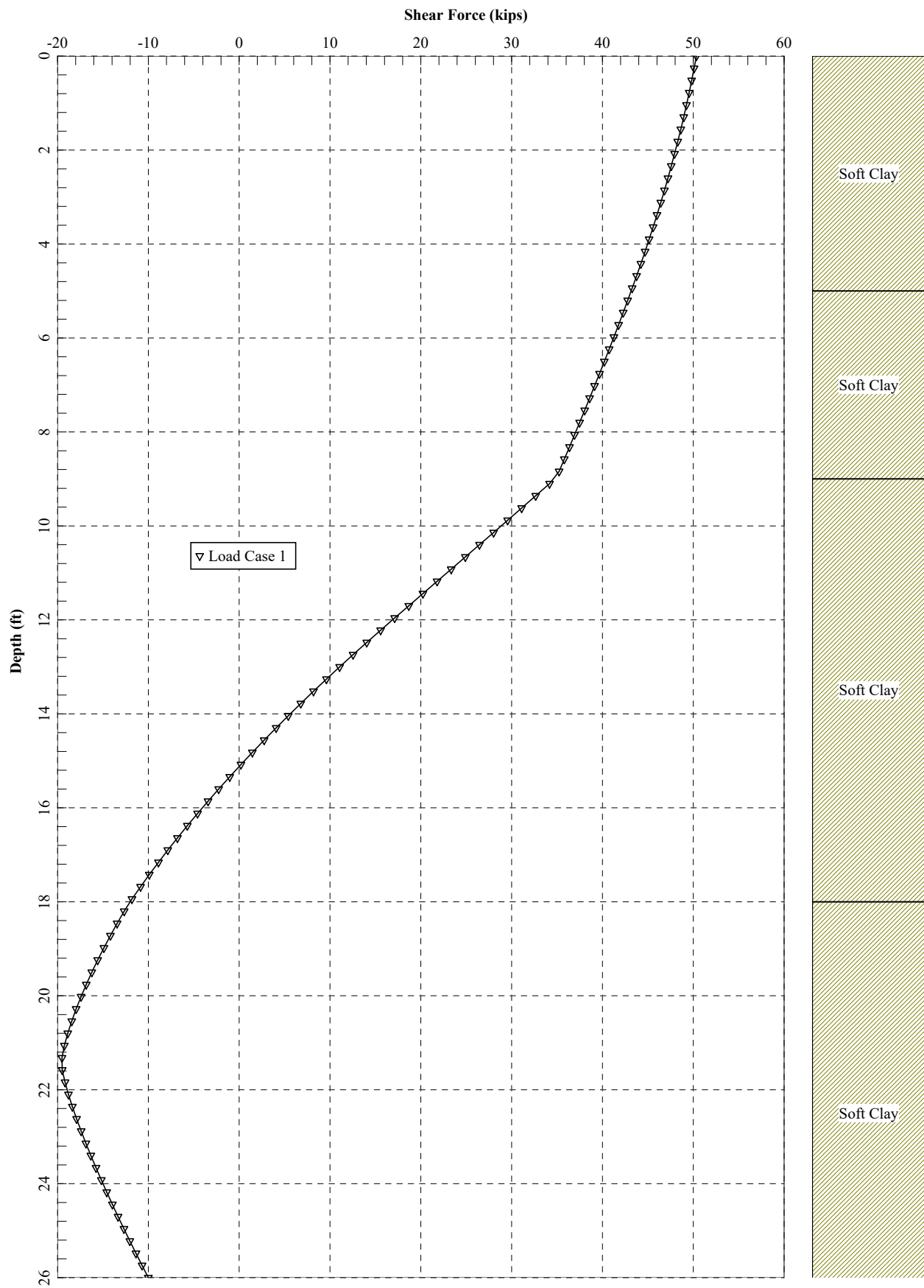
Page 12

The analysis ended normally.

Abutment 1_14x89.1p9o







Abutment 1 - 26 foot Pile
Embedded 3" into rock

Abutment 1_14x89.lp9o

LPILE for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\\09 Jobs\0025900s\09.0025988.00 - Gorham - Bridge over Little River\09.0025988.01 - Final Design\Work\LPILE\Abutment
1\HP 14x89\26 Foot length - bb-101\3-inches Embedded into rock\

Name of input data file:

Abutment 1_14x89.lp9d

Name of output report file:

Abutment 1_14x89.lp9o

Name of plot output file:

Abutment 1_14x89.lp9p

Page 1

Abutment 1_14x89.lp9o

Name of runtime message file:

Abutment 1_14x89.lp9r

Date and Time of Analysis

Date: August 21, 2019

Time: 12:50:27

Problem Title

Project Name: Little River Bridge #3112

Job Number: 09.0025988.00

Client: Hoyle, Tanner & Associates

Engineer: N. Williams

Description: Abutment 1

Program Options and Settings

Computational Options:

Page 2

Abutment 1_14x89.lp9o

- Use unfactored loads in computations (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
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- 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	=	26.250 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.

Page 3

Abutment 1_14x89.lp9o

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	26.250	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile		
Cross-sectional Shape	=	Strong H-Pile
Length of section	=	26.250000 ft
Flange Width	=	14.700000 in
Section Depth	=	13.800000 in
Flange Thickness	=	0.615000 in
Web Thickness	=	0.615000 in
Section Area	=	25.811550 sq. in
Moment of Inertia	=	888.177980 in^4
Elastic Modulus	=	29000000. psi

 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 5 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 0.0000 ft
 Distance from top of pile to bottom of layer = 5.000000 ft
 Effective unit weight at top of layer = 115.000000 pcf
 Effective unit weight at bottom of layer = 115.000000 pcf
 Undrained cohesion at top of layer = 400.000000 psf
 Undrained cohesion at bottom of layer = 400.000000 psf
 Epsilon-50 at top of layer = 0.020000
 Epsilon-50 at bottom of layer = 0.020000

Layer 2 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 5.000000 ft
 Distance from top of pile to bottom of layer = 9.000000 ft
 Effective unit weight at top of layer = 53.000000 pcf
 Effective unit weight at bottom of layer = 53.000000 pcf
 Undrained cohesion at top of layer = 400.000000 psf
 Undrained cohesion at bottom of layer = 400.000000 psf
 Epsilon-50 at top of layer = 0.020000
 Epsilon-50 at bottom of layer = 0.020000

Layer 3 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 9.000000 ft
 Distance from top of pile to bottom of layer = 18.000000 ft
 Effective unit weight at top of layer = 53.000000 pcf
 Effective unit weight at bottom of layer = 53.000000 pcf
 Undrained cohesion at top of layer = 900.000000 psf
 Undrained cohesion at bottom of layer = 900.000000 psf
 Epsilon-50 at top of layer = 0.008000
 Epsilon-50 at bottom of layer = 0.008000

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Layer 4 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer = 18.000000 ft
 Distance from top of pile to bottom of layer = 26.000000 ft
 Effective unit weight at top of layer = 43.000000 pcf
 Effective unit weight at bottom of layer = 43.000000 pcf
 Undrained cohesion at top of layer = 800.000000 psf
 Undrained cohesion at bottom of layer = 800.000000 psf
 Epsilon-50 at top of layer = 0.008000
 Epsilon-50 at bottom of layer = 0.008000

Layer 5 is strong rock (vuggy limestone)

Distance from top of pile to top of layer = 26.000000 ft
 Distance from top of pile to bottom of layer = 35.000000 ft
 Effective unit weight at top of layer = 90.000000 pcf
 Effective unit weight at bottom of layer = 90.000000 pcf
 Uniaxial compressive strength at top of layer = 4000. psi
 Uniaxial compressive strength at bottom of layer = 4000. psi

(Depth of the lowest soil layer extends 8.750 ft below the pile tip)

 Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	Uniaxial qu psi	E50 or krm
1	Soft	0.00	115.0000	400.0000	--	0.02000
	Clay	5.0000	115.0000	400.0000	--	0.02000
2	Soft	5.0000	53.0000	400.0000	--	0.02000
	Clay	9.0000	53.0000	400.0000	--	0.02000
3	Soft	9.0000	53.0000	900.0000	--	0.00800

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Abutment 1_14x89.1p9o						
4	Clay	18.0000	53.0000	900.0000	--	0.00800
	Soft	18.0000	43.0000	800.0000	--	0.00800
5	Clay	26.0000	43.0000	800.0000	--	0.00800
	Strong Rock	26.0000	90.0000	--	4000.	--
	(Vuggy Limestone)	35.0000	90.0000	--	4000.	--

 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
1	5	y = 1.500000 in	S = 0.0000 in/in	330000.	N.A.

