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Attached is one (1) copy of Soils Report 2019-15, "GEOTECHNICAL DESIGN REPORT For the Replacement of: TAYLOR BROOK BRIDGE, HOTEL ROAD OVER TAYLOR BROOK, AUBURN, MAINE" dated: March 26, 2019.

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Maine Department of Transportation
Bridge Program
Geotechnical Section
Augusta, Maine

Geotechnical Design Report

For the Replacement of:

Taylor Brook Bridge
Hotel Road Over Taylor Brook
Auburn, Maine

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Soils Report 2019-15
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March 26, 2019

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Taylor Brook Bridge which carries Hotel Road over Taylor Brook in Auburn, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical recommendations, and geotechnical design parameters for the design of the new substructures.

The existing bridge was built in 1982 and consists of two steel pipe arch culverts with spans of approximately 11 feet and spaced 4 feet apart. The structure is skewed approximately 15 degrees with the roadway. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the existing culverts are rated 4 for "poor" condition. There is extensive pitting of the pipes, rusting below the flowline, and section loss at the inlet. The Sufficiency Rating of the bridge is 72.1. In addition to addressing the structural deficiencies of the existing culverts, a secondary goal of the project is to improve hydraulic conditions.

The recommended bridge replacement alternative is a 71-foot single-span integral abutment bridge, with a NEXT F beam superstructure with integral abutments on driven H-piles. The proposed bridge will be built on alignment. During construction of the new bridge, traffic will be maintained by an off-site detour.

2.0 GEOLOGIC SETTING

The existing structure carries Hotel Road over Taylor Brook as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology of the Minot Quadrangle, Open File Map 02-231 (2002), indicates the surficial soils in the vicinity of the bridge project consist of artificial fill and stream alluvium, with contacts to glaciomarine deposits (Presumpscot Formation). Stream alluvium is generally comprised of sand, silt, gravel and organics and is found in flood plains along present streams. The Presumpscot Formation generally consists of clay and silt that washed out of the Lake Wisconsinan glacier and accumulated on the ocean floor when the relative sea level was higher than at present. This soil unit typically overlies an irregular surface of glacial till and may include areas of till exposed at the ground surface. Glacial till is a heterogeneous mixture of sand, silt, clay and stones.

The MGS Bedrock Map of Maine (1985), indicates that the bedrock at the project site is part of the Sangerville Formation consisting of Sandstone and Schist. Bedrock cores retrieved at the site are identified as Schist.

3.0 PREVIOUS SUBSURFACE INVESTIGATIONS

A subsurface investigation was conducted in 1980 for the proposed widening of the pre-existing bridge structure and raise in the roadway grade. The pre-existing structure consisted of a single-span concrete slab bridge on mass concrete abutments. The investigation consisted of two test borings. Details of this subsurface investigation are provided in MaineDOT Soils Report 80-105 which is provided in Appendix E.

4.0 SUBSURFACE INVESTIGATION

Four test borings were drilled at the site: BB-ATB-101, BB-ATB-102, BB-ATB-102A, and BB-ATB-103. Borings BB-ATB-101 was drilled at proposed Abutment No. 1 and BB-ATB-103 was drilled at proposed Abutment No. 2. Borings BB-ATB-102 and BB-ATB-102A were drilled behind the existing southern pipe arch to investigate a narrower frame structure. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled on May 8 and 9, 2018 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained in 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in June 2018. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.928 for all borings. The hammer efficiency factor (0.928) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core calculated. A MaineDOT geotechnical engineer logged the subsurface conditions encountered. The geotechnical engineer selected the boring location and drilling methods, designated type and depth of sampling techniques, reviewed boring log and identified field testing requirements. The borings were surveyed in the field at the completion of the drilling program.

5.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Soil laboratory testing consisted of: thirteen standard grain size analyses with natural water content, one grain size analysis with hydrometer and natural water content, and one Atterberg limits test. Bedrock laboratory testing consisted of three unconfined compressive tests with elastic moduli. The results of soil and rock tests are included as Appendix B – Laboratory Test Results.

Moisture content information and other soil and rock test results are also shown on the boring logs provided in Appendix A – Boring Logs.

6.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of Fill, Stream Alluvium, Glaciomarine Deposits, and Glacial Till underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered:

6.1 Fill

A layer of fill was encountered in all the test borings. The thickness of the fill unit encountered ranged from approximately 10 to 16 feet thick and generally consisted of:

- Brown, damp to moist, sand, little to some silt, trace to little gravel, trace brick fragments;
- Brown, moist, gravelly sand, little silt.

SPT N-values in the fill unit ranged from 3 to 46 blows per foot (bpf) indicating that the fill is very loose to dense in consistency. Grain size analyses conducted on samples from the fill unit resulted in the soils being classified as A-1-b and A-2-4 under the AASHTO Classification System and as SM and SW-SM under the Unified Soil Classification System (USCS). The moisture contents of the samples tested ranged from approximately 8 to 13 percent.

6.2 Stream Alluvium

Stream alluvium was encountered beneath the fill unit in two borings. The thickness of the alluvial deposit encountered was approximately 5 to 10 feet thick and generally consisted of:

- Brown to dark brown, wet, fine to medium sand, trace to some silt, trace gravel, trace coarse sand;

SPT N-values in the alluvial deposit ranged from 2 to 39 bpf indicating that the alluvium is very loose to dense in consistency. Grain size analyses conducted on samples from the alluvium deposit classified the soils as A-2-4 and A-3 under the AASHTO Classification System and as SP and SP-SM under the USCS. The moisture contents of the samples tested ranged from approximately 18 to 34 percent.

6.3 Glaciomarine Deposits

Glaciomarine silty clay was encountered beneath the stream alluvium in boring BB-ATB-103. The thickness of the glaciomarine deposit encountered was approximately 5 feet and generally consisted of grey, wet, silty clay, trace sand.

One SPT N-value in the glaciomarine deposit was 3 bpf indicating that the deposit is soft in consistency. One grain size analysis with hydrometer was conducted on the sample from the deposit indicating the soil is classified as A-6 under the AASHTO Classification System and as CL under the USCS. The moisture content of the sample tested was 43.5 percent.

One Atterberg limits test was conducted on a sample of the glaciomarine deposit, and is summarized in Table 1.

Boring No. and Sample No.	Soil Description	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ATB-103, 4D	Silty CLAY	43.5	36	23	13	1.6

Table 1 - Summary of Atterberg Limits Test Results

The plasticity index of the sample indicates that the soil has medium plasticity (Burmister, 1949). The natural water content was 43.5 percent and the liquid limit was 36. The liquidity index was 1.6. Soils with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 also have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a liquid. Liquidity index values greater than or equal to 1 are also indicative of soils that are unconsolidated and are commonly referred to as "quick."

6.4 Glacial Till

Glacial till was encountered underlying the stream alluvium in BB-ATB-101, below the glaciomarine clay in BB-ATB-103, and underlying the fill soils in BB-ATB-102A. The thickness of the deposit encountered was approximately 6 to 12 feet thick and generally consisted of:

- Grey, moist to wet, sand, little to some gravel, little to some silt;
- Grey, wet, gravelly sand, some silt.

SPT N-values in the glacial till ranged from 23 to 90 bpf indicating that the glacial till is medium dense to very dense in consistency. Grain size analyses with water content tests conducted on samples from the glacial till deposit indicate the soils are classified as A-1-b and A-2-4 under the AASHTO Classification System and SM under the USCS. The moisture contents of the samples tested ranged from approximately 10 to 11 percent.

6.5 Bedrock

Bedrock was encountered and cored in three of the four borings. Table 2 summarizes approximate depth to bedrock, corresponding approximate top of the bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD R1, R2 (%)
BB-ATB-101	171+50	6.8 L	23.5	227.9	95, 83
BB-ATB-102A	171+89	11.6 R	28.7	221.6	88, 62
BB-ATB-103	172+45.1	11.9 R	31.3	218.9	100, 45

Table 2 – Summary of Approximate Bedrock Depths and Elevations

The bedrock recovered from the boring is generally identified as black and grey, fine grained, Schist, very hard, fresh, joints at horizontal to moderately dipping and vertical, close to moderately close, open, little infilling. The RQD of the bedrock ranged from 45 to 100 percent correlating to a rock mass quality of poor to excellent.

Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. Photographs of rock cores are presented in Appendix B – Rock Core Photographs.

6.6 Groundwater

Groundwater was observed at 8.2 ft bgs in one boring during the subsurface investigations. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

7.0 FOUNDATION ALTERNATIVES

The October 2018 Preliminary Design Report considered buried structures and at-grade structures for bridge replacement options. The buried structure options included twin box culverts, steel pipe arches, and three-sided concrete frames (arches). At-grade structure alternatives included box beam, steel girder, and NEXT beam superstructures on pile-supported integral abutments. Spun steel piles, driven H-piles, and micropiles socketed into bedrock, were all considered for foundation support. The recommended alternative is a NEXT F beam superstructure founded on pile-supported integral abutments. During final design it was determined foundation support would consist of driven H-piles fitted with special driving tips to improve pile penetration into bedrock.

8.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

The following sections provide geotechnical design considerations and recommendations for H-pile supported integral bridge abutments, which are the proposed foundation type for the Taylor Brook Bridge replacement project.

8.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of driven H-piles. The piles shall be end bearing and driven to the required nominal resistance on or within bedrock. Piles may be HP 12x53, 14x73, 14x89, or 14x117 depending on the factored design axial loads and ability to resist lateral loads. H-piles shall be 50 ksi, Grade A572 steel.

Abutment No. 1 piles should be fitted with Rock Injector HP-80500 Pile Points manufactured by Associated Pile and Fitting (APF), or equivalent, to improve penetration and improve friction at the pile tip to support pinned or fixed pile tip assumptions. Abutment No. 2 piles may also require Rock Injector Pile Points to satisfy design requirements; otherwise the Abutment No. 2 pile may be fitted with standard pile points conforming to MaineDOT Standard Specification 711.10.

Pile lengths at the proposed abutments may be estimated based on Table 3.

Location	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	241.0	227	14
Abutment No. 2	240.2	218	22

Table 3 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths in Table 3 do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment.

8.1.1 Strength Limit State Design

The design of pile foundations bearing on bedrock at the strength limit state shall consider:

- compressive axial geotechnical resistance of individual piles bearing on bedrock,
- drivability resistance of individual piles driven to bedrock,
- structural resistance of individual piles in axial compression, and
- structural resistance of individual piles in combined axial loading and flexure.

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Per AASHTO LRFD Bridge Design Specifications 8th Edition 2017 (LRFD) Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.50 (severe driving conditions) shall be applied to the structural compressive resistance of the pile. Since the H-piles will be subjected to lateral loading, the piles shall also be checked for resistance against combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). H-piles shall also be analyzed for fixity using LPile® v2016 (LPile) software, or similar.

<u>Structural Resistance.</u> The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the structural axial resistance of four H-pile sections were calculated for the lower braced pile segment. The resistances shown in Table 3 are for the lower braced pile segment, using a resistance factor, ϕ_c = 0.50 for severe driving conditions and an assumed effective length factor (K). Supporting calculations are provided in Appendix C – Calculations.

Factored structural resistances should be calculated for upper and lower unbraced segments based on LPile results using a resistance factor ϕ_c = 0.70, for combined axial loading and bending. This is the responsibility of the structural engineer.

Geotechnical Resistance. The static geotechnical resistance of piles driven to hard rock was estimated using the Intact Rock Method (IRM).[1] The nominal axial geotechnical resistance in the strength limit state was also calculated using the guidance in LRFD Article 10.7.3.2.1 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c, of 0.50, for severe driving conditions applied. The resulting limiting factored geotechnical compressive resistances for piles driven to rock are provided in Table 4.

Drivability Analyses. Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances were calculated using the resistance factor, φ_{dyn}, of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of four H-pile sections for the strength limit states are provided in Table 4 below. Supporting calculations are provided in Appendix D – Calculations.

[1] MaineDOT Transportation Research Division Technical Report 14-01, Sandford, January 2014, based on Rowe and Armitage (1987b) per NCHRP Synthesis 360, Turner, Rock-Socketed Shafts for Highway Structure Foundations, 2006.

Pile Section	Strength Limit State Factored Axial Pile Resistance				
	Structural Resistance[1] ϕ_c=0.50 (kips)	Static Geotechnical Resistance φ_{static} = 0.45[2] (kips)	Controlling Geotechnical Resistance[3] (kips)	Drivability Resistance[4] φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	387[5]	137	387	267	267
HP 14 x 73	535[6]	190	535	390	390
HP 14 x 89	652	231	652	475	475
HP 14 x 117	860	304	860	553(546)[6]	553(546) [6]

Table 4 – Factored Axial Compressive Resistances for H-Piles at Strength Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, for the site conditions, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Local experience also supports the estimated factored resistances from the drivability analyses. Therefore, drivability controls and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 4. The maximum applied factored axial pile load should not exceed the governing factored axial pile resistance shown in Table 3 above. The actual nominal resistance of the piles during field installation may be less if piles walk out of position before reaching the established driving criteria.

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[1] Structural resistances were calculated for a braced pile segment in pure axial compression, using a resistance factor, ϕ_c, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced pile segments based upon L-Pile results using a resistance factor of $\phi_c = 0.70$ for combined axial loading and bending.

[2] Static geotechnical resistance was estimated using the Intact Rock Method (IRM) proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, January 2014, based on Rowe and Armitage (1987), NCHRP Synthesis Report 360, (2006).

[3] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock.*

[4] Nominal drivability resistances were determined based on a limiting driving criteria of 12 bpi and a maximum driving stress of 45 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria.

[5] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions for slenderness. HP12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, VTrans Structures Section, 2008).

[6] Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

8.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability considering changes in soil conditions due to scour due to the design flood (Q_{100}). For the service limit state, resistance factors of $\phi = 1.0$ should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design checks for the H-piles shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. The calculated factored axial structural, geotechnical, and drivability resistances of four H-pile sections for the extreme and service limit states are provided in Table 5. Supporting documentation is provided in Appendix D – Calculations.

Pile Section	Structural Resistance[1] ϕ_c=1.0 (kips)	Service and Extreme Limit State Factored Axial Pile Resistance			
		Static Geotechnical Resistance [2] ϕ = 1.0 (kips)	Controlling Geotechnical Resistance[3] ϕ=1.0 (kips)	Drivability Resistance[4] ϕ= 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	775 [5]	305	775	410	410
HP 14 x 73	1070	421	1070	600	600
HP 14 x 89	1305	513	1305	730	730
HP 14 x 117	1720	676	1720	850(840)[6]	850(840)[6]

Table 5 – Factored Axial Compressive Resistances for H-Piles at Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the H-pile sections are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistance for a braced pile segment. Therefore, drivability controls, and the recommended governing resistances for pile design are the resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 4. The maximum applied factored axial pile load for the extreme and service limit states should not exceed the governing factored pile resistance shown in Table 5 above. Actual nominal resistances of piles during field installation may be less if piles walk out of position before reaching the established driving criterion.

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[1] Structural resistances were calculated for a braced pile segment in pure axial compression. Factored structural resistances should be calculated for upper and lower unbraced pile segments upon L-Pile results. These resistances may be the controlling values.

[2] Static geotechnical resistance was estimated using the Intact Rock Method (IRM) per Sandford (2014) based on Rowe and Armitage (1987), NCHRP Synthesis Report 360, (2006).

[3] Nominal pile axial compressive resistance calculated per guidance in LRFD Article 10.7.3.2.3. *Piles Driven to Hard Rock.*

[4] Nominal drivability resistances were determined based on a limiting driving criteria of 12 bpi and a maximum driving stress of 45 ksi. These theoretical pile resistances may not be achievable if piles walk out of position before reaching the specified driving criteria.