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

Page 7

Abutment 1_14x89.1p9o
 Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

 Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	11611.
2	5.0000	5.0000	Yes	No	11611.	14844.
3	9.0000	9.0000	Yes	No	26455.	86736.
4	18.0000	18.0000	Yes	No	113191.	155247.
5	26.0000	26.0000	No	Yes	N.A.	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)

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Abutment 1_14x89.lp9o
Displacement of pile head = 1.50000 inches
Rotation of pile head = 0.000E+00 radians
Axial load on pile head = 330000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.5009	-4797192.	51262.	0.00	52484.	2.58E+10	-77.7075	81.5427	0.00
0.2625	1.5000	-4635831.	50999.	-5.77E-04	51148.	2.58E+10	-82.4207	173.0835	0.00
0.5250	1.4973	-4474698.	50732.	-0.00113	49815.	2.58E+10	-87.0973	183.2355	0.00
0.7875	1.4929	-4313861.	50450.	-0.00167	48484.	2.58E+10	-91.7329	193.5609	0.00
1.0500	1.4868	-4153386.	50154.	-0.00219	47156.	2.58E+10	-96.3231	204.0796	0.00
1.3125	1.4791	-3993338.	49844.	-0.00269	45831.	2.58E+10	-100.8636	214.8116	0.00
1.5750	1.4698	-3833783.	49519.	-0.00317	44511.	2.58E+10	-105.3505	225.7769	0.00
1.8375	1.4591	-3674787.	49180.	-0.00362	43195.	2.58E+10	-109.7797	236.9963	0.00
2.1000	1.4470	-3516413.	48827.	-0.00406	41885.	2.58E+10	-114.1475	248.4908	0.00
2.3625	1.4335	-3358724.	48461.	-0.00449	40580.	2.58E+10	-118.4501	260.2822	0.00
2.6250	1.4187	-3201783.	48081.	-0.00489	39281.	2.58E+10	-122.6839	272.3930	0.00
2.8875	1.4027	-3045653.	47688.	-0.00527	37989.	2.58E+10	-126.8454	284.8467	0.00
3.1500	1.3855	-2890394.	47282.	-0.00563	36704.	2.58E+10	-130.9310	297.6675	0.00
3.4125	1.3673	-2736067.	46864.	-0.00598	35427.	2.58E+10	-134.9375	310.8811	0.00
3.6750	1.3479	-2582731.	46432.	-0.00630	34158.	2.58E+10	-138.8615	324.5140	0.00
3.9375	1.3276	-2430445.	45989.	-0.00661	32898.	2.58E+10	-142.6998	338.5945	0.00
4.2000	1.3063	-2279266.	45533.	-0.00690	31647.	2.58E+10	-146.4493	353.1522	0.00
4.4625	1.2841	-2129250.	45066.	-0.00716	30405.	2.58E+10	-150.1069	368.2184	0.00
4.7250	1.2611	-1980452.	44588.	-0.00742	29174.	2.58E+10	-153.6696	383.8263	0.00
4.9875	1.2374	-1832928.	44098.	-0.00765	27953.	2.58E+10	-157.1345	400.0111	0.00
5.2500	1.2130	-1686730.	43600.	-0.00786	26743.	2.58E+10	-159.5637	414.3822	0.00
5.5125	1.1879	-1541901.	43093.	-0.00806	25545.	2.58E+10	-161.8560	429.2161	0.00
5.7750	1.1622	-1398481.	42580.	-0.00824	24358.	2.58E+10	-164.0563	444.6686	0.00
6.0375	1.1359	-1256512.	42060.	-0.00840	23183.	2.58E+10	-166.1628	460.7780	0.00
6.3000	1.1092	-1116032.	41533.	-0.00855	22021.	2.58E+10	-168.1736	477.5856	0.00
6.5625	1.0821	-977078.	41001.	-0.00868	20871.	2.58E+10	-170.0868	495.1355	0.00
6.8250	1.0546	-839689.	40462.	-0.00879	19734.	2.58E+10	-171.9006	513.4750	0.00
7.0875	1.0267	-703898.	39918.	-0.00888	18610.	2.58E+10	-173.6133	532.6550	0.00
7.3500	0.9986	-569740.	39368.	-0.00896	17500.	2.58E+10	-175.2232	552.7302	0.00
7.6125	0.9703	-437248.	38814.	-0.00902	16403.	2.58E+10	-176.7286	573.7593	0.00
7.8750	0.9418	-306455.	38255.	-0.00907	15321.	2.58E+10	-178.1281	595.8055	0.00
8.1375	0.9131	-177390.	37692.	-0.00910	14253.	2.58E+10	-179.4200	618.9371	0.00
8.4000	0.8844	-50082.	37125.	-0.00911	13199.	2.58E+10	-180.6030	643.2275	0.00

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Abutment 1_14x89.lp9o									
8.6625	0.8557	75439.	36554.	-0.00911	13409.	2.58E+10	-181.6757	668.7557	0.00
8.9250	0.8271	199149.	35981.	-0.00909	14433.	2.58E+10	-182.6369	695.6071	0.00
9.1875	0.7985	321020.	34922.	-0.00906	15442.	2.58E+10	-489.3726	1931.	0.00
9.4500	0.7700	437996.	33378.	-0.00901	16410.	2.58E+10	-491.2469	2010.	0.00
9.7125	0.7417	550041.	31828.	-0.00895	17337.	2.58E+10	-492.8178	2093.	0.00
9.9750	0.7136	657126.	30273.	-0.00888	18223.	2.58E+10	-494.0828	2181.	0.00
10.2375	0.6857	759225.	28716.	-0.00879	19068.	2.58E+10	-495.0395	2274.	0.00
10.5000	0.6582	856316.	27155.	-0.00869	19871.	2.58E+10	-495.6860	2372.	0.00
10.7625	0.6309	948379.	25593.	-0.00858	20633.	2.58E+10	-496.0201	2476.	0.00
11.0250	0.6041	1035400.	24031.	-0.00846	21353.	2.58E+10	-496.0398	2587.	0.00
11.2875	0.5776	1117367.	22469.	-0.00833	22032.	2.58E+10	-495.7433	2703.	0.00
11.5500	0.5516	1194274.	20908.	-0.00819	22668.	2.58E+10	-495.1288	2828.	0.00
11.8125	0.5260	1266115.	19350.	-0.00804	23263.	2.58E+10	-494.1946	2959.	0.00
12.0750	0.5009	1332892.	17795.	-0.00788	23815.	2.58E+10	-492.9392	3100.	0.00
12.3375	0.4764	1394609.	16254.	-0.00771	24326.	2.58E+10	-485.6037	3211.	0.00
12.6000	0.4523	1451329.	14737.	-0.00754	24795.	2.58E+10	-477.2976	3324.	0.00
12.8625	0.4289	1503130.	13247.	-0.00736	25224.	2.58E+10	-468.8957	3444.	0.00
13.1250	0.4060	1550086.	11783.	-0.00717	25613.	2.58E+10	-460.4004	3572.	0.00
13.3875	0.3837	1592277.	10347.	-0.00698	25962.	2.58E+10	-451.8139	3709.	0.00
13.6500	0.3620	1629783.	8937.	-0.00678	26272.	2.58E+10	-443.1385	3856.	0.00
13.9125	0.3410	1662684.	7555.	-0.00658	26544.	2.58E+10	-434.3761	4013.	0.00
14.1750	0.3205	1691064.	6201.	-0.00638	26779.	2.58E+10	-425.5288	4182.	0.00
14.4375	0.3008	1715006.	4874.	-0.00617	26977.	2.58E+10	-416.5985	4363.	0.00
14.7000	0.2817	1734597.	3576.	-0.00596	27139.	2.58E+10	-407.5868	4558.	0.00
14.9625	0.2632	1749923.	2307.	-0.00574	27266.	2.58E+10	-398.4954	4768.	0.00
15.2250	0.2455	1761073.	1066.	-0.00553	27358.	2.58E+10	-389.3258	4996.	0.00
15.4875	0.2284	1768135.	-145.9014	-0.00531	27417.	2.58E+10	-380.0792	5242.	0.00
15.7500	0.2120	1771202.	-1328.	-0.00510	27442.	2.58E+10	-370.7569	5509.	0.00
16.0125	0.1963	1770364.	-2482.	-0.00488	27435.	2.58E+10	-361.3597	5799.	0.00
16.2750	0.1813	1765716.	-3605.	-0.00466	27397.	2.58E+10	-351.8885	6115.	0.00
16.5375	0.1669	1757352.	-4698.	-0.00445	27328.	2.58E+10	-342.3436	6461.	0.00
16.8000	0.1532	1745367.	-5762.	-0.00424	27229.	2.58E+10	-332.7252	6840.	0.00
17.0625	0.1402	1729859.	-6794.	-0.00402	27100.	2.58E+10	-323.0331	7257.	0.00
17.3250	0.1279	1710926.	-7797.	-0.00381	26944.	2.58E+10	-313.2667	7716.	0.00
17.5875	0.1162	1688667.	-8768.	-0.00360	26759.	2.58E+10	-303.4248	8225.	0.00
17.8500	0.1052	1663182.	-9708.	-0.00340	26548.	2.58E+10	-293.5055	8791.	0.00
18.1125	0.09479	1634574.	-10567.	-0.00320	26312.	2.58E+10	-282.0057	9375.	0.00
18.3750	0.08503	1603257.	-11347.	-0.00300	26053.	2.58E+10	-270.0435	9904.	0.00
18.6375	0.07589	1569326.	-12098.	-0.00281	25772.	2.58E+10	-257.0029	10413.	0.00
18.9000	0.06735	1532872.	-12821.	-0.00262	25470.	2.58E+10	-244.8788	10918.	0.00
19.1625	0.05940	1493993.	-13515.	-0.00243	25148.	2.58E+10	-232.6640	11436.	0.00