[5] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions for slenderness. HP12x53 and 14x73 sections do not comply with LRFD slenderness requirements and generally should be avoided for simplified pile design methods, (ref: Integral Abutment Bridge Design Guidelines, 2nd Ed., VTrans, 2008).

[6]Drivability resistance based on a Delmag D19-42. Drivability resistance with a Delmag D36-32 shown in parentheses.

8.1.3 Lateral Pile Resistance/Behavior

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

Geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses for three foundation alternatives (driven piles, spun pipe piles and rock-socketed piles) are provided in Table 6. In general, the models developed should emulate the soil at the site by using the soil layers (referenced in Tables 6 and 7) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

LPile Input Parameters Abutment No. 1 (South) – Borings BB-ATB-101, BB-ATB-102/102B						
Soil Layer	Soil/Rock Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg) / S_u[3] (psf)/ UC (psi)	k_s[4] (pci) / ε_{50}[5] / E_m[6](psi)/ k_m[7]/ RQD (%)
Granular Fill	Reese Sand	252	13	125	32°	225
Dense Alluvium	Reese Sand	239	5	70	34°	90
Glacial Till	Reese Sand	234	6	71	36°	90
H-Pile grouted in Rock Socket	Strong Rock (Vuggy Limestone)	227	tbd	88	4000 psi	–
Aggregate-filled Rock Socket	Reese Sand	237	tbd	63	34°	125
Spun Pile in Bedrock	Weak Rock Model	227	tbd	109	9140 psi	E_m=1.45 x10^6 k_m=0.0005 RQD=78

Table 6 – Soil and Rock Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 1, for Driven Piles, Spun Pipe Piles and Rock-Socketed Piles

[1] Effective Unit Weight.
[2] Effective Internal angle of friction.
[3] Undrained shear strength.
[4] Soil modulus constant.
[5] Strain at 50 percent of the ultimate stress.
[6] Initial Modulus of Rock Mass
[7] Strain Factor (for rock)

LPile Input Parameters Abutment No. 2 (North) – Boring BB-ATB-103						
Soil Layer	Soil Model	Top Elevation of Layer (ft)	Layer Thickness (ft)	γ_e[1] (pcf)	ϕ'[2] (deg) / S_u[3] (psf)	k_s[4] (pci) / ε_{50}[5]
Granular Fill	Reese Sand	250.5	10.5	125	32°	225
Loose Alluvium	Reese Sand	240	5	52	28°	20
Medium dense Alluvium	Reese Sand	235	5	83	31°	35
Soft Silty Clay	Matlock Clay	230	5	43	750 psf	ε_{50}=0.020
Glacial Till	Reese Sand	225	6	58	36°	60

Table 7 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves at Abutment No. 2

8.1.4 Driven Pile Quality Control

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct dynamic pile load tests with signal matching at each abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrike tests are recommended to verify "pile relaxation" does not occur in the bedrock, in which the stopping criteria shall be adjusted. Further, it is recommended that one additional dynamic test may be required as part of the pile field quality control program at Abutment No. 1 should a pile tip not be firmly embedded in bedrock, or if a pile "walks" out of position. The location of the second pile test will be selected by the Resident.

With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the

[1] Effective Unit Weight.
[2] Effective Internal angle of friction.
[3] Undrained shear strength.
[4] Soil modulus constant.
[5] Strain factor.

Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving may be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

8.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutment at the strength limit state shall consider reinforced-concrete structural design.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: angle of internal friction (ϕ) of 32 degrees, total unit weight (γ) of 125 pcf, and a soil-concrete interface friction angle (δ) of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 8:

Abutment Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥20	2.0

Table 8 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 – Granular Borrow for Underwater Backfill. The gradation of this material specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to facilitate drainage and minimize frost action behind the structure.

Slopes in front of the pile supported integral abutments should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V in accordance with MaineDOT Standard Detail 610(03).

8.3 In-line Wingwalls

In-line, cantilevered wingwalls will be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, and collision loads, as well as forces due to creep, temperature, and shrinkage deformations. The design of in-line wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. These additional bending stresses may require wingwalls longer than 10 feet to be independently supported.

The design of the in-line wingwalls shall at a minimum consider a load case where the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. See Appendix D – Calculations for supporting documentation. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4. An at-rest earth pressure coefficient, K_o, of 0.47 should be used for live load surcharge loads placed upon wingwalls cantilevered off of abutments with the top of the wall restrained from movement. See Appendix D – Calculations for supporting documentation.

There are no bearing resistance considerations or special foundation supports needed for wingwalls that are cantilevered off the abutment.

8.4 Settlement

The alluvial sands encountered beneath the bridge approaches range from very loose to dense in consistency. These coarse-grained materials undergo elastic, immediate compression in response to an increase in vertical overburden pressure. A deposit of soft glaciomarine silty clay was also encountered underlying the north bridge approach. The vertical extent of the silty clay was approximately 5 feet and would typically be subject to long term consolidation settlement. However, little to no increase in vertical overburden pressure is expected. As a result, any settlement is anticipated to be small and will occur relatively quickly.

Any loose or soft material encountered at the abutment subgrade level should be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill and the exposed subgrade then thoroughly compacted. With these provisions, post-construction settlement of the approaches is anticipated to be minimal.

Settlement of the bridge abutments will be due to elastic compression of the foundation piles and is anticipated to be minimal.

8.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per MaineDOT BDG Figure 5-2.

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Auburn has an air design freezing index (DFI) of approximately 1420 F-degree days. The anticipated coarse-grained fill material was assigned a water content of 10%. These components correlate to a frost depth of 6.6 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Lewiston, Maine has an air DFI from the Modberg database of approximately 1224 F-degree days. Lewiston was selected because it lies along the same isoline as Auburn and Auburn is not available in the Modberg database. A water content of 10% was used. These components correlate to a frost depth of approximately 7.5 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.6 feet for frost protection. See Appendix D – Calculations for supporting calculations.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

8.6 Scour and Riprap

Grain size analyses were performed on samples from the stream alluvium and glaciomarine deposit to generate grain size curves for determining parameters to be used in scour analyses. Three soil samples were judged to be similar in nature to the soils likely to be exposed to scour conditions. The recommended streambed grain size parameters derived from the average D_{50} of these samples are presented in Table 9, below.

Soil Unit	Representative Zone	Sample	Sample Depth (ft)	D_{50} (mm)	Representative D_{50} (mm)	Classification
Alluvium	Streambed	BB-ATB-101 3D	15-17	0.11	0.19	Fine Sand
		BB-ATB-103 3D	15-17	0.26		
Glaciomarine	Below Streambed	BB-ATB-103 4D	20-22	0.002	0.002	Clay

Table 9 – Average D_{50} of Representative Samples for Scour

The grain size curves are included in Appendix B – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less than the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of the pile supported abutments, the abutment slopes will be armored with a layer of plain riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour design. Typically, the top of the riprap is located at, or above, the Q_{50} elevation.

Plain riprap shall conform to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The toe of the riprap section shall be constructed at least 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

8.7 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in Table 10.

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.089g
Acceleration Coefficient (A_S)	0.14g
S_{DS} (Period = 0.2 sec)	0.29g
S_{D1} (Period = 1.0 sec)	0.11g
Site Class	D
Seismic Zone	1

Table 10 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix D – Calculations for supporting documentation.

8.8 Construction Considerations

The very loose saturated alluvium poses constructability issues. Vibrating or driving temporary sheeting and piles may result in ground subsidence and ground loss. It is possible there may be subsidence associated with the withdrawal of temporary sheeting and shoring materials. Construction-phase dewatering to limit vibration-induced disturbance and settlement of the alluvium and marine sand deposits may be required.

Temporary lateral earth support systems may be required to permit construction of driven pile foundations at the proposed abutments. The remnants of a pre-existing single-span concrete slab bridge on mass concrete abutments are likely buried within the abutment foreslope at the north end of the proposed bridge. Implications of the remnants of previous bridge structures buried in the approaches include difficulty installing and damage to temporary lateral earth support systems and piles. The previous structure may impede construction efforts and the contractor should assume excavation and removal is necessary.

Construction of the proposed structure will require pile driving. The contractor should assume that the existing pipe arches and remnants of the previous abutments, if not removed entirely, will obstruct pile driving operations. The contractor shall be responsible for excavating those portions of existing structures that conflict with piles by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers.

The contractor should assume the use of conventional excavation methods, pre-augering, predrilling, rock chisels, or down-hole hammers is necessary to clear obstructions and allow pile driving activities. Care should be taken to drive H-piles within allowable tolerances without damaging the H-piles. Exploratory borings indicate that bedrock slopes downward across the site from the south to the north. Shallow bedrock increases the likelihood that piles may walk out of place before reaching the full driving criteria.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

9.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Taylor Brook Bridge in Auburn, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface explorations at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the earthwork and foundation recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

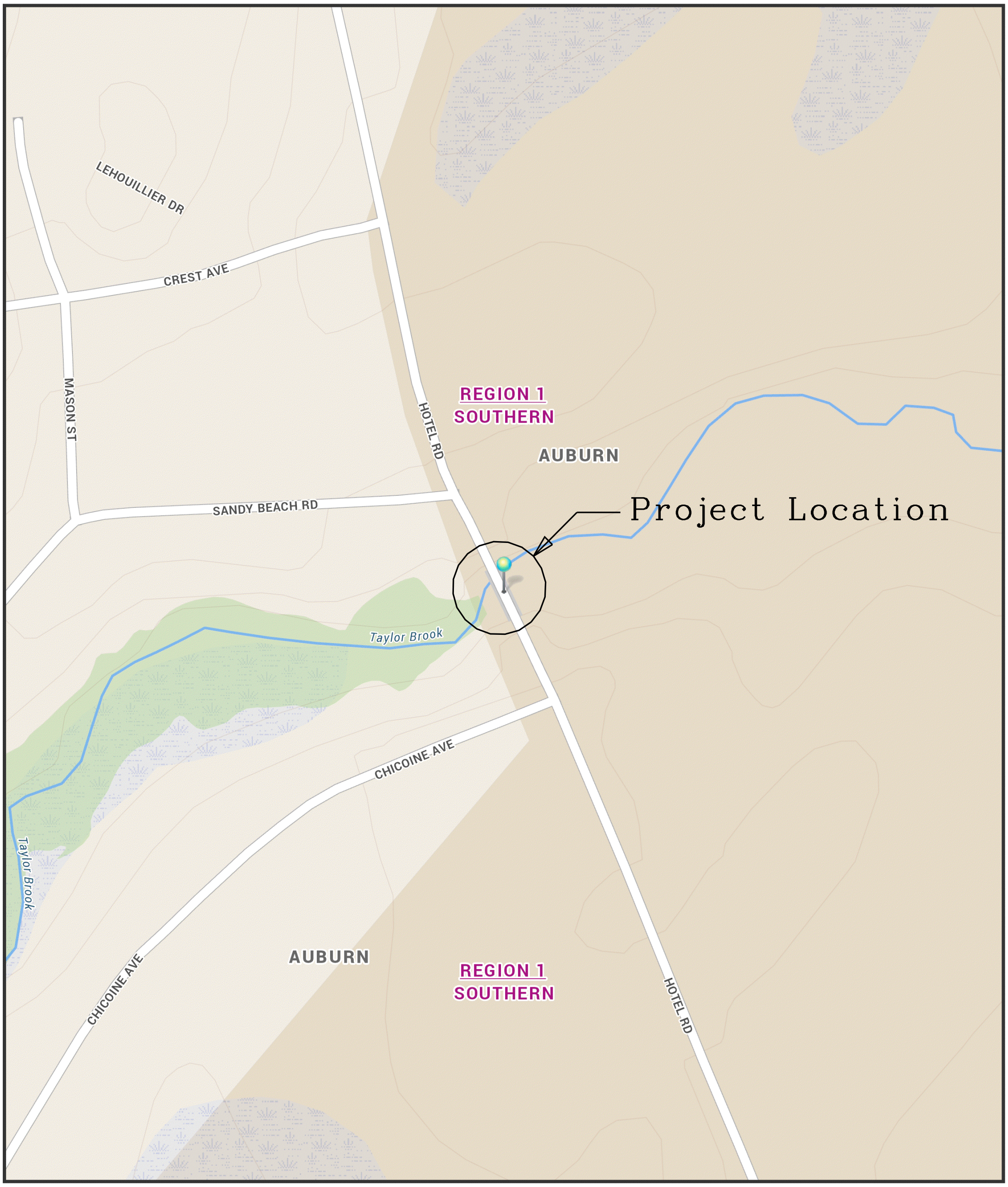
Sheets

Date:3/20/2019

Division: GEOTECH

Username: Terry.White

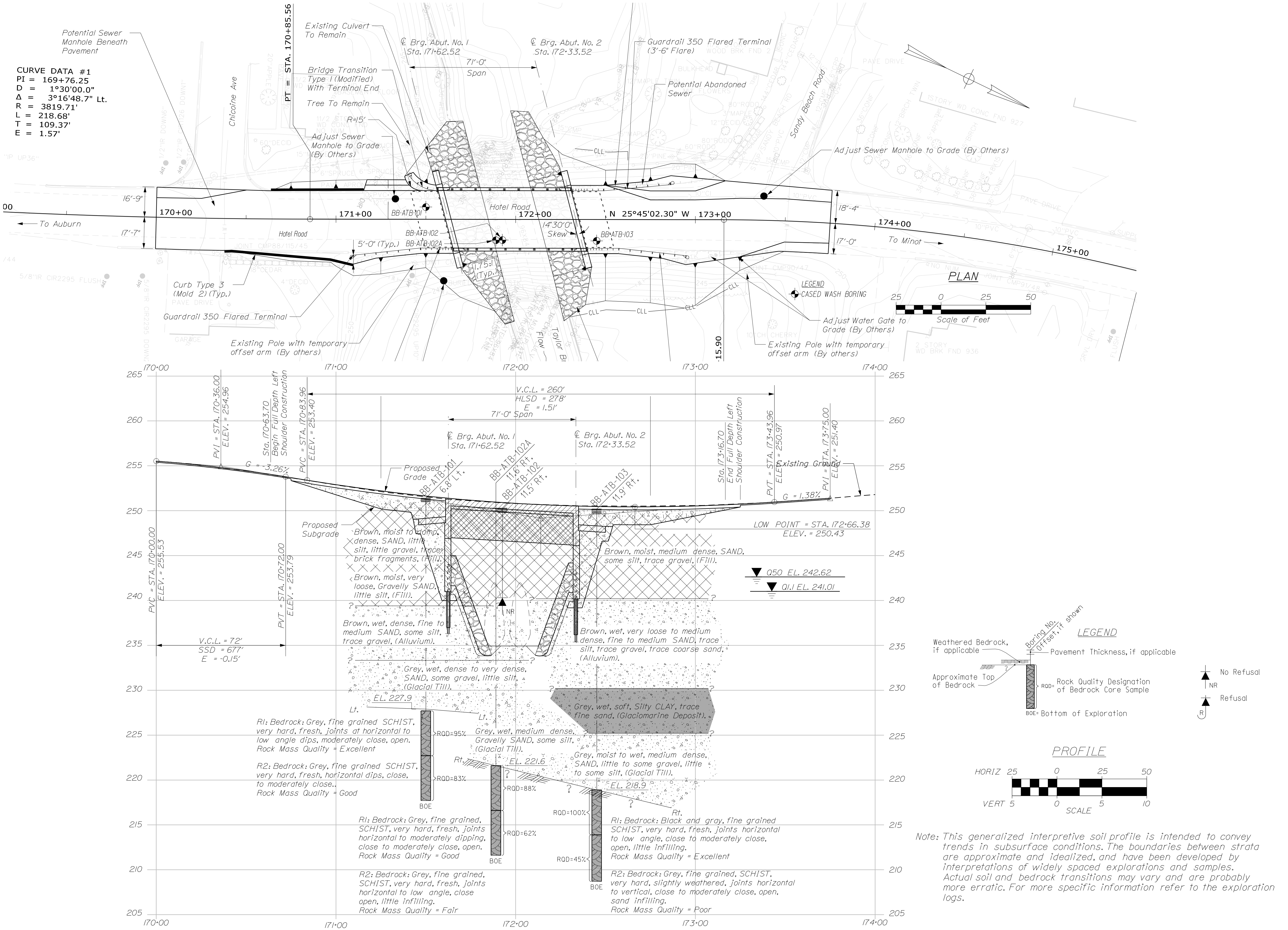
AUBURN, MAINE



0.04 Miles
1 inch = 0.04 miles

Date: 3/20/2019
Time: 9:36:32 AM

SHEET NUMBER	TAYLOR BROOK BRIDGE TAYLOR BROOK	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	AUBURN ANDROSCOGGIN COUNTY	2222400
OF 3	LOCATION MAP	WIN BRIDGE NO. 3225 022224.00 BRIDGE PLANS



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2222400

WIN 022224.00

BRIDGE PLANS

BRIDGE NO. 3225

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	A.VANBUSKIRK	
DESIGN3-DETAILED3	T.WHITE	JUN 2018
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

TAYLOR BROOK BRIDGE
TAYLOR BROOK
AUBURN ANDROSCOGGIN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: Andrew.VanBuskirk Date:3/28/2019

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-101
WIN: 22224.00

Driller:	MaineDOT	Elevation (ft.)	251.4	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/8/2018; 08:30-11:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	171+50, 6.8 ft Lt.	Casing ID/OD:	NW-3"	Water Level*:	8.2 ft bgs.
Hammer Efficiency Factor: .928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	251.0	
5	1D	24/19	5.00 - 7.00	9/16/13/13	29	45			G#270755 A-1-b, SM WC=8.2%
10	2D	24/8	10.00 - 12.00	1/1/1/14	2	3	12		G#270756 A-1-b, SW-SM WC=8.3%
							17	239.4	
							58		
							65		
							70		
15	3D	24/24	15.00 - 17.00	10/13/12/14	25	39	28		G#270757 A-2-4, SM WC=18.3%
							47		
							58	233.9	
							93		
							96		
20	4D	24/9	20.00 - 22.00	19/32/26/27	58	90	70		G#270758 A-1-b, SM WC=9.8%
							94		
							151		
	R1	60/60	23.70 - 28.70	RQD = 95%			[a]138 NQ-2	227.9 227.7	GTX 26.2-26.6 q_p= 7,859 psi
25									
	R2	60/57	28.70 - 33.70	RQD = 83%				222.7	
30									
								217.7	
35									
40									
45									
50									

Visual Description and Remarks:

5" HMA. —0.4

Brown, damp, dense, SAND, little silt, little gravel, fragments of red brick, (Fill).

Brown, moist, very loose, Gravelly SAND, little silt, (Fill). —12.0

Brown, wet, dense, fine to medium SAND, some silt, trace gravel, (Stream Alluvium). —17.5

Grey, wet, very dense, SAND, some gravel, little silt (Glacial Till).

[a]138 blows for 0.5 ft. —23.5

Top of Bedrock at Elev. 227.9 ft.
Roller Coned ahead into Bedrock to 23.7 ft bgs. —23.7

R1: Bedrock: Grey, fine grained, banded biotite, amphibole (hornblende) quartz, feldspar SCHIST with quartz and feldspar zones/veins, very hard, fresh, joints at horizontal to low angle dips, moderately close, open.
Sangerville Formation
Rock Mass Quality = Excellent
R1: Core Times (min:sec)
23.7-24.7 ft (1:27)
24.7-25.7 ft (1:24)
25.7-26.7 ft (1:51)
26.7-27.7 ft (2:01)
27.7-28.7 ft (1:34)
100% Recovery —28.7

R2: Bedrock: Grey, fine grained, SCHIST, very hard, fresh, horizontal dips, close to moderately close, open.
Sangerville Formation
Rock Mass Quality = Good
R2: Core Times (min:sec)
28.7-29.7 ft (1:43)
29.7-30.7 ft (2:05)
30.7-31.7 ft (2:12)
31.7-32.7 ft (2:03)
32.7-33.7 ft (2:07)
95% Recovery —33.7

Bottom of Exploration at 33.7 feet below ground surface.

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ATB-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-102
WIN: 22224.00

Driller:	MaineDOT	Elevation (ft.)	250.3	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/8/2018; 11:00-12:00	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	171+92.6, 11.5 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: .928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	249.8	
5	1D	24/19	5.00 - 7.00	3/8/22/12	30	46	10		G#270759 A-2-4, SM WC=10.3%
							43		
							79		
							113		
							[a]135		
10								240.3	
15									
20									
25									

Visual Description and Remarks:

6" HMA. —0.5

Brown, moist, dense, SAND, little silt, little gravel, (Fill).

[a]135 blows for 0.6 ft.

Roller Coned through cobbles from 9.6-10.0 ft bgs. —10.0

Bottom of Exploration at 10.0 feet below ground surface. Popped through Culvert. Pulled up casing and moved to BB-ATB-102A.

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ATB-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-102A
WIN: 22224.00

Driller:	MaineDOT	Elevation (ft.)	250.3	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/8/2018-5/9/2018	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	171+89, 11.6 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: .928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA		
5									
10	1D	10.8/8	10.00 - 10.90	8/50(2.8")	---				G#270760 A-2-4, SM WC=7.6%
15							1		
	2D	24/9	16.50 - 18.50	14/2/6/7	8	12	198	233.8	
							49		
							18		
							75		
20	3D	24/9	20.00 - 22.00	16/10/14/48	24	37	35		G#270761 A-1-b, SM WC=11.7%
							17	228.8	
							20		
							25		
							27		
25	4D	24/4	25.00 - 27.00	7/6/9/38	15	23	13		G#270762 A-2-4, SM WC=10.0%
							29		
							49		
	R1	60/55	28.70 - 33.70	RQD = 88%			62 NQ-2	221.6	GTX 28.7-29.1 q_p= 10,422 psi
30									
	R2	60/58	33.70 - 38.70	RQD = 62%				216.6	
35									
								211.6	
40									
45									
50									

Visual Description and Remarks:

No soil samples taken from 0 to 10 ft bgs. Soils similar to BB-ATB-102.

Brown, damp, SAND, little silt, little gravel, (Fill).

Encountered obstruction at 10.9 ft bgs. Top of culvert. —16.5

Bottom of culvert. 2D: Non-representative sample.

Grey, wet, dense, SAND, some gravel, little silt, (Glacial Till). —21.5

Grey, wet, medium dense, Gravelly SAND, some silt, (Glacial Till).

Roller Coned ahead to 28.7 ft bgs. —28.7

Top of Bedrock at Elev. 221.6 ft.
R1: Bedrock: Grey, fine grained, SCHIST with pyrite deposit and iron staining (limonite), very hard, fresh, joints horizontal to moderately dipping, close to moderately close, open.
Sangerville Formation
Rock Mass Quality = Good
R1: Core Times (min:sec)
28.7-29.7 ft (1:14)
29.7-30.7 ft (1:51)
30.7-31.7 ft (1:59)
31.7-32.7 ft (2:17)
32.7-33.7 ft (2:35)
92% Recovery —33.7

R2: Bedrock: Grey, fine grained, SCHIST with pyrite and possible graphite, very hard, fresh, joints horizontal to low angle, close, open, little infilling.
Sangerville Formation
Rock Mass Quality = Fair
R2: Core Times (min:sec)
33.7-34.7 ft (2:05)
34.7-35.7 ft (1:55)
35.7-36.7 ft (1:56)
36.7-37.7 ft (1:39)
37.7-38.7 ft (2:18)
97% Recovery —38.7

Bottom of Exploration at 38.7 feet below ground surface.

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ATB-102A

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-103
WIN: 22224.00

Driller:	MaineDOT	Elevation (ft.)	250.2	Auger ID/OD:	5" Solid Stem
Operator:	Daggett/Niles/Wilder	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	A. Van Buskirk	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/9/2018; 11:30-3:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	172+45.1, 11.9 ft Rt.	Casing ID/OD:	NW-3"	Water Level*:	None Observed
Hammer Efficiency Factor: .928		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	249.7	
5	1D	24/24	5.00 - 7.00	13/10/10/10	20	31			G#270763 A-2-4, SM WC=12.9%
10	2D	24/16	10.00 - 12.00	1/WOR/1/WOR	1	2	WOC	240.2	G#270764 A-3, SP-SM WC=23.9%
							2		
							7		
							9		
							25		
15	3D	24/9	15.00 - 17.00	5/6/3/4	9	14	17		G#270765 A-3, SP WC=33.5%
							22		
							23		
							28		
							42		
20	4D	24/24	20.00 - 22.00	1/1/1/1	2	3	26	230.2	G#270766 A-6, CL WC=43.5% LL=36 PL=23 PI=13
							30		
							29		
							20		
							23		
25	5D	24/9	25.00 - 27.00	6/8/11/22	19	29	22	225.2	G#270767 A-2-4, SM WC=10.6%
							25		
							39		
							33		
							59		
30	6D	14.4/6	30.00 - 31.20	9/12/10(2.4")	---		36		G#270768 A-1-b, SM WC=10.4%
	R1	60/60	31.30 - 36.30	RQD = 100%			[a]50 NQ-2	218.9	GTX 33.6-33.9 q_p=17,929 psi
35									
	R2	62.4/ 62.4	36.30 - 41.50	RQD = 45%				213.9	
40									
								208.7	
45									
50									

Visual Description and Remarks:

6" HMA. —0.5

Brown, moist, dense, SAND, some silt, trace gravel, (Fill). —10.0

Brown, wet, very loose, fine to medium SAND, trace coarse sand, trace silt, trace gravel, (Alluvium).

Dark brown, wet, medium dense, fine to medium SAND, trace silt, trace gravel, (Alluvium). —20.0

Grey, wet, soft, Silty CLAY, trace sand, (Glaciomarine Deposit). —25.0

Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till).
Roller Coned ahead to 30.0 ft bgs.

Grey, moist, SAND, some gravel, little silt, (Glacial Till).

[a]50 blows for 0.3 ft bgs. —31.3

Top of Bedrock at Elev. 218.9 ft.
R1: Bedrock: Black and grey, fine grained, SCHIST with pyrite, very hard, fresh, horizontal to low angle, close to moderately close, open, little infilling.
Sangerville Formation
Rock Mass Quaity = Excellent
R1: Core times (min:sec)
No core times recorded.
100% Recovery —36.3

R2: Bedrock: Grey, fine grained, SCHIST, very hard, slightly weathered, joints horizontal to vertical, close to moderately close, open, sand infilling.
Similar to R1.
Sangerville Formation
Rock Mass Quality = Poor
R2: Core Times (min:sec)
36.3-37.3 ft (1:37)
37.3-38.3 ft (1:18)
38.3-39.3 ft (1:25)
39.3-40.3 ft (1:25)
40.3-41.5 ft (1:30)
100% Recovery —41.5

Bottom of Exploration at 41.5 feet below ground surface.

Remarks:

WOC = weight of casing

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

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

Page 1 of 1
Boring No.: BB-ATB-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2222400

WIN 022224.00

BRIDGE PLANS

BRIDGE NO. 3225

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	A.VANBUSKIRK T.WHITE	MAR 2019
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

TAYLOR BROOK BRIDGE
TAYLOR BROOK
AUBURN ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

RQD (%) = sum of the lengths of intact pieces of core* > 4 inches / length of core advance

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

Correlation of RQD to Rock Mass Quality

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

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock 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))

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-101
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	251.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/8/2018; 08:30-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	171+50, 6.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	8.2 ft bgs.
Hammer Efficiency Factor:	.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	251.0		5" HMA. 0.4	
5	1D	24/19	5.00 - 7.00	9/16/13/13	29	45				Brown, damp, dense, SAND, little silt, little gravel, fragments of red brick, (Fill).	G#270755 A-1-b, SM WC=8.2%
10	2D	24/8	10.00 - 12.00	1/1/1/14	2	3	12			Brown, moist, very loose, Gravelly SAND, little silt, (Fill).	G#270756 A-1-b, SW-SM WC=8.3%
							17				
							58	239.4		12.0	
							65				
							70				
15	3D	24/24	15.00 - 17.00	10/13/12/14	25	39	28			Brown, wet, dense, fine to medium SAND, some silt, trace gravel, (Stream Alluvium).	G#270757 A-2-4, SM WC=18.3%
							47				
							58	233.9		17.5	
							93				
							96				
20	4D	24/9	20.00 - 22.00	19/32/26/27	58	90	70			Grey, wet, very dense, SAND, some gravel, little silt (Glacial Till).	G#270758 A-1-b, SM WC=9.8%
							94				
							151				
	R1	60/60	23.70 - 28.70	RQD = 95%			a138 NQ-2	227.9 227.7		a138 blows for 0.5 ft. 23.5 Top of Bedrock at Elev. 227.9 ft. Roller Coned ahead into Bedrock to 23.7 ft bgs.	
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-101
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	251.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/8/2018; 08:30-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	171+50, 6.8 ft Lt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	8.2 ft bgs.
Hammer Efficiency Factor:	.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	60/57	28.70 - 33.70	RQD = 83%				217.7		23.7 R1: Bedrock: Grey, fine grained, banded biotite, amphibole (hornblende) quartz, feldspar SCHIST with quartz and feldspar zones/ veins, very hard, fresh, joints at horizontal to low angle dips, moderately close, open. Sangerville Formation Rock Mass Quality = Excellent R1: Core Times (min:sec) 23.7-24.7 ft (1:27) 24.7-25.7 ft (1:24) 25.7-26.7 ft (1:51) 26.7-27.7 ft (2:01) 27.7-28.7 ft (1:34) 100% Recovery R2: Bedrock: Grey, fine grained, SCHIST, very hard, fresh, horizontal dips, close to moderately close, open. Sangerville Formation Rock Mass Quality = Good R2: Core Times (min:sec) 28.7-29.7 ft (1:43) 29.7-30.7 ft (2:05) 30.7-31.7 ft (2:12) 31.7-32.7 ft (2:03) 32.7-33.7 ft (2:07) 95% Recovery 33.7	GTX 26.2-26.6 q_p= 7,859 psi
30											
35										**Bottom of Exploration at 33.7 feet below ground surface.**	
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-102
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	250.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/8/2018; 11:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	171+92.6, 11.5 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: .928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	249.8		6" HMA. (0.5)	
5	1D	24/19	5.00 - 7.00	3/8/22/12	30	46	10			Brown, moist, dense, SAND, little silt, little gravel, (Fill).	G#270759 A-2-4, SM WC=10.3%
							43				
							79				
							113				
							a135			a135 blows for 0.6 ft. Roller Coned through cobbles from 9.6-10.0 ft bgs.	
10								240.3		(10.0) **Bottom of Exploration at 10.0 feet below ground surface.** Popped through Culvert. Pulled up casing and moved to BB-ATB-102A.	
15											
20											
25											