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19.4250	0.05203	1452783.	-14180.	-0.00225	24807.	2.58E+10	-206.3498	12492.	0.00
19.6875	0.04522	1409341.	-14815.	-0.00208	24448.	2.58E+10	-196.9244	13717.	0.00
19.9500	0.03895	1363767.	-15420.	-0.00191	24071.	2.58E+10	-187.3727	15152.	0.00
20.2125	0.03321	1316159.	-15995.	-0.00174	23677.	2.58E+10	-177.6746	16852.	0.00
20.4750	0.02798	1266622.	-16539.	-0.00158	23267.	2.58E+10	-167.8030	18894.	0.00
20.7375	0.02323	1215258.	-17052.	-0.00143	22842.	2.58E+10	-157.7207	21389.	0.00
21.0000	0.01895	1162175.	-17532.	-0.00129	22402.	2.58E+10	-147.3754	24499.	0.00
21.2625	0.01512	1107482.	-17980.	-0.00115	21950.	2.58E+10	-136.6909	28483.	0.00
21.5250	0.01171	1051291.	-18393.	-0.00102	21485.	2.58E+10	-125.5513	33768.	0.00
21.7875	0.00871	993721.	-18770.	-8.92E-04	21008.	2.58E+10	-113.7685	41136.	0.00
22.0500	0.00609	934896.	-19108.	-7.74E-04	20522.	2.58E+10	-101.0103	52207.	0.00
22.3125	0.00384	874950.	-19403.	-6.63E-04	20026.	2.58E+10	-86.6013	71086.	0.00
22.5750	0.00192	814034.	-19648.	-5.60E-04	19521.	2.58E+10	-68.7724	112976.	0.00
22.8375	3.11E-04	752331.	-19816.	-4.64E-04	19011.	2.58E+10	-37.7989	382755.	0.00
23.1000	-0.00101	690158.	-19788.	-3.76E-04	18496.	2.58E+10	55.2221	172991.	0.00
23.3625	-0.00206	628445.	-19591.	-2.95E-04	17986.	2.58E+10	70.1952	107531.	0.00
23.6250	-0.00286	567349.	-19357.	-2.22E-04	17480.	2.58E+10	78.4326	86237.	0.00
23.8875	-0.00346	506958.	-19102.	-1.56E-04	16980.	2.58E+10	83.5015	76130.	0.00
24.1500	-0.00385	447332.	-18834.	-9.80E-05	16487.	2.58E+10	86.5780	70840.	0.00
24.4125	-0.00407	388508.	-18559.	-4.69E-05	16000.	2.58E+10	88.2216	68242.	0.00
24.6750	-0.00415	330510.	-18280.	-2.90E-06	15520.	2.58E+10	88.7495	67444.	0.00
24.9375	-0.00409	273351.	-18001.	3.40E-05	15047.	2.58E+10	88.3630	68046.	0.00
25.2000	-0.00393	217033.	-17724.	6.40E-05	14581.	2.58E+10	87.2008	69881.	0.00
25.4625	-0.00369	161554.	-17453.	8.72E-05	14122.	2.58E+10	85.3659	72927.	0.00
25.7250	-0.00338	106900.	-17188.	1.04E-04	13670.	2.58E+10	82.9427	77262.	0.00
25.9875	-0.00303	53057.	-16931.	1.13E-04	13224.	2.58E+10	80.0086	83047.	0.00
26.2500	-0.00267	0.00	0.00	1.17E-04	12785.	2.58E+10	10670.	6300000.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.50092402 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -4797192. inch-lbs
 Maximum shear force = 51262. 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 = 21
 Number of zero deflection points = 1

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Abutment 1_14x89.lp9o

Summary of Pile-head Responses for Conventional Analyses

Definitions of Pile-head Loading Conditions:

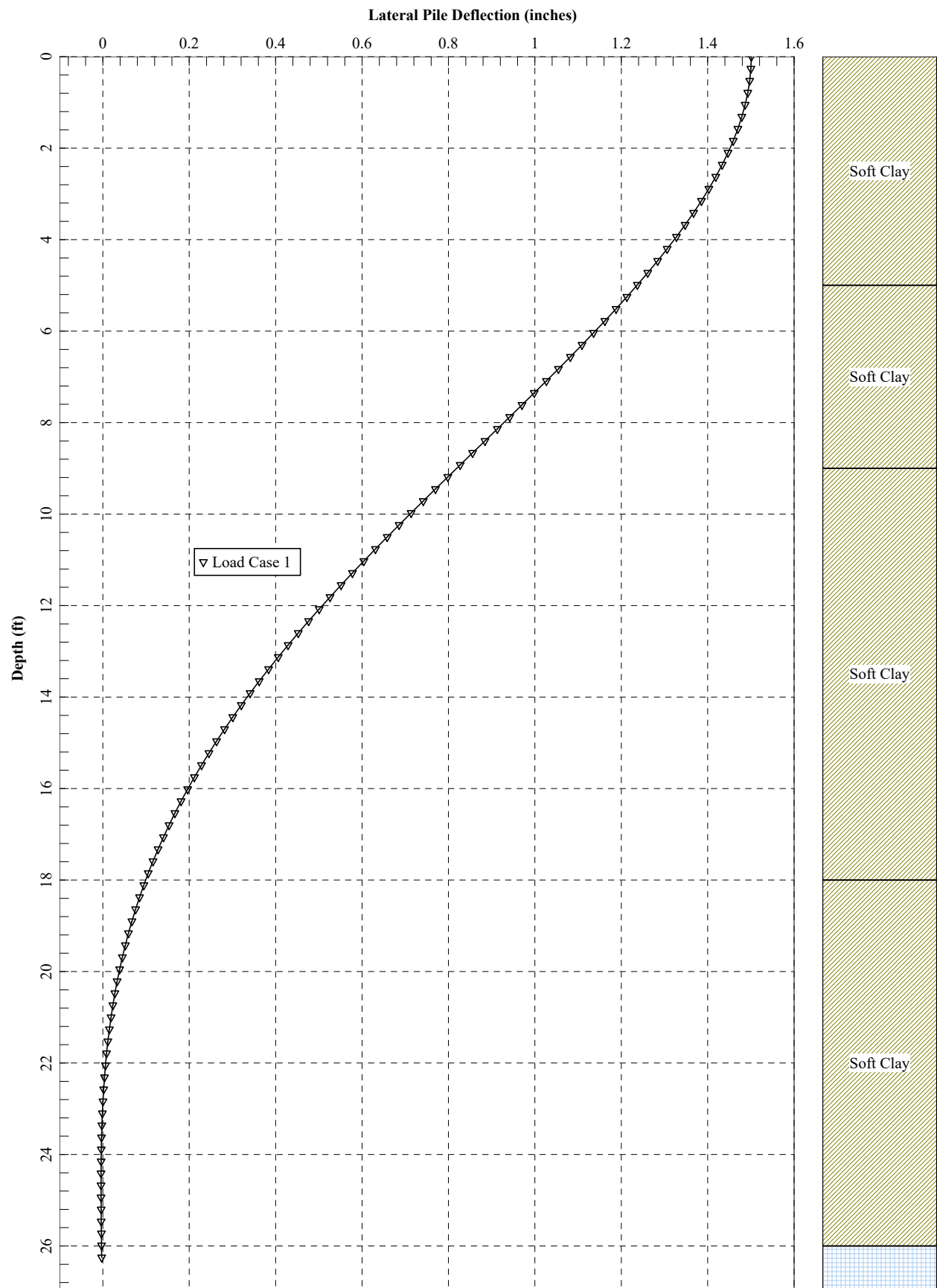
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

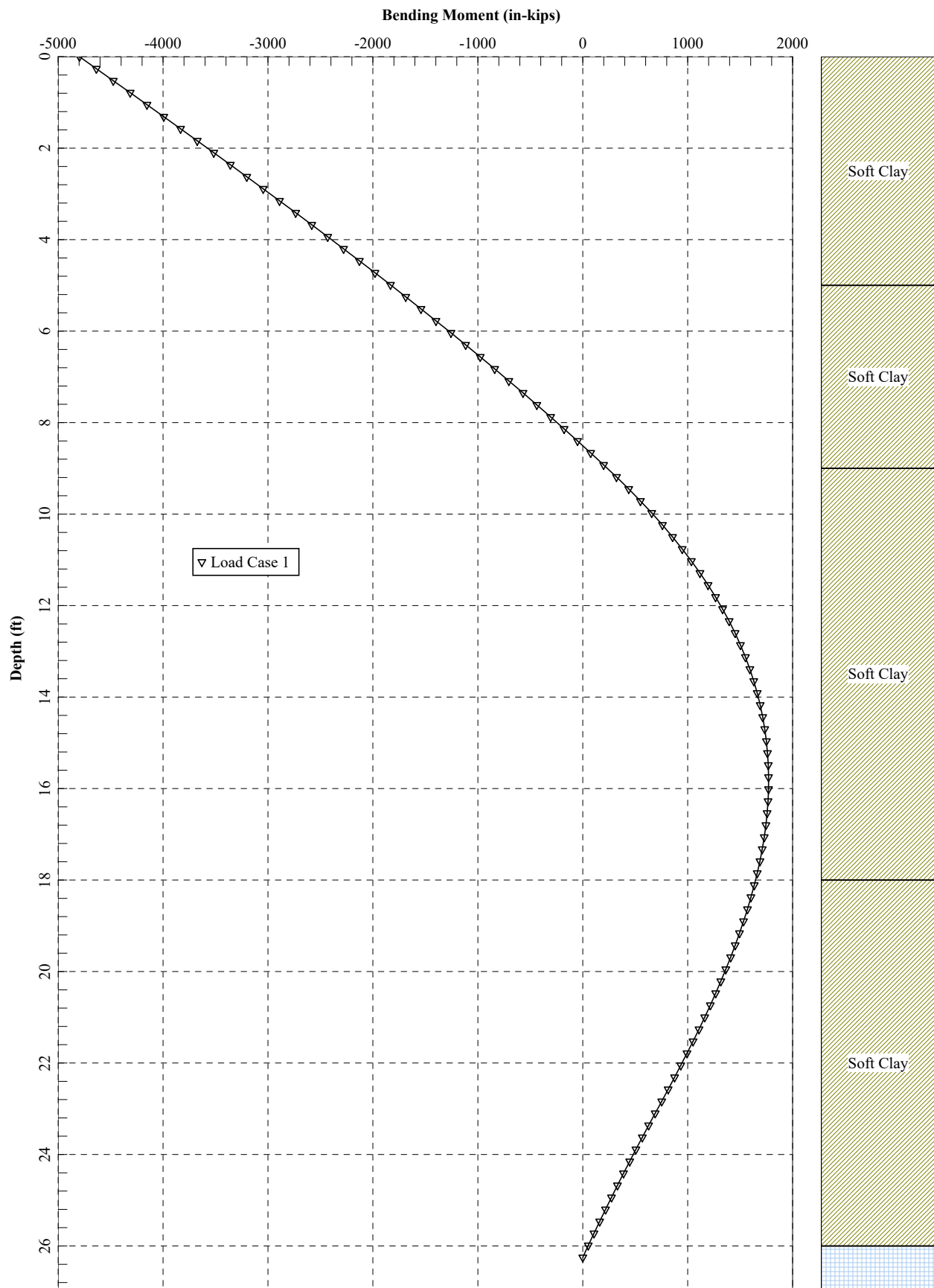
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.5000	S, rad	0.00	330000.	1.5009	0.00	51262.	-4797192.

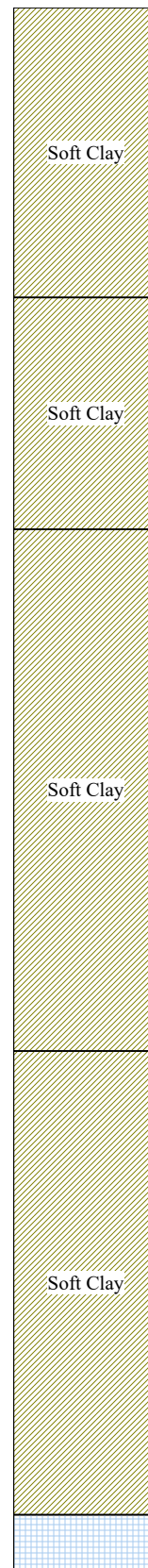
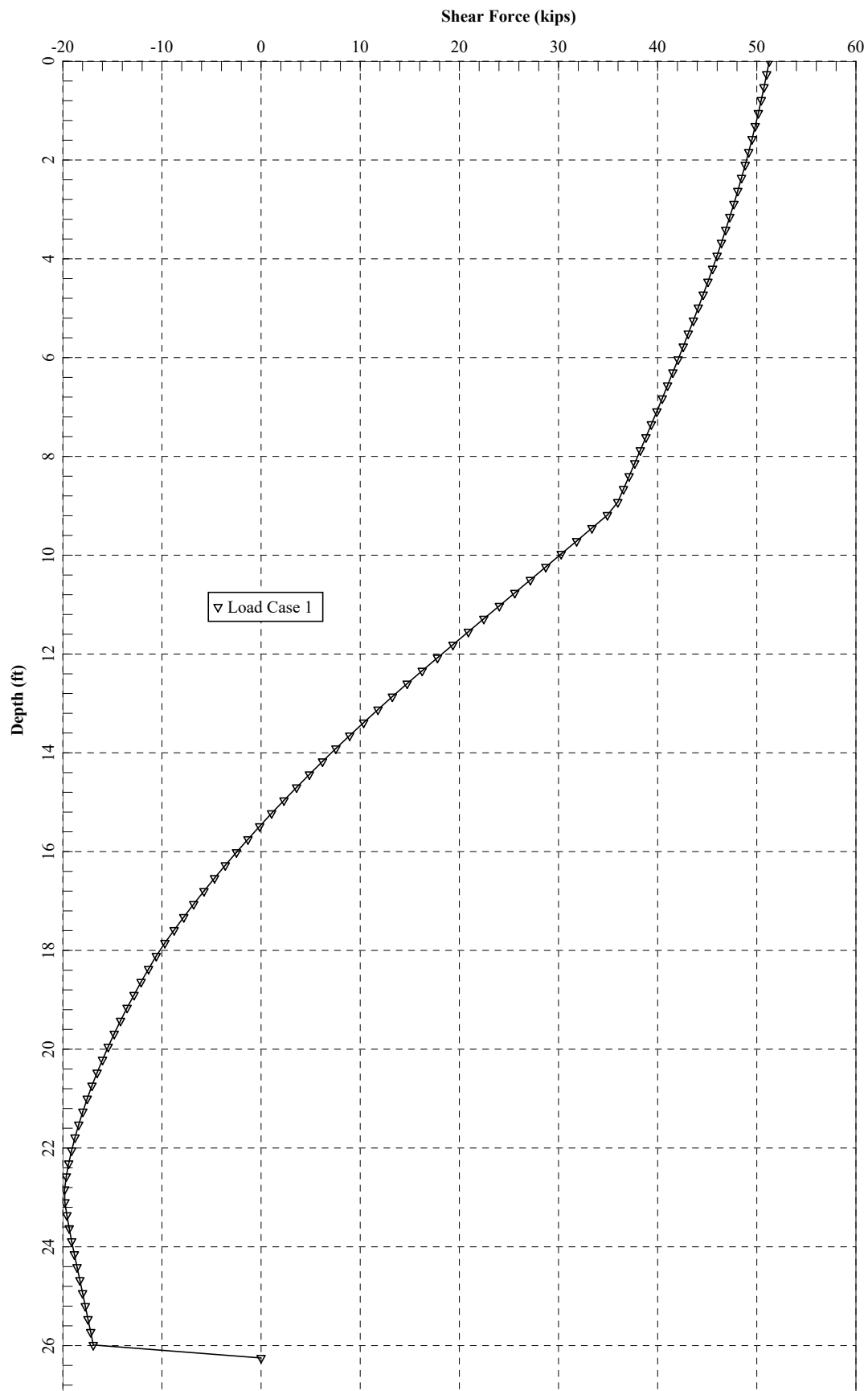
Maximum pile-head deflection = 1.5009240175 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.