Remarks:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-102A
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	250.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/8/2018-5/9/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	171+89, 11.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.928	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			No soil samples taken from 0 to 10 ft bgs. Soils similar to BB-ATB-102.	
5											
10	1D	10.8/8	10.00 - 10.90	8/50(2.8")	---					Brown, damp, SAND, little silt, little gravel, (Fill). Encountered obstruction at 10.9 ft bgs. Top of culvert.	G#270760 A-2-4, SM WC=7.6%
15							1				
	2D	24/9	16.50 - 18.50	14/2/6/7	8	12	198	233.8		16.5 Bottom of culvert. 2D: Non-representative sample.	
							49				
							18				
							75				
20	3D	24/9	20.00 - 22.00	16/10/14/48	24	37	35			Grey, wet, dense, SAND, some gravel, little silt, (Glacial Till).	G#270761 A-1-b, SM WC=11.7%
							17	228.8		21.5	
							20				
							25				
							27				
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook **Location:** Auburn, Maine	**Boring No.:** BB-ATB-102A **WIN:** 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	250.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/8/2018-5/9/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	171+89, 11.6 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	4D	24/4	25.00 - 27.00	7/6/9/38	15	23	13			Grey, wet, medium dense, Gravelly SAND, some silt, (Glacial Till).	G#270762 A-2-4, SM WC=10.0%
							29				
							49			Roller Coned ahead to 28.7 ft bgs.	
	R1	60/55	28.70 - 33.70	RQD = 88%			62 NQ-2	221.6		28.7	
30										Top of Bedrock at Elev. 221.6 ft. R1: Bedrock: Grey, fine grained, SCHIST with pyrite deposit and iron staining (limonite), very hard, fresh, joints horizontal to moderately dipping, close to moderately close, open. Sangerville Formation Rock Mass Quality = Good R1: Core Times (min:sec) 28.7-29.7 ft (1:14) 29.7-30.7 ft (1:51) 30.7-31.7 ft (1:59) 31.7-32.7 ft (2:17) 32.7-33.7 ft (2:35) 92% Recovery	GTX 28.7-29.1 q_p= 10,422 psi
	R2	60/58	33.70 - 38.70	RQD = 62%							
35										R2: Bedrock: Grey, fine grained, SCHIST with pyrite and possible graphite, very hard, fresh, joints horizontal to low angle, close, open, little infilling. Sangerville Formation Rock Mass Quality = Fair R2: Core Times (min:sec) 33.7-34.7 ft (2:05) 34.7-35.7 ft (1:55) 35.7-36.7 ft (1:56) 36.7-37.7 ft (1:39) 37.7-38.7 ft (2:18) 97% Recovery	
								211.6		38.7	
40										**Bottom of Exploration at 38.7 feet below ground surface.**	
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-103
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	250.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/9/2018; 11:30-3:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	172+45.1, 11.9 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor: .928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	249.7		6" HMA. 0.5	
5	1D	24/24	5.00 - 7.00	13/10/10/10	20	31				Brown, moist, dense, SAND, some silt, trace gravel, (Fill).	G#270763 A-2-4, SM WC=12.9%
10	2D	24/16	10.00 - 12.00	1/WOR/1/WOR	1	2	WOC	240.2		10.0 Brown, wet, very loose, fine to medium SAND, trace coarse sand, trace silt, trace gravel, (Alluvium).	G#270764 A-3, SP-SM WC=23.9%
							2				
							7				
							9				
							25				
15	3D	24/9	15.00 - 17.00	5/6/3/4	9	14	17			Dark brown, wet, medium dense, fine to medium SAND, trace silt, trace gravel, (Alluvium).	G#270765 A-3, SP WC=33.5%
							22				
							23				
							28				
							42				
20	4D	24/24	20.00 - 22.00	1/1/1/1	2	3	26	230.2		20.0 Grey, wet, soft, Silty CLAY, trace sand, (Glaciomarine Deposit).	G#270766 A-6, CL WC=43.5% LL=36 PL=23 PI=13
							30				
							29				
							20				
25							23				

Remarks:

WOC = weight of casing

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

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

Boring No.: BB-ATB-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Taylor Brook Bridge #3225 carries Hotel Road over Taylor Brook
Location: Auburn, Maine

Boring No.: BB-ATB-103
WIN: 22224.00

Driller:	MaineDOT	**Elevation (ft.)**	250.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Daggett/Niles/Wilder	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/9/2018; 11:30-3:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	172+45.1, 11.9 ft Rt.	**Casing ID/OD:**	NW-3"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/9	25.00 - 27.00	6/8/11/22	19	29	22	225.2		25.0 Grey, wet, medium dense, SAND, some silt, little gravel, (Glacial Till). Roller Coned ahead to 30.0 ft bgs.	G#270767 A-2-4, SM WC=10.6%
							25				
							39				
							33				
							59				
30	6D	14.4/6	30.00 - 31.20	9/12/10(2.4")	---		36			Grey, moist, SAND, some gravel, little silt, (Glacial Till). a50 blows for 0.3 ft bgs.	G#270768 A-1-b, SM WC=10.4%
	R1	60/60	31.30 - 36.30	RQD = 100%			a50 NQ-2	218.9		31.3 Top of Bedrock at Elev. 218.9 ft. R1: Bedrock: Black and grey, fine grained, SCHIST with pyrite, very hard, fresh, horizontal to low angle, close to moderately close, open, little infilling. Sangerville Formation Rock Mass Quaity = Excellent R1: Core times (min:sec) No core times recorded. 100% Recovery	GTX 33.6-33.9 q_p=17,929
35											
	R2	62.4/62.4	36.30 - 41.50	RQD = 45%						R2: Bedrock: Grey, fine grained, SCHIST, very hard, slightly weathered, joints horizontal to vertical, close to moderately close, open, sand infilling. Similar to R1. Sangerville Formation Rock Mass Quality = Poor R2: Core Times (min:sec) 36.3-37.3 ft (1:37) 37.3-38.3 ft (1:18) 38.3-39.3 ft (1:25) 39.3-40.3 ft (1:25) 40.3-41.5 ft (1:30) 100% Recovery	
40											
								208.7		41.5 **Bottom of Exploration at 41.5 feet below ground surface.**	
45											
50											

Remarks:

WOC = weight of casing

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

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

Appendix B

Rock Core Photographs

MaineDOT

MaineDOT

Taylor Brook Bridge No. 3225 Carries Hotel Road Over Taylor Brook

Auburn, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-ATB-101	R1	23.7 – 28.7	60	100%	57	95%	SCHIST	1
BB-ATB-101	R2	28.7 – 33.7	57	95%	50	83%	SCHIST	2
BB-ATB-102A	R1	28.7 – 33.7	55	92%	53	88%	SCHIST	3
BB-ATB-102A	R2	33.7 – 38.7	58	97%	37	62%	SCHIST	4

Note: "Box row" indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

MaineDOT

Taylor Brook Bridge #3225 Carries Hotel Road Over Taylor Brook

Auburn, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-ATB-103	R1	31.3 – 36.3	60	100%	60	100%	SCHIST	1
BB-ATB-103	R2	36.3 – 41.5	62	100%	28	45%	SCHIST	2 and 3

Note: “Box row” indicates the section of the box where the core run is contained: 1 = top, 4 = bottom.

Appendix C

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Auburn   Work Number: 22224.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-ATB-101, 1D	171+50	6.8 Lt.	5.0-7.0	270755	1	8.2			SM	A-1-b	II
BB-ATB-101, 2D	171+50	6.8 Lt.	10.0-12.0	270756	1	8.3			SW-SM	A-1-b	0
BB-ATB-101, 3D	171+50	6.8 Lt.	15.0-17.0	270757	1	18.3			SM	A-2-4	II
BB-ATB-101, 4D	171+50	6.8 Lt.	20.0-22.0	270758	1	9.8			SM	A-1-b	II
BB-ATB-102, 1D	171+92.6	11.5 Rt.	5.0-7.0	270759	2	10.3			SM	A-2-4	II
BB-ATB-102A, 1D	171+89	11.6 Rt.	10.0-10.9	270760	2	7.6			SM	A-2-4	II
BB-ATB-102A, 3D	171+89	11.6 Rt.	20.0-22.0	270761	2	11.7			SM	A-1-b	II
BB-ATB-102A, 4D	171+89	11.6 Rt.	25.0-27.0	270762	2	10.0			SM	A-2-4	II
BB-ATB-103, 1D	172+45.1	11.9 Rt.	5.0-7.0	270763	3	12.9			SM	A-2-4	II
BB-ATB-103, 2D	172+45.1	11.9 Rt.	10.0-12.0	270764	3	23.9			SP-SM	A-3	0
BB-ATB-103, 3D	172+45.1	11.9 Rt.	15.0-17.0	270765	3	33.5			SP	A-3	0
BB-ATB-103, 4D	172+45.1	11.9 Rt.	20.0-22.0	270766	3	43.5	36	13	CL	A-6	III
BB-ATB-103, 5D	172+45.1	11.9 Rt.	25.0-27.0	270767	3	10.6			SM	A-2-4	II
BB-ATB-103, 6D	172+45.1	11.9 Rt.	30.0-31.2	270768	3	10.4			SM	A-1-b	II

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

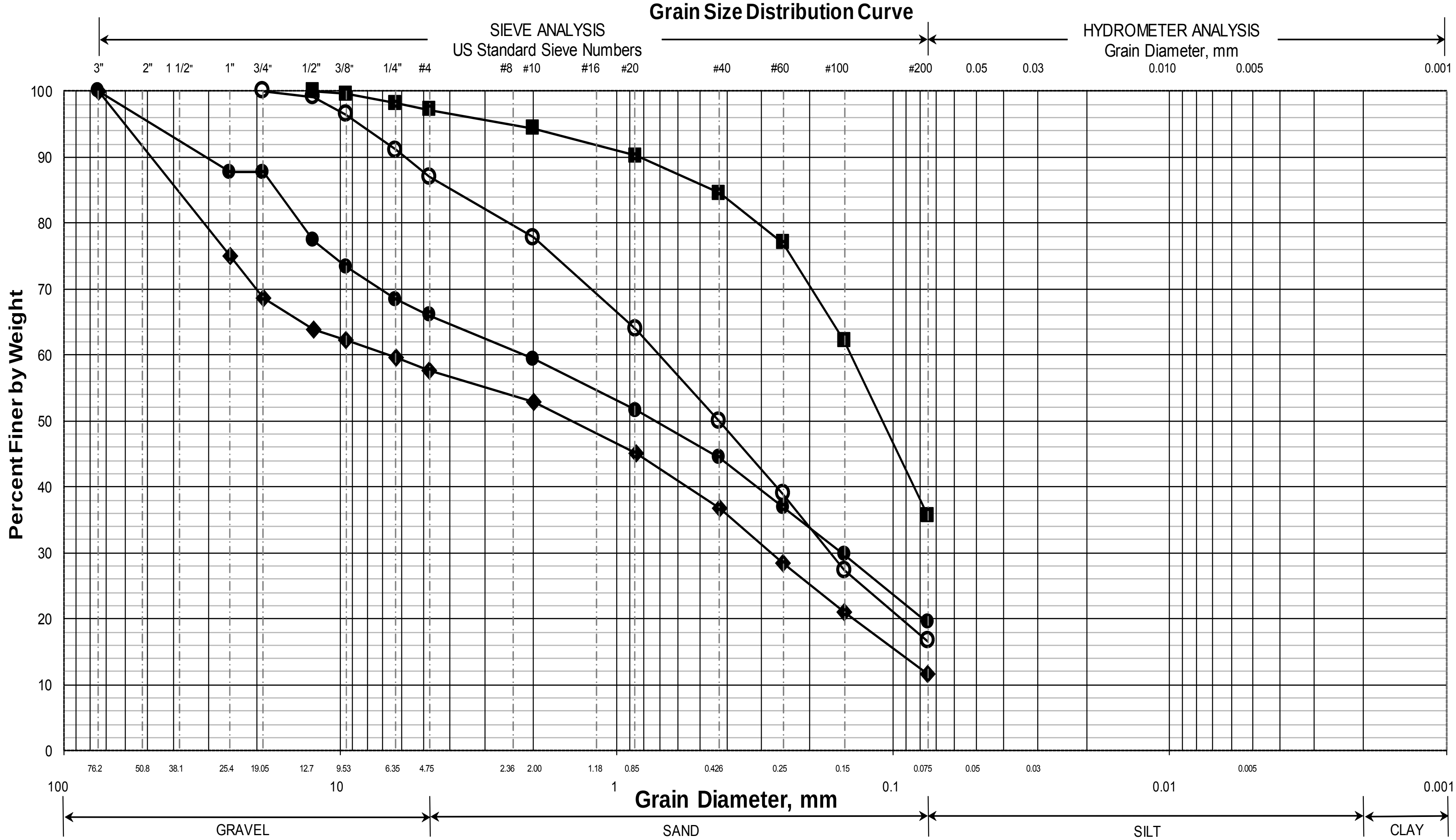
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

Maine Department of Transportation
Grain Size Distribution Curve



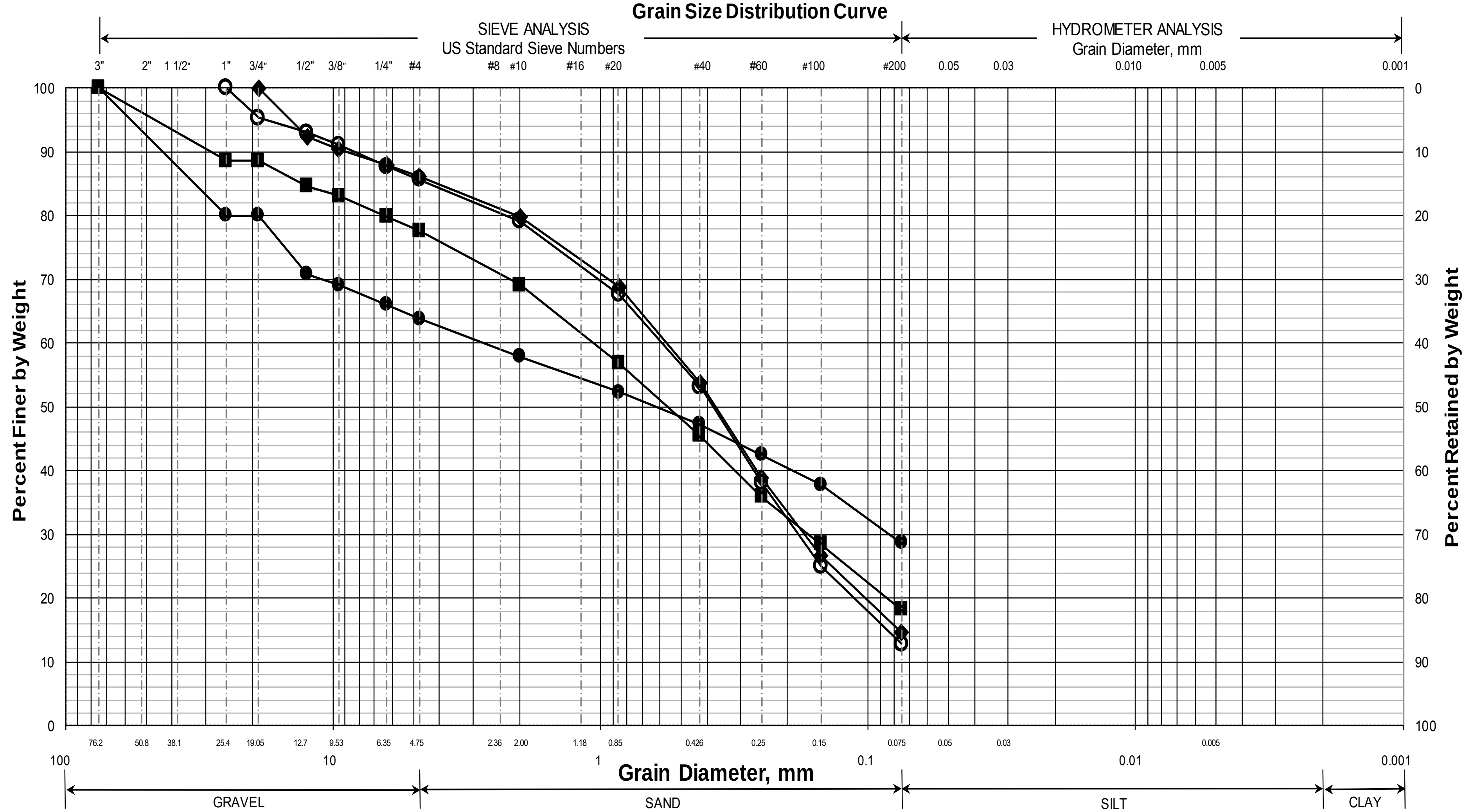
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-ATB-101/1D	171+50	6.8 LT	5.0-7.0	SAND, little silt, little gravel.	8.2			
◆	BB-ATB-101/2D	171+50	6.8 LT	10.0-12.0	Gravelly SAND, little silt.	8.3			
■	BB-ATB-101/3D	171+50	6.8 LT	15.0-17.0	SAND, some silt, trace gravel.	18.3			
●	BB-ATB-101/4D	171+50	6.8 LT	20.0-22.0	SAND, some gravel, little silt.	9.8			
▲									
×									

WIN	
022224.00	
Town	
Auburn	
Reported by/Date	
WHITE, TERRY A	3/11/2019

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



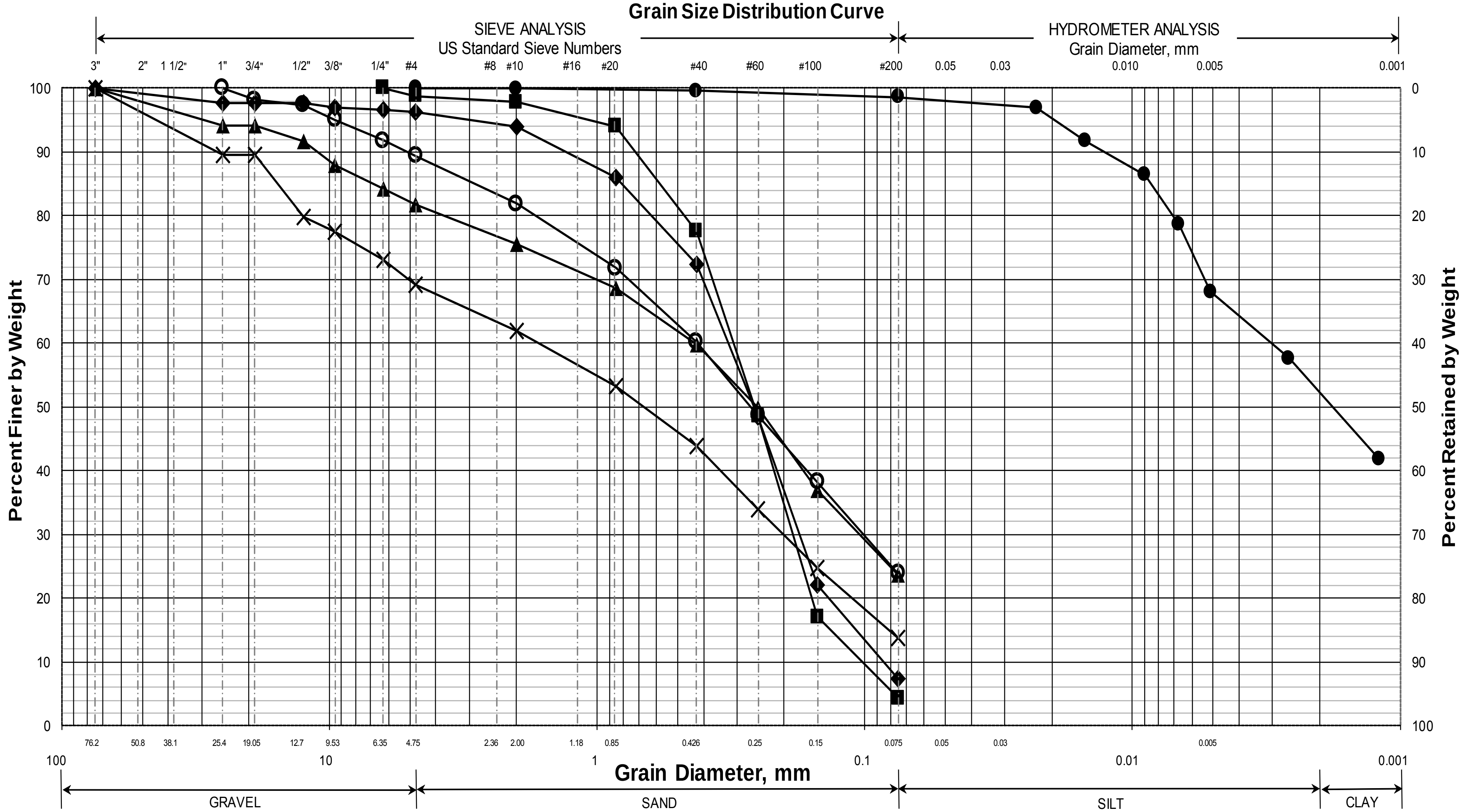
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-ATB-102/1D	171+92.6	11.5 RT	5.0-7.0	SAND, little gravel, little silt.	10.3			
◆	BB-ATB-102A/1D	171+89	11.6 RT	10.0-10.9	SAND, little silt, little gravel.	7.6			
■	BB-ATB-102A/3D	171+89	11.6 RT	20.0-22.0	SAND, some gravel, little silt.	11.7			
●	BB-ATB-102A/4D	171+89	11.6 RT	25.0-27.0	Gravelly SAND, some silt.	10			
▲									
×									

WIN	
022224.00	
Town	
Auburn	
Reported by/Date	
WHITE, TERRY A	3/11/2019

SHEET 2

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-ATB-103/1D	172+45.1	11.9 RT	5.0-7.0	SAND, some silt, trace gravel.	12.9			
◆	BB-ATB-103/2D	172+45.1	11.9 RT	10.0-12.0	SAND, trace silt, trace gravel.	23.9			
■	BB-ATB-103/3D	172+45.1	11.9 RT	15.0-17.0	SAND, trace silt, trace gravel.	33.5			
●	BB-ATB-103/4D	172+45.1	11.9 RT	20.0-22.0	Silty CLAY, trace sand.	43.5	36	23	13
▲	BB-ATB-103/5D	172+45.1	11.9 RT	25.0-27.0	SAND, some silt, little gravel.	10.6			
×	BB-ATB-103/6D	172+45.1	11.9 RT	30.0-31.2	SAND, some gravel, little silt.	10.4			

WIN	
022224.00	
Town	
Auburn	
Reported by/Date	
WHITE, TERRY A	3/11/2019

SHEET 3

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
270766	**BB-ATB-103/4D**	**GEOTECHNICAL (DISTURBED)**	**5/9/2018**	**5/16/2018**

Sample Type: **GEOTECHNICAL** Location: Station: **172+45.1** Offset, ft: **11.9 RT** Dbfg, ft: **20.0-22.0**

WIN/Town **022224.00 - AUBURN** Sampler: **ANDREW VAN**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.6**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**98.6**
[0.0228 mm]	**96.9**
[0.0150 mm]	**91.7**
[0.0090 mm]	**86.4**
[0.0067 mm]	**78.6**
[0.0051 mm]	**68.1**
[0.0026 mm]	**57.6**
[0.0012 mm]	**41.9**

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**36**
Plastic Limit (T 90), %	**23**
Plasticity Index (T 90), %	**13**
Specific Gravity, Corrected to 20°C (T 100)	**2.67**
Loss on Ignition, % (T 267)	
Water Content (T 265), %	**43.5**

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

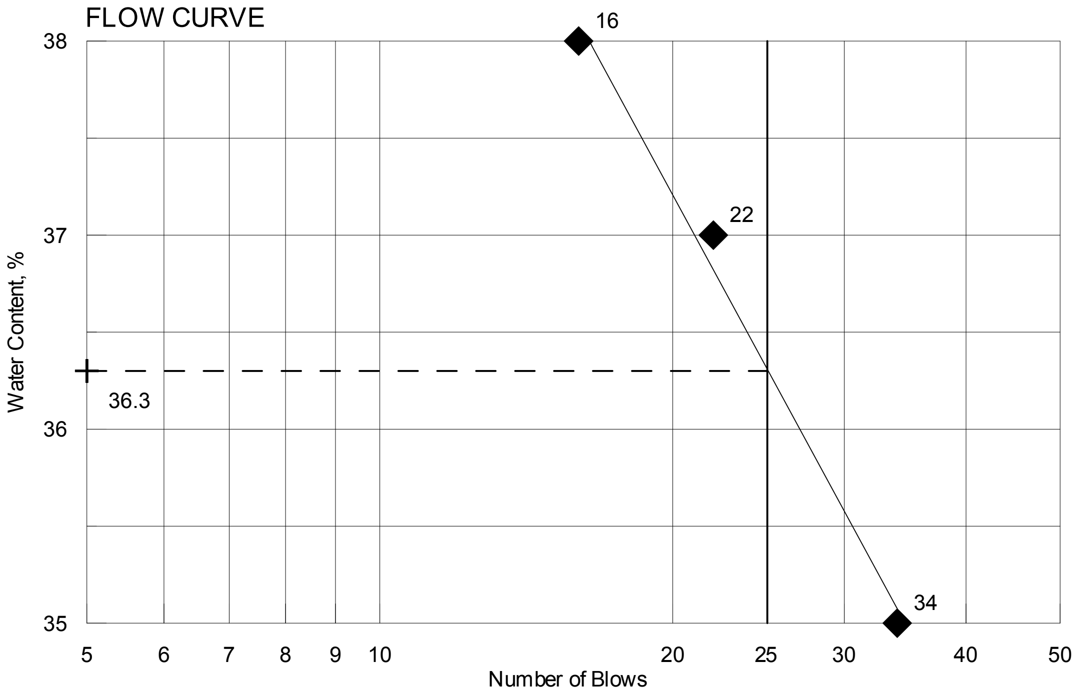
Comments:

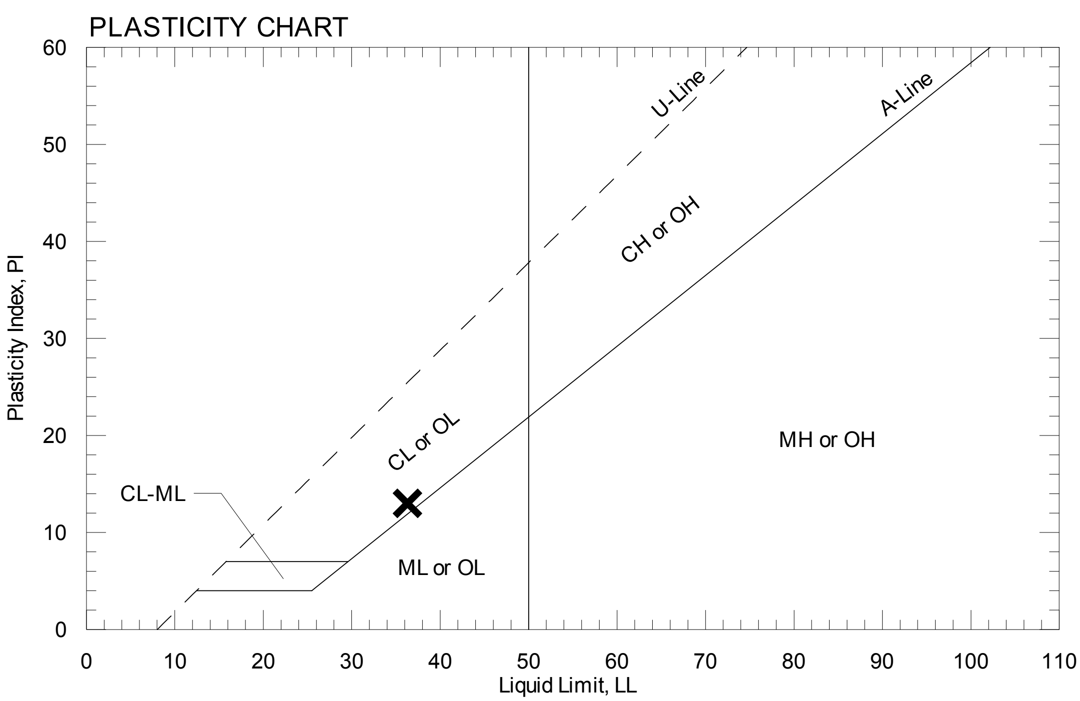
AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE** Date Reported: **5/23/2018**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	270766
WIN	022224.00	Water Content, %	43.5
Sampled	5/9/2018	Liquid Limit @ 25 blows (T 89), %	36
Boring No./Sample No.	BB-ATB-103/4D	Plastic Limit (T 90), %	23
Station	172+45.1	Plasticity Index (T 90), %	13
Depth	20.0-22.0	Tested By	BBURR

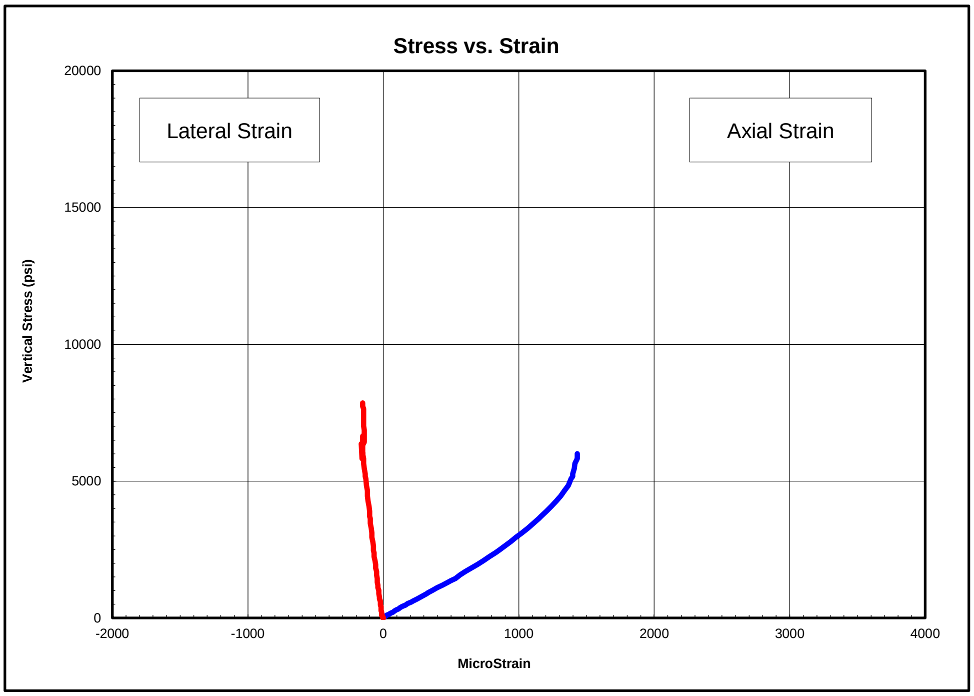




GeoTesting EXPRESS

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-101
Sample ID:	R1
Depth, ft:	26.24-26.6
Sample Type:	rock core
Sample Description:	See Photographs Intact material failure

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



Peak Compressive Stress: 7,859 psi

The graph above does not include values up to the peak stress value. The axial strain gauges failed before the peak value was attained.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
800-2900	3,010,000	0.07
2900-5000	4,880,000	0.11
5000-7100	---	---

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

GeoTesting EXPRESS

Client:	Maine Department of Transportation	Test Date:	6/22/2018
Project Name:	Taylor Brook Bridge	Tested By:	pas/trm
Project Location:	Auburn, ME	Checked By:	jsc
GTX #:	308324		
Boring ID:	BB-ATB-101		
Sample ID:	R1		
Depth:	26.24-26.6 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.16	4.16	4.16		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	567.55				
Bulk Density, lb/ft^3	168			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