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Abutment 1 - 34 foot Pile

Abutment 1_14x89.lp9o

LPIle for Windows, Version 2016-09.010

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method
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Files Used for Analysis

Path to file locations:

\\09 Jobs\0025900s\09.0025988.00 - Gorham - Bridge over Little River\09.0025988.01 - Final Design\Work\LPIle\Abutment
1\HP 14x89\34 Foot Length\

Name of input data file:

Abutment 1_14x89.lp9d

Name of output report file:

Abutment 1_14x89.lp9o

Name of plot output file:

Abutment 1_14x89.lp9p

Page 1

Abutment 1_14x89.lp9o

Name of runtime message file:

Abutment 1_14x89.lp9r

Date and Time of Analysis

Date: August 19, 2019

Time: 13:48:59

Problem Title

Project Name: Little River Bridge #3112

Job Number: 09.0025988.00

Client: Hoyle, Tanner & Associates

Engineer: N. Williams

Description: Abutment 1

Program Options and Settings

Computational Options:

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Abutment 1_14x89.lp9o

- Use unfactored loads in computations (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
- No distributed lateral loads are entered
- Loading by lateral soil movements acting on pile not selected
- Input of shear resistance at the pile tip not selected
- 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 = 34.000 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.
 Page 3

Abutment 1_14x89.lp9o

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	34.000	14.7000

Input Structural Properties for Pile Sections:

Pile Section No. 1:

Section 1 is an elastic pile	
Cross-sectional Shape	= Strong H-Pile
Length of section	= 34.000000 ft
Flange Width	= 14.700000 in
Section Depth	= 13.800000 in
Flange Thickness	= 0.615000 in
Web Thickness	= 0.615000 in
Section Area	= 25.811550 sq. in
Moment of Inertia	= 888.177980 in^4
Elastic Modulus	= 29000000. psi

----- 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 5 layers

Layer 1 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	0.0000	ft
Distance from top of pile to bottom of layer	=	5.000000	ft
Effective unit weight at top of layer	=	115.000000	pcf
Effective unit weight at bottom of layer	=	115.000000	pcf
Undrained cohesion at top of layer	=	400.000000	psf
Undrained cohesion at bottom of layer	=	400.000000	psf
Epsilon-50 at top of layer	=	0.020000	
Epsilon-50 at bottom of layer	=	0.020000	

Layer 2 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	5.000000	ft
Distance from top of pile to bottom of layer	=	9.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	400.000000	psf
Undrained cohesion at bottom of layer	=	400.000000	psf
Epsilon-50 at top of layer	=	0.020000	
Epsilon-50 at bottom of layer	=	0.020000	

Layer 3 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	9.000000	ft
Distance from top of pile to bottom of layer	=	18.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	900.000000	psf
Undrained cohesion at bottom of layer	=	900.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

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Layer 4 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	18.000000	ft
Distance from top of pile to bottom of layer	=	27.000000	ft
Effective unit weight at top of layer	=	43.000000	pcf
Effective unit weight at bottom of layer	=	43.000000	pcf
Undrained cohesion at top of layer	=	800.000000	psf
Undrained cohesion at bottom of layer	=	800.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

Layer 5 is soft clay, p-y criteria by Matlock, 1970

Distance from top of pile to top of layer	=	27.000000	ft
Distance from top of pile to bottom of layer	=	35.000000	ft
Effective unit weight at top of layer	=	53.000000	pcf
Effective unit weight at bottom of layer	=	53.000000	pcf
Undrained cohesion at top of layer	=	550.000000	psf
Undrained cohesion at bottom of layer	=	550.000000	psf
Epsilon-50 at top of layer	=	0.008000	
Epsilon-50 at bottom of layer	=	0.008000	

(Depth of the lowest soil layer extends 1.000 ft below the pile tip)

Summary of Input Soil Properties

Layer Layer Num.	Soil Type Name (p-y Curve Type)	Layer Depth ft	Effective Unit Wt. pcf	Undrained Cohesion psf	E50 or krm
1	Soft	0.00	115.0000	400.0000	0.02000
	Clay	5.0000	115.0000	400.0000	0.02000
2	Soft	5.0000	53.0000	400.0000	0.02000

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3	Clay	9.0000	53.0000	400.0000	0.02000
	Soft	9.0000	53.0000	900.0000	0.00800
4	Clay	18.0000	53.0000	900.0000	0.00800
	Soft	18.0000	43.0000	800.0000	0.00800
5	Clay	27.0000	43.0000	800.0000	0.00800
	Soft	27.0000	53.0000	550.0000	0.00800
	Clay	35.0000	53.0000	550.0000	0.00800

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
1	5	y = 1.500000 in	S = 0.0000 in/in	330000.	N.A.

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
Page 7

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Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:

Moment-curvature properties were derived from elastic section properties

Layering Correction Equivalent Depths of Soil & Rock Layers

Layer No.	Top of Layer Below Pile Head ft	Equivalent Top Depth Below Grnd Surf ft	Same Layer Type As Layer Above	Layer is Rock or is Below Rock Layer	F0 Integral for Layer lbs	F1 Integral for Layer lbs
1	0.00	0.00	N.A.	No	0.00	11611.
2	5.0000	5.0000	Yes	No	11611.	14844.
3	9.0000	9.0000	Yes	No	26455.	86736.
4	18.0000	18.0000	Yes	No	113191.	79355.
5	27.0000	27.0000	Yes	No	192546.	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

Abutment 1_14x89.lp9o

Pile-head conditions are Displacement and Pile-head Rotation (Loading Type 5)
 Displacement of pile head = 1.500000 inches
 Rotation of pile head = 0.000E+00 radians
 Axial load on pile head = 330000.0 lbs

Depth X feet	Deflect. y inches	Bending Moment in-lbs	Shear Force lbs	Slope S radians	Total Stress psi*	Bending Stiffness in-lb^2	Soil Res. p lb/inch	Soil Spr. Es*h lb/inch	Distrib. Lat. Load lb/inch
0.00	1.5015	-4790507.	51072.	0.00	52428.	2.58E+10	-77.7183	105.5879	0.00
0.3400	1.5000	-4582340.	50725.	-7.42E-04	50706.	2.58E+10	-83.8169	227.9821	0.00
0.6800	1.4955	-4374591.	50371.	-0.00145	48986.	2.58E+10	-89.8521	245.1346	0.00
1.0200	1.4882	-4167406.	49992.	-0.00213	47272.	2.58E+10	-95.8141	262.6889	0.00
1.3600	1.4781	-3960926.	49589.	-0.00277	45563.	2.58E+10	-101.6938	280.7008	0.00
1.7000	1.4655	-3755294.	49162.	-0.00338	43861.	2.58E+10	-107.4823	299.2273	0.00
2.0400	1.4505	-3550651.	48712.	-0.00396	42168.	2.58E+10	-113.1711	318.3267	0.00
2.3800	1.4332	-3347134.	48239.	-0.00451	40484.	2.58E+10	-118.7522	338.0596	0.00
2.7200	1.4137	-3144880.	47743.	-0.00502	38810.	2.58E+10	-124.2174	358.4892	0.00
3.0600	1.3922	-2944024.	47226.	-0.00550	37148.	2.58E+10	-129.5593	379.6818	0.00
3.4000	1.3688	-2744696.	46686.	-0.00596	35498.	2.58E+10	-134.7703	401.7077	0.00
3.7400	1.3436	-2547026.	46126.	-0.00637	33863.	2.58E+10	-139.8433	424.6414	0.00
4.0800	1.3168	-2351141.	45546.	-0.00676	32242.	2.58E+10	-144.7713	448.5625	0.00
4.4200	1.2885	-2157164.	44945.	-0.00712	30636.	2.58E+10	-149.5475	473.5564	0.00
4.7600	1.2587	-1965217.	44326.	-0.00745	29048.	2.58E+10	-154.1653	499.7149	0.00
5.1000	1.2277	-1775417.	43688.	-0.00774	27477.	2.58E+10	-158.2428	525.8893	0.00
5.4400	1.1955	-1587872.	43037.	-0.00801	25925.	2.58E+10	-161.2626	550.3419	0.00
5.7800	1.1623	-1402673.	42373.	-0.00825	24393.	2.58E+10	-164.1290	576.1167	0.00
6.1200	1.1283	-1219908.	41698.	-0.00845	22880.	2.58E+10	-166.8377	603.3213	0.00
6.4600	1.0934	-1039659.	41012.	-0.00863	21389.	2.58E+10	-169.3846	632.0734	0.00
6.8000	1.0578	-862008.	40316.	-0.00878	19918.	2.58E+10	-171.7657	662.5019	0.00
7.1400	1.0217	-687033.	39610.	-0.00891	18470.	2.58E+10	-173.9771	694.7483	0.00
7.4800	0.9851	-514807.	38896.	-0.00900	17045.	2.58E+10	-176.0150	728.9678	0.00
7.8200	0.9483	-345401.	38175.	-0.00907	15643.	2.58E+10	-177.8758	765.3316	0.00
8.1600	0.9111	-178883.	37445.	-0.00911	14265.	2.58E+10	-179.5562	804.0278	0.00
8.5000	0.8739	-15316.	36710.	-0.00913	12912.	2.58E+10	-181.0528	845.2643	0.00
8.8400	0.8367	145241.	35968.	-0.00912	13987.	2.58E+10	-182.3626	889.2702	0.00
9.1800	0.7995	302731.	34598.	-0.00908	15290.	2.58E+10	-489.3686	2497.	0.00
9.5200	0.7626	452011.	32597.	-0.00902	16526.	2.58E+10	-491.7344	2631.	0.00
9.8600	0.7259	593008.	30587.	-0.00894	17692.	2.58E+10	-493.5857	2774.	0.00
10.2000	0.6897	725663.	28570.	-0.00883	18790.	2.58E+10	-494.9168	2928.	0.00