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

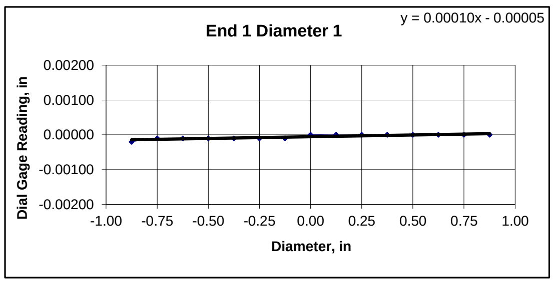
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00010

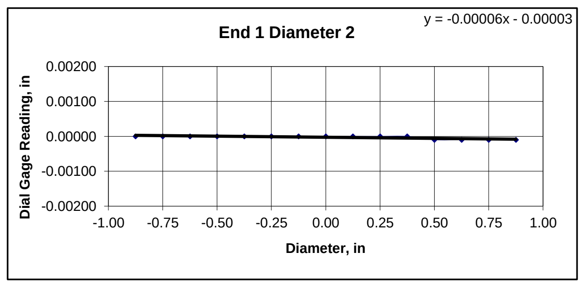
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010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

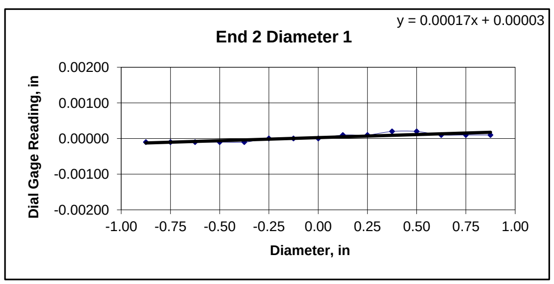
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0002

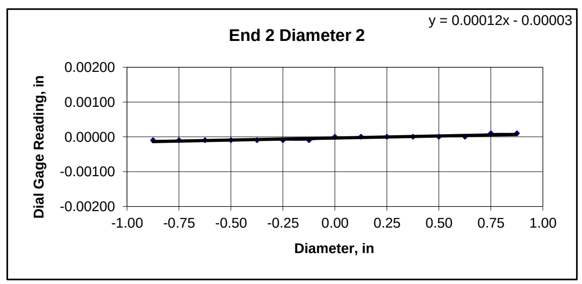
Maximum difference must be < 0.0020 in. Difference = ± 0.00015

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00010
Angle of Best Fit Line: 0.00573

End 2:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00982

Maximum Angular Difference: 0.00409

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00006
Angle of Best Fit Line: 0.00360

End 2:
Slope of Best Fit Line 0.00012
Angle of Best Fit Line: 0.00671

Maximum Angular Difference: 0.00311

Parallelism Tolerance Met? YES
Spherically Seated

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

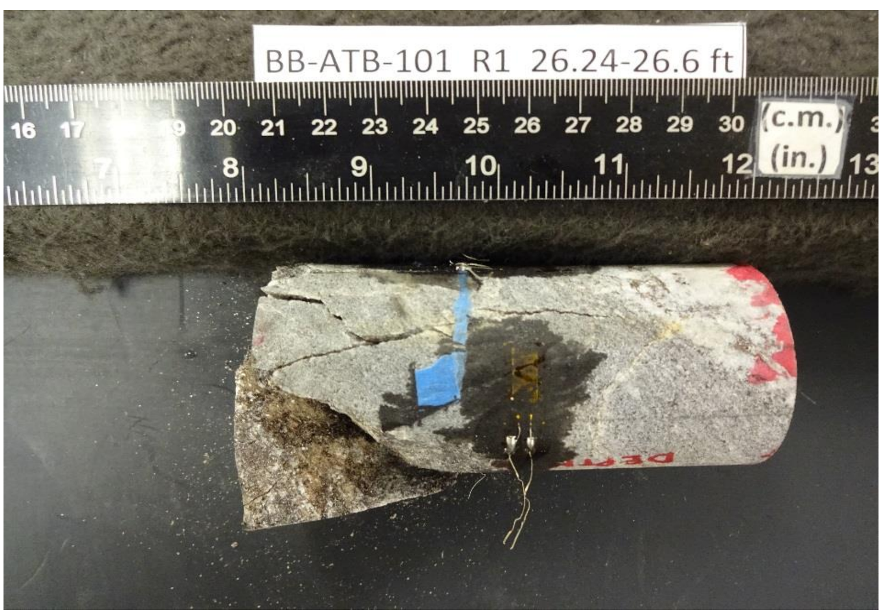
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-101
Sample ID:	R1
Depth, ft:	26.24-26.6 ft

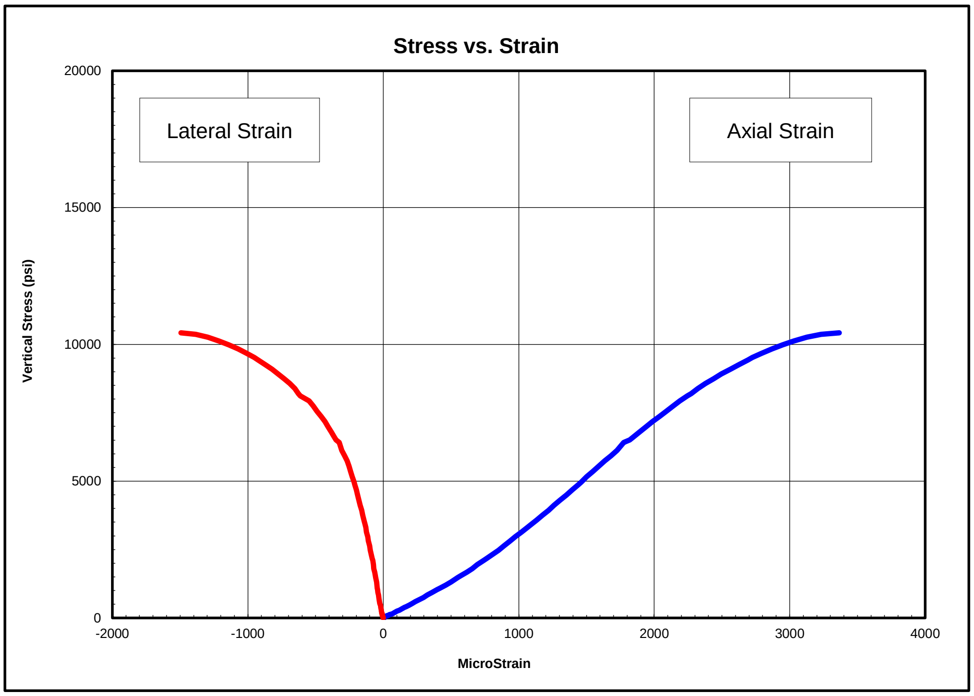
After cutting and grinding



After break

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-102A
Sample ID:	R1
Depth, ft:	28.73-29.08
Sample Type:	rock core
Sample Description:	See Photographs Intact material failure

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



Peak Compressive Stress: 10,422 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3800	3,470,000	0.14
3800-6600	4,360,000	0.31
6600-9400	3,430,000	---

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

Client:	Maine Department of Transportation	Test Date:	6/22/2018
Project Name:	Taylor Brook Bridge	Tested By:	pas/trm
Project Location:	Auburn, ME	Checked By:	jsc
GTX #:	308324		
Boring ID:	BB-ATB-102B		
Sample ID:	R1		
Depth:	28.73-29.08 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.13	4.12	4.13
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	580.61		
Bulk Density, lb/ft^3	174	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

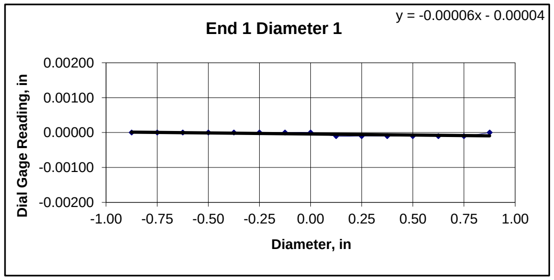
END FLATNESS AND PARALLELISM (Procedure FP1)

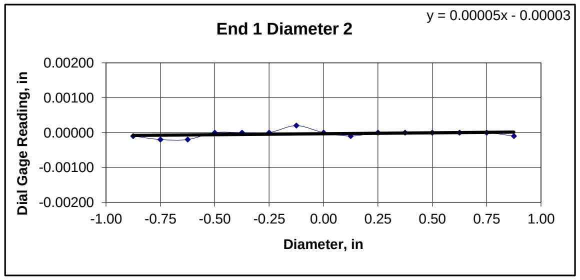
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00020	-0.00020	0.00000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

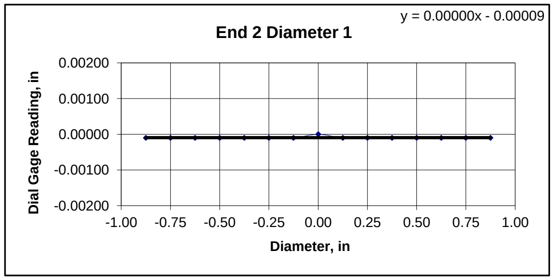
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00040

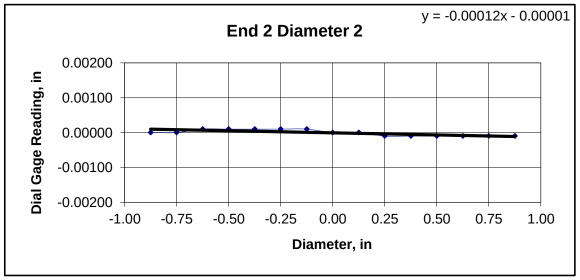
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00010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

Difference between max and min readings, in:
0° = 0.0001 90° = 0.0002
Maximum difference must be < 0.0020 in. Difference = ± 0.00020
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00006
Angle of Best Fit Line: 0.00344

End 2:
Slope of Best Fit Line 0.00000
Angle of Best Fit Line: 0.00000

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00005
Angle of Best Fit Line: 0.00311

End 2:
Slope of Best Fit Line 0.00012
Angle of Best Fit Line: 0.00688

Maximum Angular Difference: 0.00377

Parallelism Tolerance Met? **YES**
Spherically Seated

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

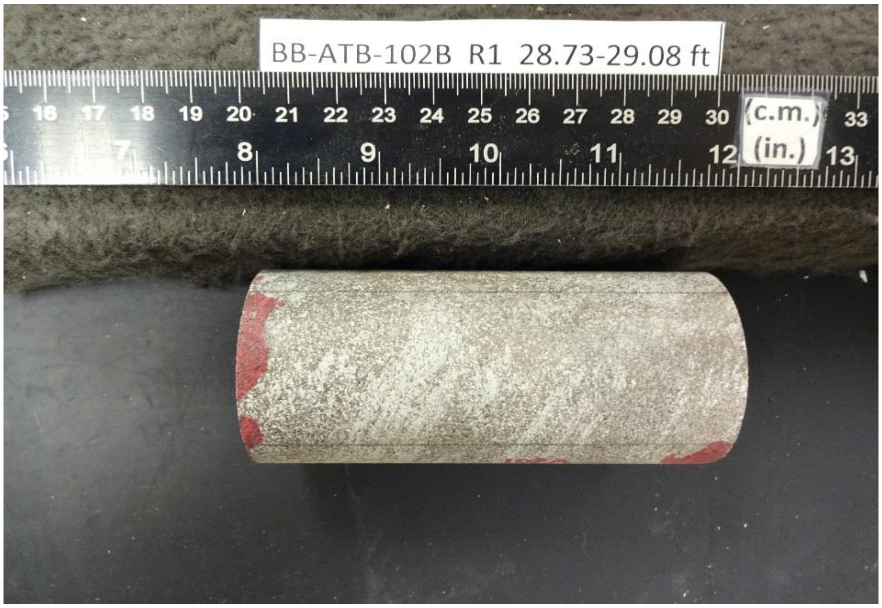
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

GeoTesting EXPRESS

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-102A
Sample ID:	R1
Depth, ft:	28.73-29.08 ft

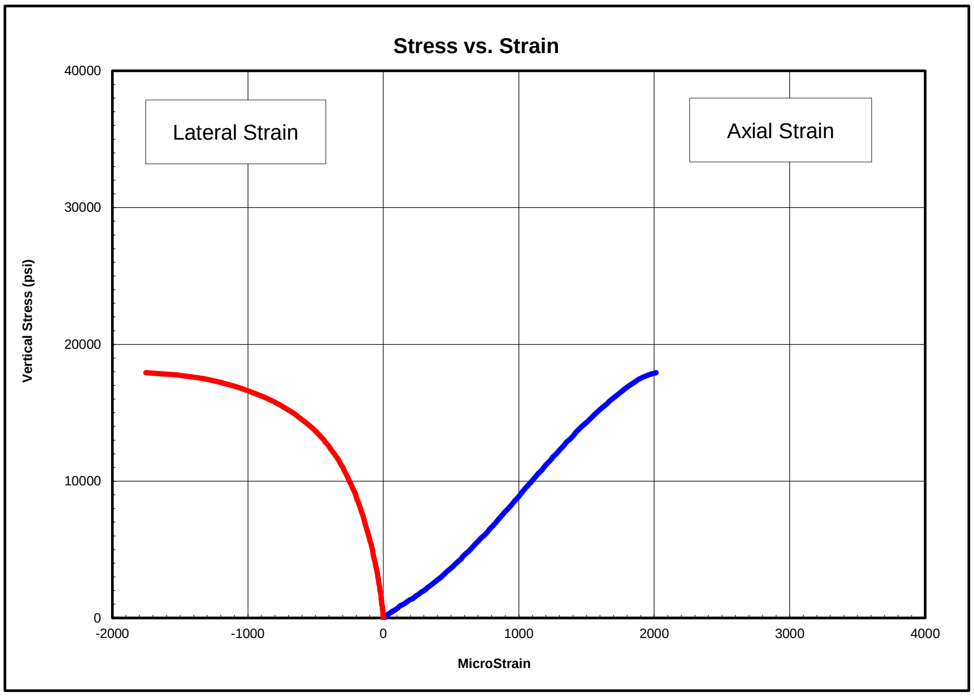


After cutting and grinding

After break

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-103
Sample ID:	R1
Depth, ft:	33.56-33.92
Sample Type:	rock core
Sample Description:	See Photographs Intact material failure

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



Peak Compressive Stress: 17,929 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6600	9,140,000	0.20
6600-11400	10,300,000	0.31
11400-16100	9,880,000	---

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

Client:	Maine Department of Transportation	Test Date:	6/22/2018
Project Name:	Taylor Brook Bridge	Tested By:	pas/trm
Project Location:	Auburn, ME	Checked By:	jsc
GTX #:	308324		
Boring ID:	BB-ATB-103		
Sample ID:	R1		
Depth:	33.56-33.92 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.16	4.16	4.16		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	641.16				
Bulk Density, lb/ft^3	190	**Minimum Diameter Tolerence Met?**		**YES**	
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**		**YES**	

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

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

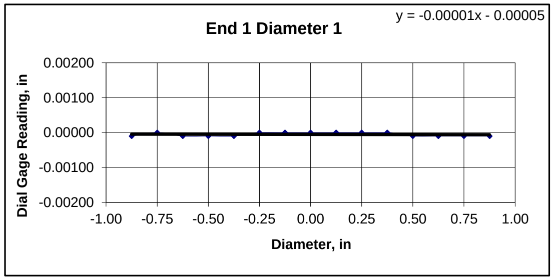
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00010

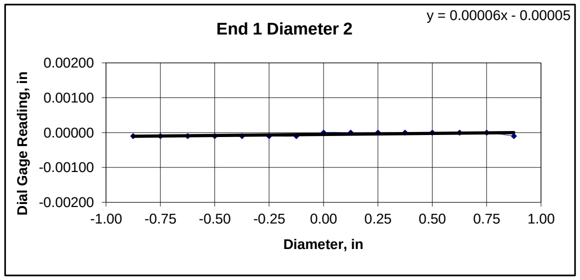
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00020	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010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

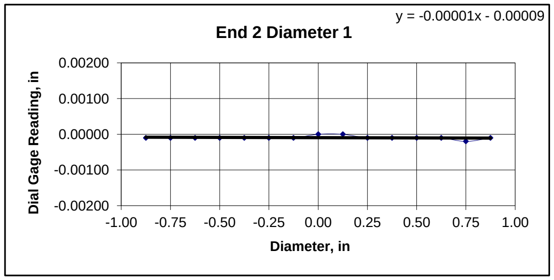
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0002

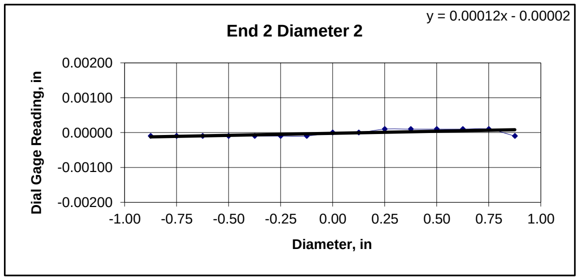
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00010

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00001
	Angle of Best Fit Line:	0.00049
End 2:		
	Slope of Best Fit Line	0.00001
	Angle of Best Fit Line:	0.00082
Maximum Angular Difference:		0.00033

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00006
	Angle of Best Fit Line:	0.00344
End 2:		
	Slope of Best Fit Line	0.00012
	Angle of Best Fit Line:	0.00671
Maximum Angular Difference:		0.00327

Parallelism Tolerance Met? **YES**
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Maine Department of Transportation
Project Name:	Taylor Brook Bridge
Project Location:	Auburn, ME
GTX #:	308324
Test Date:	6/28/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-ATB-103
Sample ID:	R1
Depth, ft:	33.56-33.92 ft

After cutting and grinding

After break

Appendix D

Calculations

H-Pile Resistance

Intact Rock Method (Sandford [2014])

Input
Calculation
Output
Linked Cell

Constants

φ_static (tip)	0.45	LRFD Table 10.5.5.2.3-1 for CGS Method on Rock

Pile Dimensions

	Depth (d)	Flange Width (b)	A_{tip}
	(in.)	(in.)	in.^{2}
12x53	11.80	12.00	15.5
14x73	13.60	14.60	21.4
14x89	13.80	14.70	26.1
14x117	14.20	14.90	34.4

Notes:

Dimensions are manufacturer suppled from skylinesteel (see attached).

Compressive Strength of Bedrock Samples

Material	Boring	Core Run	Depth (ft)	RQD	q_u (ksi)	Source of q_u
Schist	BB-ATB-101	R1	26.24 - 26.6	95%	7.859	GTX #308324

Intact Rock Method

	min (q_u)	A_{tip}	q_{tip} = 2.5 * min(q_u)	$R_{n\_tip} = A_{tip} * q_{tip}$	$R_{r\_tip} = \phi_{static} * R_n$
	ksi	in.^{2}	ksi	kips	kips
12x53	**7.9**	15.5	**19.6**	**304.5**	**137.0**
14x73	**7.9**	21.4	**19.6**	**420.5**	**189.2**
14x89	**7.9**	26.1	**19.6**	**512.8**	**230.8**
14x117	**7.9**	34.4	**19.6**	**675.9**	**304.1**

Note: For Intact Rock Method, see Sandford (2014) citing NCHRP Sythesis 360, which cites Rowe and Armitage (1987b).

Reference:

Sandford, Thomas, PhD, P.E. and Stuart, Cameron, E.I.T. MaineDOT Transportation Research Division Technical Report 14-01. January 2014.

Turner, John, NCHRP Synthesis 360, Rock-Socketed Shafts for Highway Structure Foundations, 2006.

Rowe, R.K. and H.H. Armitage. "A Design Method for Drilled Piers in Soft Rock." *Canadian Geotechnical Journal* , Vol. 24, 1987b, pp. 126-142.

For H-pile dimensions, see skylinesteel data sheet (attached).

AASHTO LRFD Bridge Design Specifications. 8th Edition. 2017

Compressive Structural Strength of H-Pile (pure axial compression)

Constants			Comments
E	ksi	29,000	LRFD C4.6.2.5-1 (a)
K	-	0.65	
F_y	ksi	50	
l_unbraced	in.	1	nominal unbraced length

Input
Calculation
Output

	r (y-y)	A_s	
	in.	in.^{2}	Comments
12x53	2.86	15.5	ref: Skyline steel Data Sheet
14x73	3.49	21.4	
14x89	3.53	26.1	
14x117	3.59	34.4	

		Comments
Φ_c	0.50	LRFD 6.5.4.2: "For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary."
Φ_service	1.0	LRFD 1.3.2.1 "For service and extreme limit states resistance factors shall be taken as 1.0, except for . . . bolts . . . and concrete columns."
Φ_extreme	1.0	

	P_{e_braced}
	(kips)
12x53	8.59E+07
14x73	1.77E+08
14x89	2.20E+08
14x117	3.00E+08

$$Pe = \frac{\pi^2 E}{(\frac{Kl}{r})^2} x\ Ag$$ (LRFD Eq. 6.9.4.1.2-1)

	P_o
	(kips)
12x53	775
14x73	1070
14x89	1305
14x117	1720

$$P_o = QFyAg$$ (LRFD Article 6.9.4.1.1)

	P_{e_braced} / P_o	>= .44
12x53	110824	TRUE
14x73	165026	TRUE
14x89	168830	TRUE
14x117	174618	TRUE

$$If\ \frac{Pe}{Po} < .44, then\ Pn = .877Pe$$
(LRFD Eq. 6.9.4.1.1-2)

$$If\ \frac{Pe}{Po} \geq .44, then\ Pn = [.658^{(\frac{Po}{Pe})}]\ P_o$$
(LRFD Eq. 6.9.4.1.1-1)

	P_n_braced
	(kips)
12x53	775
14x73	1070
14x89	1305
14x117	1720

	$P_r = \Phi_{\_strength} P_n$	$P_r = \Phi_{\_service} P_n$	$P_r = \Phi_{\_extreme} P_n$
	(kips)	(kips)	(kips)
12x53	387	775	775
14x73	535	1070	1070
14x89	652	1305	1305
14x117	860	1720	1720

Input
Linked Cell
Output

Controlling Geotechnical Resistance

	Structural Resistance (kips)	φ_c	Controlling Geotechnical Resistance (kips)
12x53	775	0.5	**387**
14x73	1070	0.5	**535**
14x89	1305	0.5	**652**
14x117	1720	0.5	**860**

Note: Based on guidance in LRFD Article 10.7.3.2.3., Piles Driven to Hard Rock. The nominal axial geotechnical resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural resistance values obtained from LRFD Article 6.9.4.1 with a resistance factor φ_c of 0.50 for severe driving conditions applied.

Drivability Analyses

Ref: LRFD Article 10.7.8

For steel piles in compression or tension, driving stresses are limited to 90% of fy

$\phi_{da} := 1.0$ Resistance factor from LRFD Table 10.5.5.2.3-1, Drivablity Analysis, steel piles

$\sigma_{dr} := 0.90 \cdot 50 \cdot (ksi) \cdot \phi_{da}$

$\sigma_{dr} = 45 \cdot ksi$ Driving stress cannot exceed 45 ksi

Limit driving stress to 45 ksi or limit blow count to 5-12 blows per inch (bpi)
(Note: 6-10 bpi is considered optimal for diesel hammers).

Compute the resistance that can be achieved in a drivablity analysis:

The resistance that must be achieved in a drivablity analysis will be the maximum factored pile load divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

$\phi_{dyn} := 0.65$ Reference LRFD Table 10.5.5.2.3-1 - for Strength Limit State

$\phi := 1.0$ For Extreme and Service Limit States

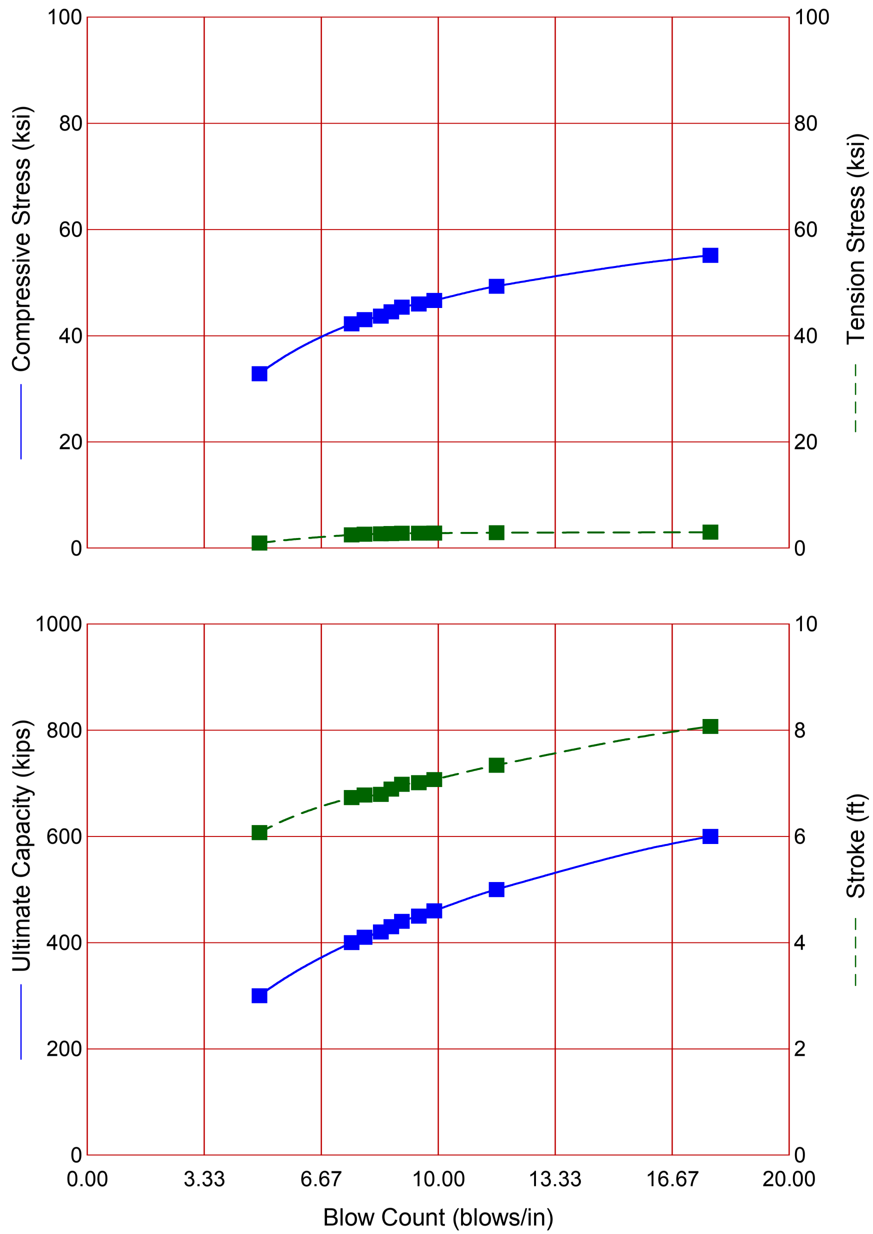
GRLWeap Soil and Pile Model Assumptions

Based on Table 2 of this Report, estimated pile lengths will be approx. from 14 to 22 ft. Drivability resistance at Abutment No. 1 will govern. Assume contractor drives pile lengths of 20 ft (extra length accommodates for attachment of dynamic testing equipment, embedment into abutment, variation in bedrock surface).

Use constant shaft resistances so that GRLWeap will assign approx. 90 kips of the ultimate capacities as skin friction, based on local experience.

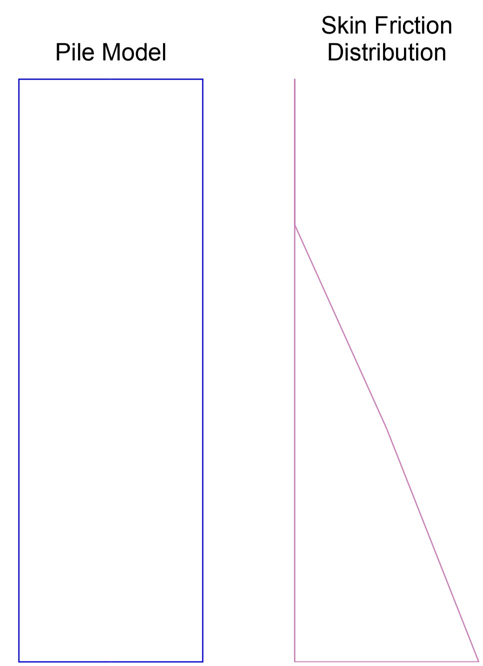
Maine DOT
Auburn 22224, HP 12x53, D 19-42, set 4

04-Jan-2019
GRLWEAP Version 2010



DELMAG D 19-42

Ram Weight	4.00 kips
Efficiency	0.800
Pressure	1165 (73%) psi
Helmet Weight	2.70 kips
Hammer Cushion	109975 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	20.00 ft
Pile Penetration	15.00 ft
Pile Top Area	15.50 in2



Res. Shaft = 90.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
300.0	32.82	0.96	4.9	6.07	10.67
400.0	42.23	2.51	7.5	6.73	11.67
410.0	43.02	2.63	7.9	6.78	11.76
420.0	43.67	2.70	8.4	6.79	11.76
430.0	44.50	2.75	8.7	6.89	11.94
440.0	45.37	2.80	9.0	6.98	12.13
450.0	45.95	2.82	9.4	7.01	12.17
460.0	46.63	2.85	9.9	7.07	12.27
500.0	49.30	2.91	11.7	7.34	12.80
600.0	55.14	3.01	17.8	8.07	14.28

Limit bpi to less than 12.