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10.5400	0.6539	849924.	26549.	-0.00871	19818.	2.58E+10	-495.7219	3093.	0.00
10.8800	0.6186	965752.	24526.	-0.00856	20777.	2.58E+10	-495.9954	3271.	0.00
11.2200	0.5840	1073117.	22503.	-0.00840	21665.	2.58E+10	-495.7318	3463.	0.00
11.5600	0.5500	1172002.	20482.	-0.00822	22484.	2.58E+10	-494.9257	3671.	0.00
11.9000	0.5169	1262397.	18465.	-0.00803	23232.	2.58E+10	-493.5716	3896.	0.00
12.2400	0.4845	1344307.	16462.	-0.00783	23910.	2.58E+10	-488.3452	4112.	0.00
12.5800	0.4530	1417802.	14492.	-0.00761	24518.	2.58E+10	-477.5262	4301.	0.00
12.9200	0.4224	1483045.	12566.	-0.00738	25058.	2.58E+10	-466.5309	4506.	0.00
13.2600	0.3928	1540205.	10685.	-0.00714	25531.	2.58E+10	-455.3620	4730.	0.00
13.6000	0.3642	1589457.	8851.	-0.00689	25938.	2.58E+10	-444.0215	4974.	0.00
13.9400	0.3366	1630979.	7062.	-0.00663	26282.	2.58E+10	-432.5105	5243.	0.00
14.2800	0.3101	1664953.	5322.	-0.00637	26563.	2.58E+10	-420.8292	5538.	0.00
14.6200	0.2846	1691566.	3629.	-0.00611	26783.	2.58E+10	-408.9769	5863.	0.00
14.9600	0.2602	1711011.	1985.	-0.00584	26944.	2.58E+10	-396.9513	6224.	0.00
15.3000	0.2369	1723483.	390.1004	-0.00557	27047.	2.58E+10	-384.7487	6625.	0.00
15.6400	0.2148	1729183.	-1154.	-0.00529	27095.	2.58E+10	-372.3635	7073.	0.00
15.9800	0.1938	1728316.	-2648.	-0.00502	27087.	2.58E+10	-359.7876	7576.	0.00
16.3200	0.1738	1721090.	-4090.	-0.00475	27028.	2.58E+10	-347.0100	8144.	0.00
16.6600	0.1550	1707722.	-5479.	-0.00447	26917.	2.58E+10	-334.0157	8790.	0.00
17.0000	0.1373	1688429.	-6815.	-0.00421	26757.	2.58E+10	-320.7848	9530.	0.00
17.3400	0.1207	1663435.	-8096.	-0.00394	26551.	2.58E+10	-307.2906	10386.	0.00
17.6800	0.1052	1632972.	-9322.	-0.00368	26298.	2.58E+10	-293.4975	11385.	0.00
18.0200	0.09070	1597275.	-10427.	-0.00342	26003.	2.58E+10	-248.3177	11170.	0.00
18.3600	0.07725	1557104.	-11414.	-0.00317	25671.	2.58E+10	-235.3816	12431.	0.00
18.7000	0.06481	1512682.	-12347.	-0.00293	25303.	2.58E+10	-221.9970	13975.	0.00
19.0400	0.05334	1464242.	-13224.	-0.00269	24902.	2.58E+10	-208.0473	15912.	0.00
19.3800	0.04283	1412027.	-14043.	-0.00247	24470.	2.58E+10	-193.3591	18421.	0.00
19.7200	0.03322	1356292.	-14800.	-0.00225	24009.	2.58E+10	-177.6603	21820.	0.00
20.0600	0.02449	1297310.	-15490.	-0.00204	23521.	2.58E+10	-160.4902	26737.	0.00
20.4000	0.01660	1235380.	-16105.	-0.00184	23008.	2.58E+10	-140.9719	34649.	0.00
20.7400	0.00951	1170840.	-16631.	-0.00165	22474.	2.58E+10	-117.0610	50238.	0.00
21.0800	0.00317	1104101.	-17036.	-0.00147	21922.	2.58E+10	-81.1516	104416.	0.00
21.4200	-0.00245	1035776.	-17049.	-0.00130	21356.	2.58E+10	74.5781	124120.	0.00
21.7600	-0.00740	968472.	-16677.	-0.00114	20799.	2.58E+10	107.7508	59372.	0.00
22.1000	-0.01173	902754.	-16201.	-9.89E-04	20256.	2.58E+10	125.6038	43682.	0.00
22.4400	-0.01548	838935.	-15664.	-8.51E-04	19727.	2.58E+10	137.7466	36316.	0.00
22.7800	-0.01868	777230.	-15084.	-7.23E-04	19217.	2.58E+10	146.6540	32037.	0.00
23.1200	-0.02138	717801.	-14471.	-6.05E-04	18725.	2.58E+10	153.4019	29279.	0.00
23.4600	-0.02361	660772.	-13835.	-4.96E-04	18253.	2.58E+10	158.5717	27401.	0.00
23.8000	-0.02542	606242.	-13180.	-3.95E-04	17802.	2.58E+10	162.5200	26085.	0.00
24.1400	-0.02684	554288.	-12511.	-3.03E-04	17372.	2.58E+10	165.4841	25159.	0.00

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Abutment 1_14x89.lp9o									
24.4800	-0.02789	504970.	-11831.	-2.19E-04	16964.	2.58E+10	167.6313	24519.	0.00
24.8200	-0.02863	458335.	-11144.	-1.43E-04	16578.	2.58E+10	169.0849	24099.	0.00
25.1600	-0.02906	414417.	-10453.	-7.40E-05	16214.	2.58E+10	169.9387	23858.	0.00
25.5000	-0.02923	373240.	-9759.	-1.16E-05	15874.	2.58E+10	170.2657	23766.	0.00
25.8400	-0.02916	334817.	-9064.	4.45E-05	15556.	2.58E+10	170.1237	23806.	0.00
26.1800	-0.02887	299155.	-8371.	9.47E-05	15261.	2.58E+10	169.5591	23966.	0.00
26.5200	-0.02838	266251.	-7682.	1.39E-04	14988.	2.58E+10	168.6088	24237.	0.00
26.8600	-0.02773	236098.	-6996.	1.79E-04	14739.	2.58E+10	167.3025	24617.	0.00
27.2000	-0.02692	208679.	-6423.	2.15E-04	14512.	2.58E+10	113.8934	17261.	0.00
27.5400	-0.02598	183111.	-5961.	2.46E-04	14300.	2.58E+10	112.5492	17677.	0.00
27.8800	-0.02492	159378.	-5505.	2.73E-04	14104.	2.58E+10	110.9968	18175.	0.00
28.2200	-0.02375	137459.	-5055.	2.96E-04	13922.	2.58E+10	109.2421	18765.	0.00
28.5600	-0.02250	117329.	-4614.	3.16E-04	13756.	2.58E+10	107.2882	19455.	0.00
28.9000	-0.02117	98960.	-4180.	3.33E-04	13604.	2.58E+10	105.1352	20261.	0.00
29.2400	-0.01978	82320.	-3756.	3.48E-04	13466.	2.58E+10	102.7795	21202.	0.00
29.5800	-0.01833	67373.	-3342.	3.60E-04	13343.	2.58E+10	100.2139	22303.	0.00
29.9200	-0.01684	54080.	-2939.	3.69E-04	13233.	2.58E+10	97.4259	23600.	0.00
30.2600	-0.01532	42397.	-2548.	3.77E-04	13136.	2.58E+10	94.3968	25141.	0.00
30.6000	-0.01377	32277.	-2169.	3.83E-04	13052.	2.58E+10	91.0994	26997.	0.00
30.9400	-0.01219	23666.	-1805.	3.87E-04	12981.	2.58E+10	87.4944	29273.	0.00
31.2800	-0.01061	16506.	-1456.	3.90E-04	12922.	2.58E+10	83.5251	32128.	0.00
31.6200	-0.00901	10734.	-1124.	3.93E-04	12874.	2.58E+10	79.1065	35828.	0.00
31.9600	-0.00740	6276.	-811.6090	3.94E-04	12837.	2.58E+10	74.1069	40842.	0.00
32.3000	-0.00579	3050.	-521.0849	3.95E-04	12810.	2.58E+10	68.3069	48105.	0.00
32.6400	-0.00418	961.0105	-256.6906	3.95E-04	12793.	2.58E+10	61.2982	59804.	0.00
32.9800	-0.00257	-108.0888	-25.2360	3.95E-04	12786.	2.58E+10	52.1600	82811.	0.00
33.3200	-9.58E-04	-308.8885	158.0413	3.95E-04	12788.	2.58E+10	37.6818	160515.	0.00
33.6600	6.54E-04	117.6437	168.2270	3.95E-04	12786.	2.58E+10	-32.6888	203916.	0.00
34.0000	0.00227	0.00	0.00	3.95E-04	12785.	2.58E+10	-49.7754	44812.	0.00

* The above values of total stress are combined axial and bending stresses.

Output Summary for Load Case No. 1:

Pile-head deflection = 1.50154801 inches
 Computed slope at pile head = 0.000000 radians
 Maximum bending moment = -4790507. inch-lbs
 Maximum shear force = 51072. lbs
 Depth of maximum bending moment = 0.000000 feet below pile head
 Depth of maximum shear force = 0.000000 feet below pile head

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Abutment 1_14x89.lp9o
 Number of iterations = 23
 Number of zero deflection points = 2

Summary of Pile-head Responses for Conventional Analyses

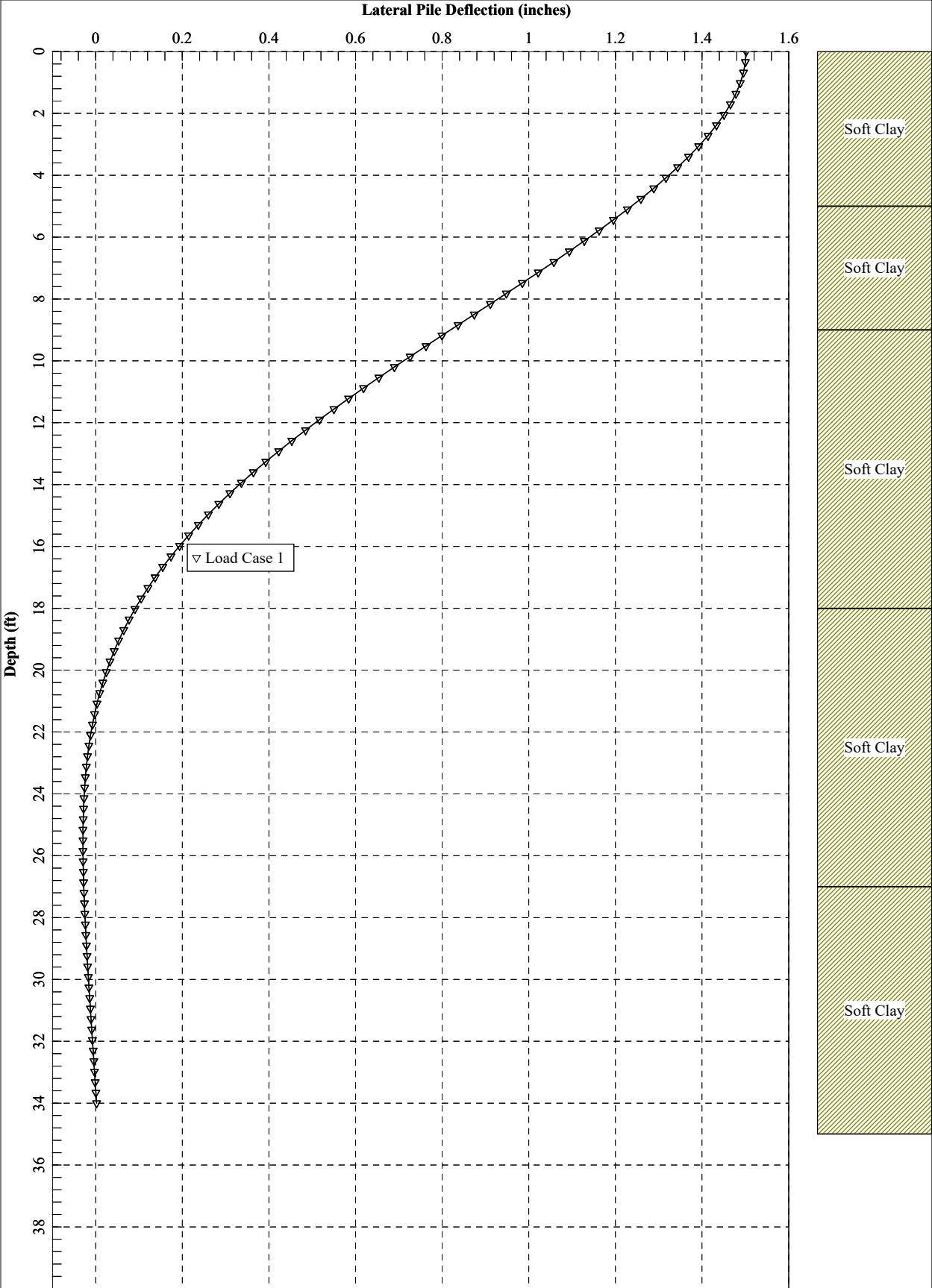
Definitions of Pile-head Loading Conditions:

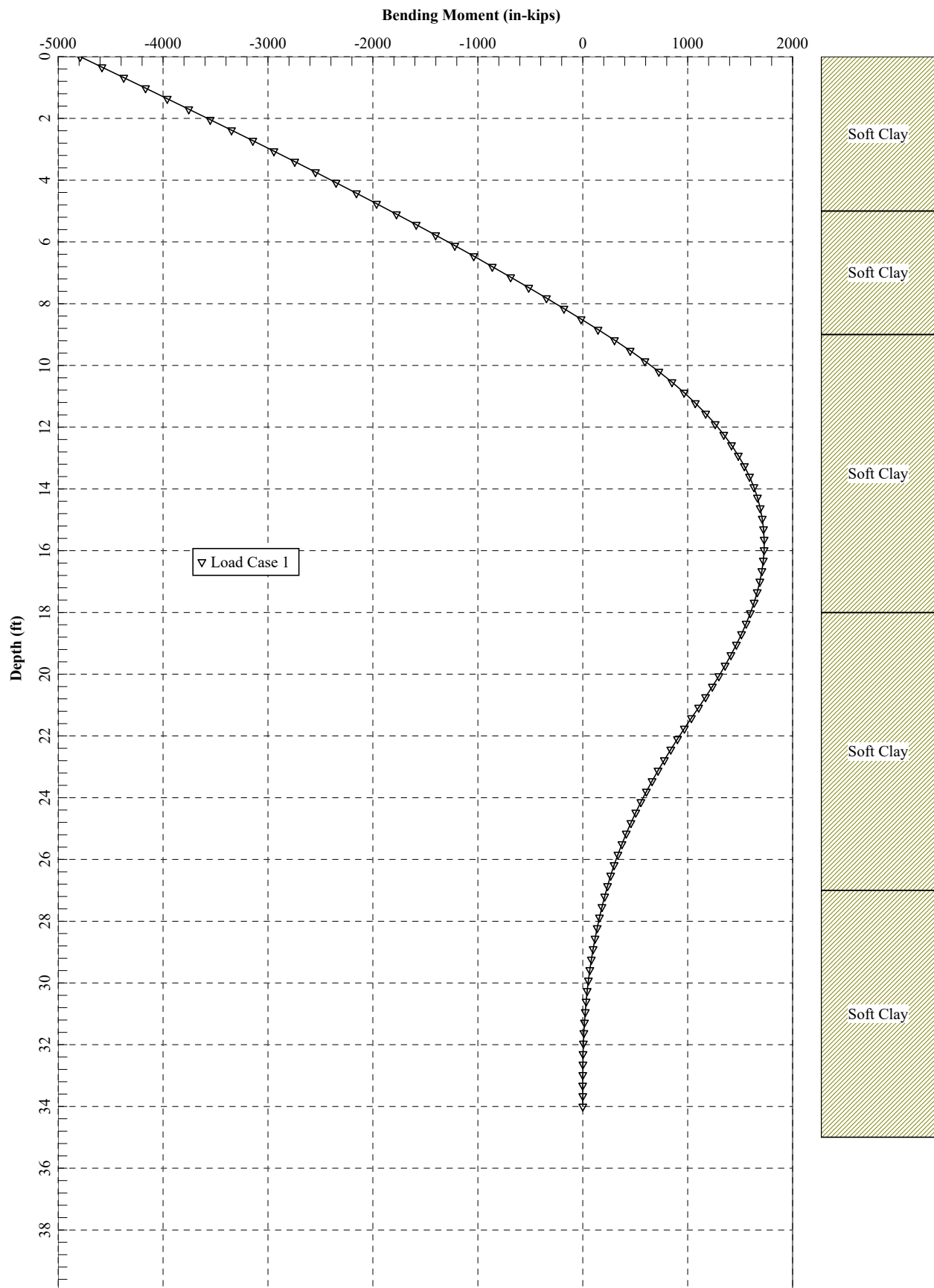
Load Type 1: Load 1 = Shear, V, lbs, and Load 2 = Moment, M, in-lbs
 Load Type 2: Load 1 = Shear, V, lbs, and Load 2 = Slope, S, radians
 Load Type 3: Load 1 = Shear, V, lbs, and Load 2 = Rot. Stiffness, R, in-lbs/rad.
 Load Type 4: Load 1 = Top Deflection, y, inches, and Load 2 = Moment, M, in-lbs
 Load Type 5: Load 1 = Top Deflection, y, inches, and Load 2 = Slope, S, radians

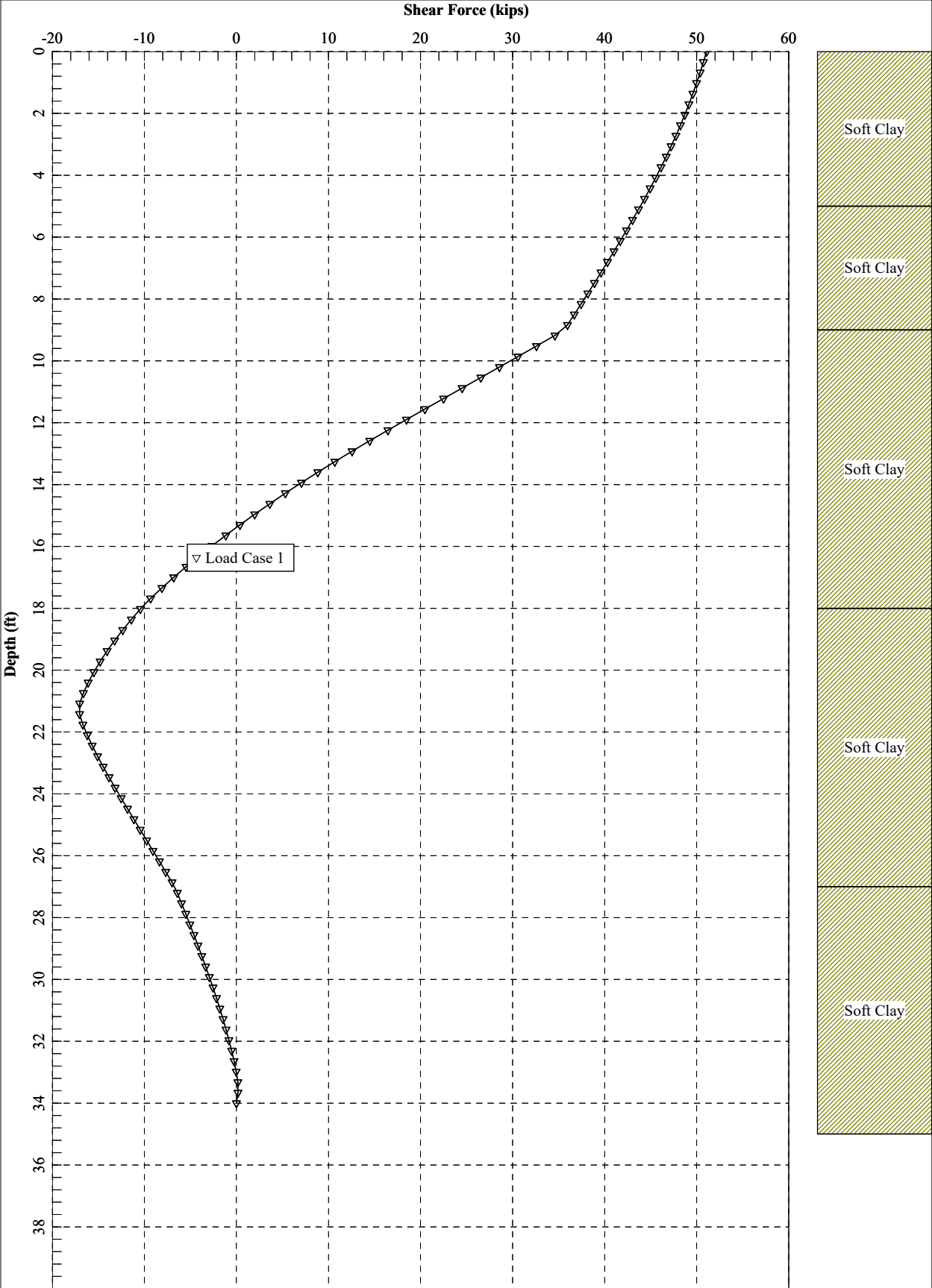
Load Case No.	Load Type	Pile-head Load 1	Load Type 2	Pile-head Load 2	Axial Loading lbs	Pile-head Deflection inches	Pile-head Rotation radians	Max Shear in Pile lbs	Max Moment in Pile in-lbs
1	y, in	1.5000	S, rad	0.00	330000.	1.5015	0.00	51072.	-4790507.

Maximum pile-head deflection = 1.5015480100 inches
 Maximum pile-head rotation = 0.0000000000 radians = 0.000000 deg.

The analysis ended normally.







Lateral Earth Pressure



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

Engineers and
 Scientists

JOB: 09.0025988.01 Little River Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 1
 CALCULATED BY N. Williams 9/12/19
 CHECKED BY A. Blaisdell 9/12/19

Subject: Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)

Input Parameters:

$\beta := 0\text{deg}$ Angle of backfill to the horizontal

$\theta := 90\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction (*Granular borrow, Soil Type 4, BDG Table 3-3*)

$\delta_f := 19.5\text{deg}$ Average value, precast concrete against clean sand/silty sand-gravel mixture (*AASHTO LRFD Table 3.11.5.3-1*)

Passive Earth Pressure on Integral Backwall:

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (H_b) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered.

$y := 1.5\text{in}$

$H_b := 13\text{ft}$

$\frac{y}{H_b} = 0.0096$ Ratio of lateral movement to abutment height is greater than 0.005, use full Coulomb Pressure

Earth Pressure Coefficients:

Coulomb Passive Earth Pressure Coefficient

$$K_{pc} := \frac{(\sin(\theta - \phi))^2}{\left[(\sin(\theta))^2 \cdot \sin(\theta + \delta_f) \cdot \left[1 - \sqrt{\frac{(\sin(\phi + \delta_f) \cdot \sin(\phi + \beta))}{(\sin(\theta + \delta_f) \cdot \sin(\theta + \beta))}} \right]^2 \right]}$$

$$K_{pc} = 6.73$$



10/01/2019

GEOTECHNICAL DESIGN REPORT
LITTLE RIVER BRIDGE NO. 3112 - GORHAM
09.0025988.01

APPENDIX G – ROCK CORE PHOTOGRAPHS



MaineDOT Little River Bridge #3112

Route 237 Over Little River

Gorham, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-GLR-101	R1	38 - 43	60	100%	51	85%	SCHIST	1
BB-GLR-101	R2	43 - 48	59	98%	50	83%	SCHIST	2
BB-GLR-102	R1	60 - 62.5	17	57%	5.5	18%	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.



MaineDOT Little River Bridge #3112

Route 237 Over Little River

Gorham, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-GLR-201	R1	45.1 - 50.1	56	93%	56	93%	SCHIST	1
BB-GLR-201	R2	50.1 - 55.1	60	100%	60	100%	SCHIST	2



- 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.