$R_{dr\_12x53} := 410kip$

Strength Limit State

$\varphi_{d\_yn} := 0.65$

$R_{dr\_12x53\_strength} := R_{dr\_12x53} \cdot \varphi_{d\_yn} = 267 \cdot kip$

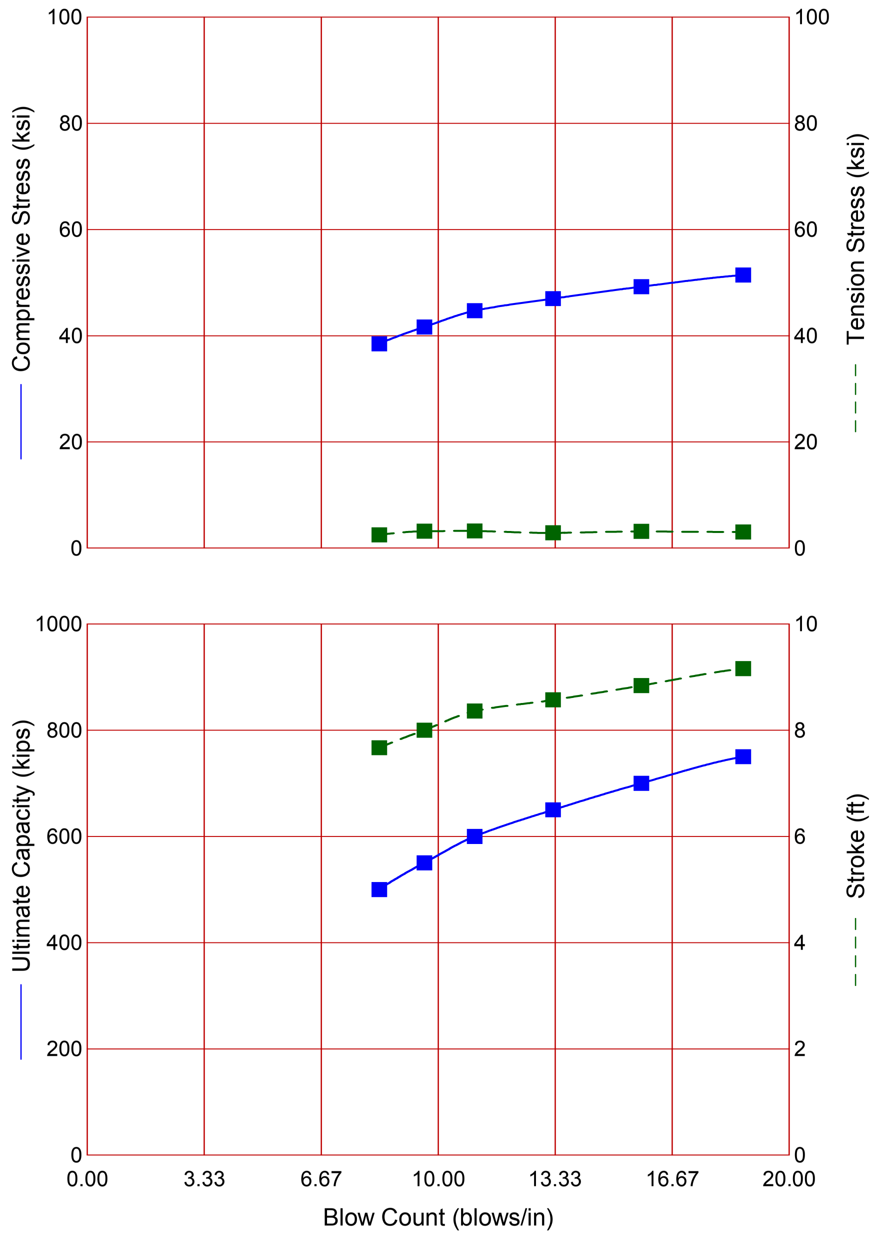
Service and Extreme Limit State

$\varphi := 1.0$

$R_{dr\_12x53\_servext} := R_{dr\_12x53} \cdot \varphi = 410 \cdot kip$

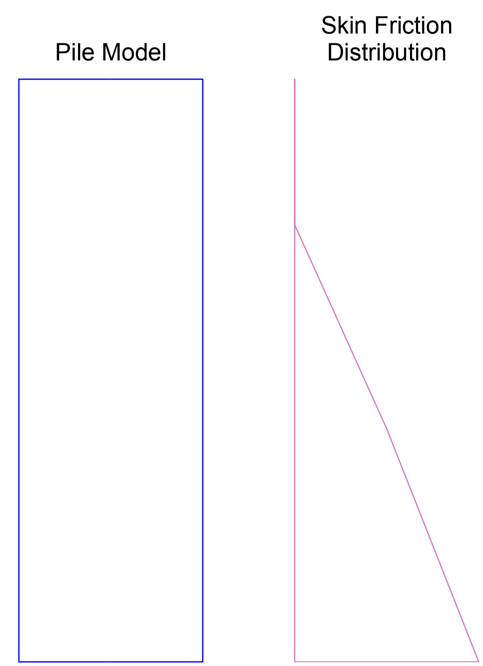
Maine DOT
Auburn 22224, HP 14x73, D 19-42, set 3

07-Jan-2019
GRLWEAP Version 2010



DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1295 (81%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	15.00	ft
Pile Top Area	21.40	in2



Res. Shaft = 90.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
500.0	38.48	2.50	8.3	7.67	13.45
550.0	41.63	3.19	9.6	8.00	14.03
600.0	44.70	3.25	11.0	8.36	14.69
650.0	46.98	2.88	13.3	8.57	14.96
700.0	49.23	3.15	15.8	8.84	15.41
750.0	51.42	3.05	18.7	9.16	15.98

Limit bpi to less than 12.

$R_{dr\_14x73} := 600kip$

Strength Limit State

$\varphi_{d\ yn} := 0.65$

$R_{dr\_14x73\_strength} := R_{dr\_14x73} \cdot \varphi_{d\ yn} = 390 \cdot kip$

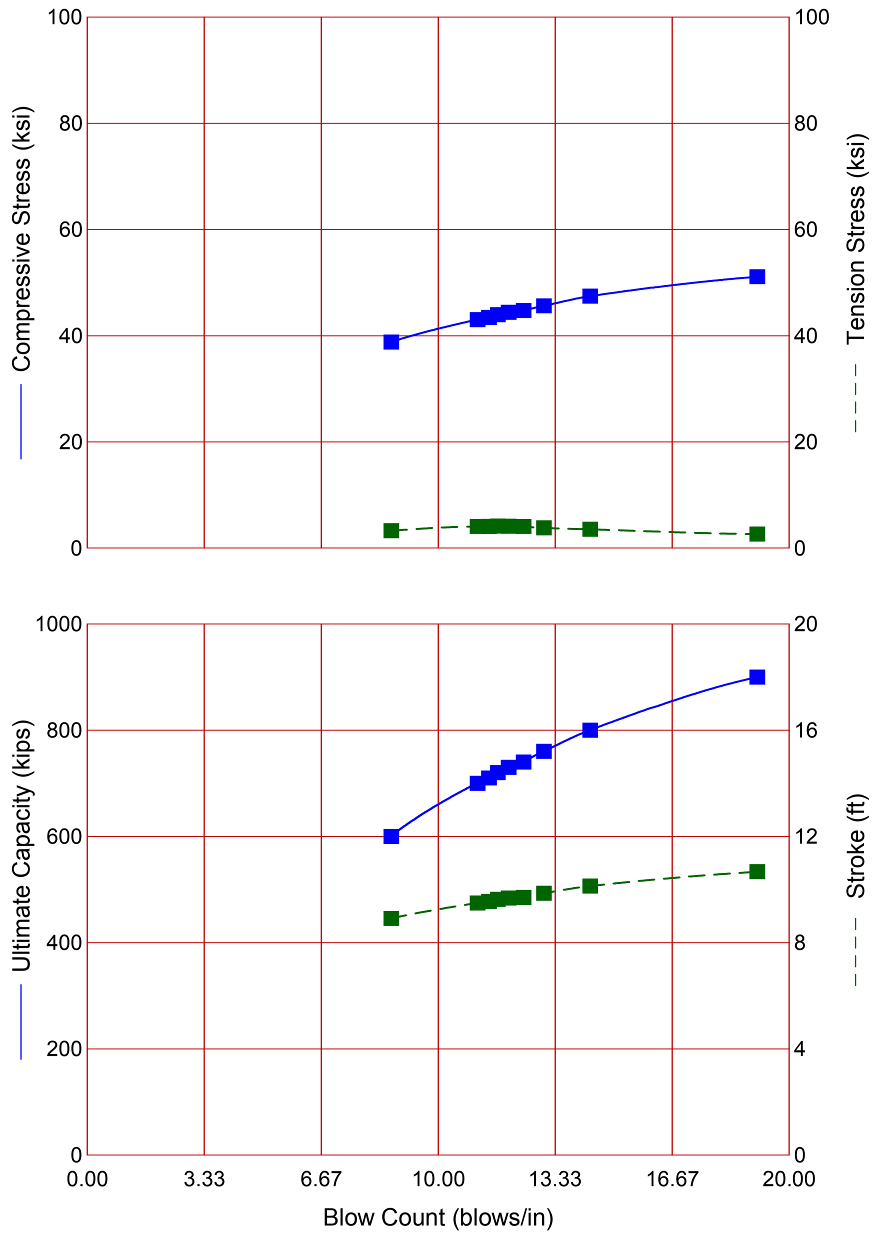
Service and Extreme Limit State

$\varphi := 1.0$

$R_{dr\_14x73\_servext} := R_{dr\_14x73} \cdot \varphi = 600 \cdot kip$

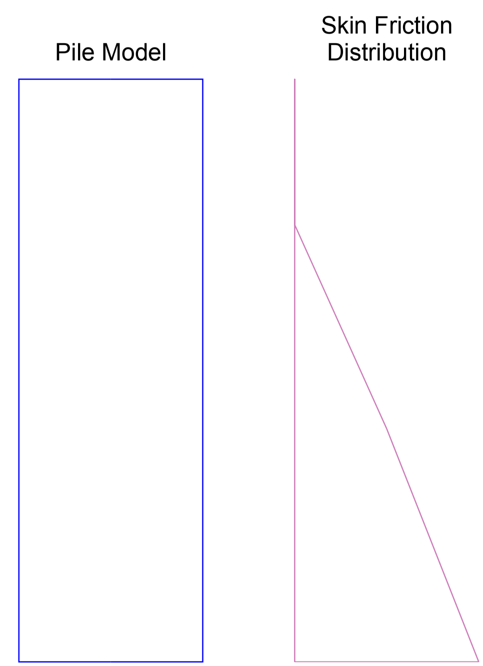
Maine DOT
Auburn 22224, HP 14x89, D 19-42, set 2

07-Jan-2019
GRLWEAP Version 2010



DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1440 (90%)	psi
Helmet Weight	2.70	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	15.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 90.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
600.0	38.79	3.27	8.7	8.91	15.98
700.0	43.01	4.09	11.1	9.49	16.92
710.0	43.44	4.11	11.4	9.55	16.98
720.0	43.95	4.15	11.7	9.63	17.12
730.0	44.41	4.15	12.0	9.68	17.21
740.0	44.74	4.08	12.4	9.70	17.21
760.0	45.62	3.82	13.0	9.86	17.47
800.0	47.44	3.56	14.3	10.13	17.95
900.0	51.11	2.65	19.1	10.67	18.76

Limit bpi to less than 12.

$R_{dr\_14x89} := 730kip$

Strength Limit State

$\varphi_{d\ \ yn} := 0.65$

$R_{dr\_14x89\_strength} := R_{dr\_14x89} \cdot \varphi_{d\ \ yn} = 475 \cdot kip$

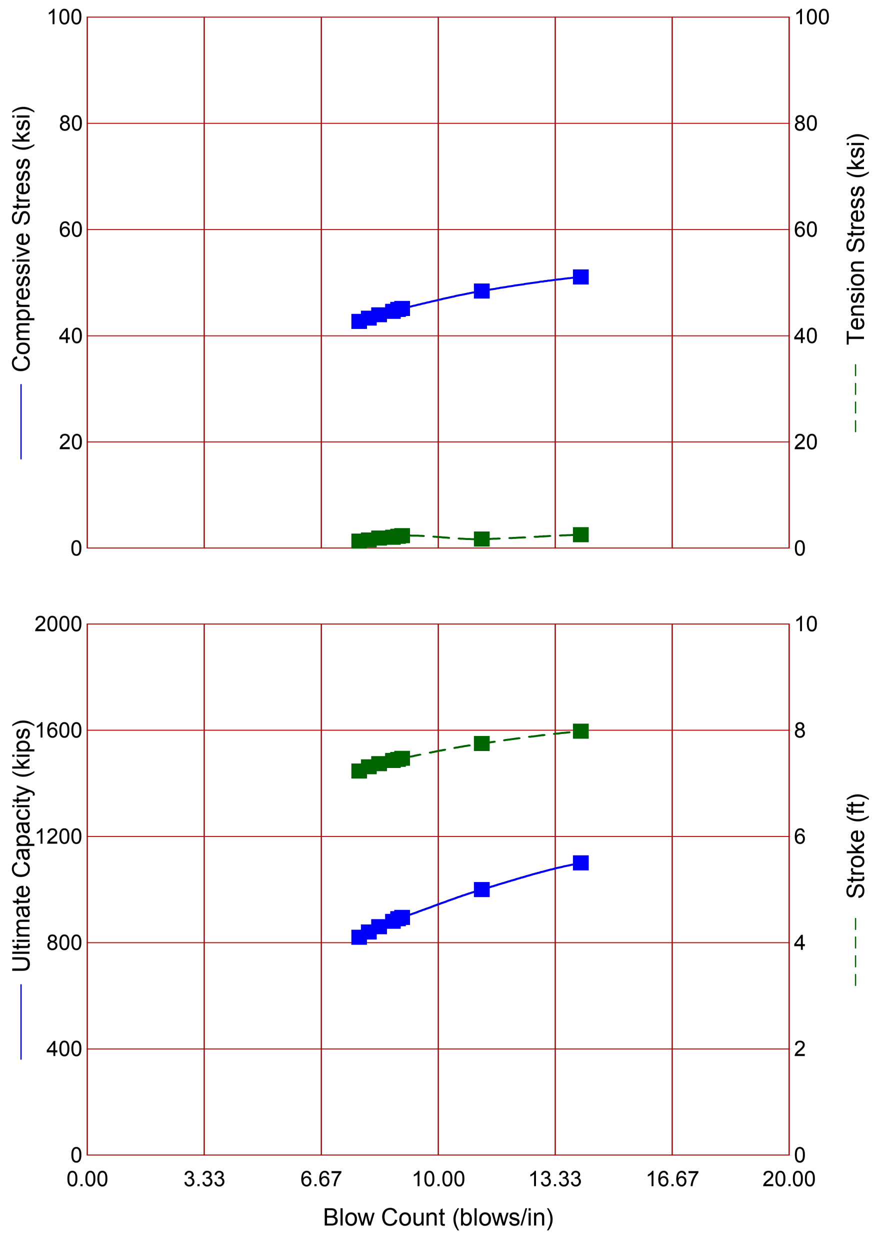
Service and Extreme Limit State

$\varphi := 1.0$

$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \varphi = 730 \cdot kip$

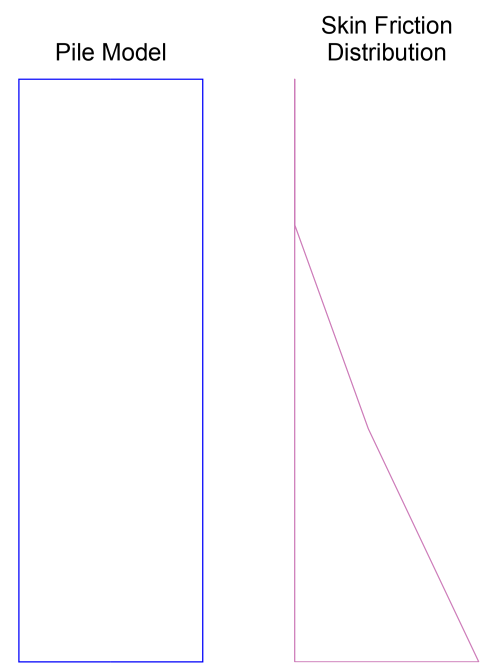
Maine DOT
Auburn 22224, HP 14x117 D 36-32, set 4

07-Jan-2019
GRLWEAP Version 2010



DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	1095 (73%)	psi
Helmet Weight	3.20	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	15.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 11 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
820.0	42.68	1.33	7.8	7.23	23.68
840.0	43.30	1.56	8.0	7.31	23.92
860.0	43.93	1.90	8.3	7.37	24.18
880.0	44.59	2.05	8.7	7.43	24.23
890.0	44.94	2.22	8.9	7.45	24.34
895.0	45.13	2.37	9.0	7.47	24.34
1000.0	48.43	1.72	11.2	7.75	25.14
1100.0	51.06	2.54	14.1	7.98	25.78

Limit bpi to less than 12.

$R_{dr\_14x117} := 840kip$

Strength Limit State

$\varphi_{dyn} := 0.65$

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \varphi_{dyn} = 546 \cdot kip$

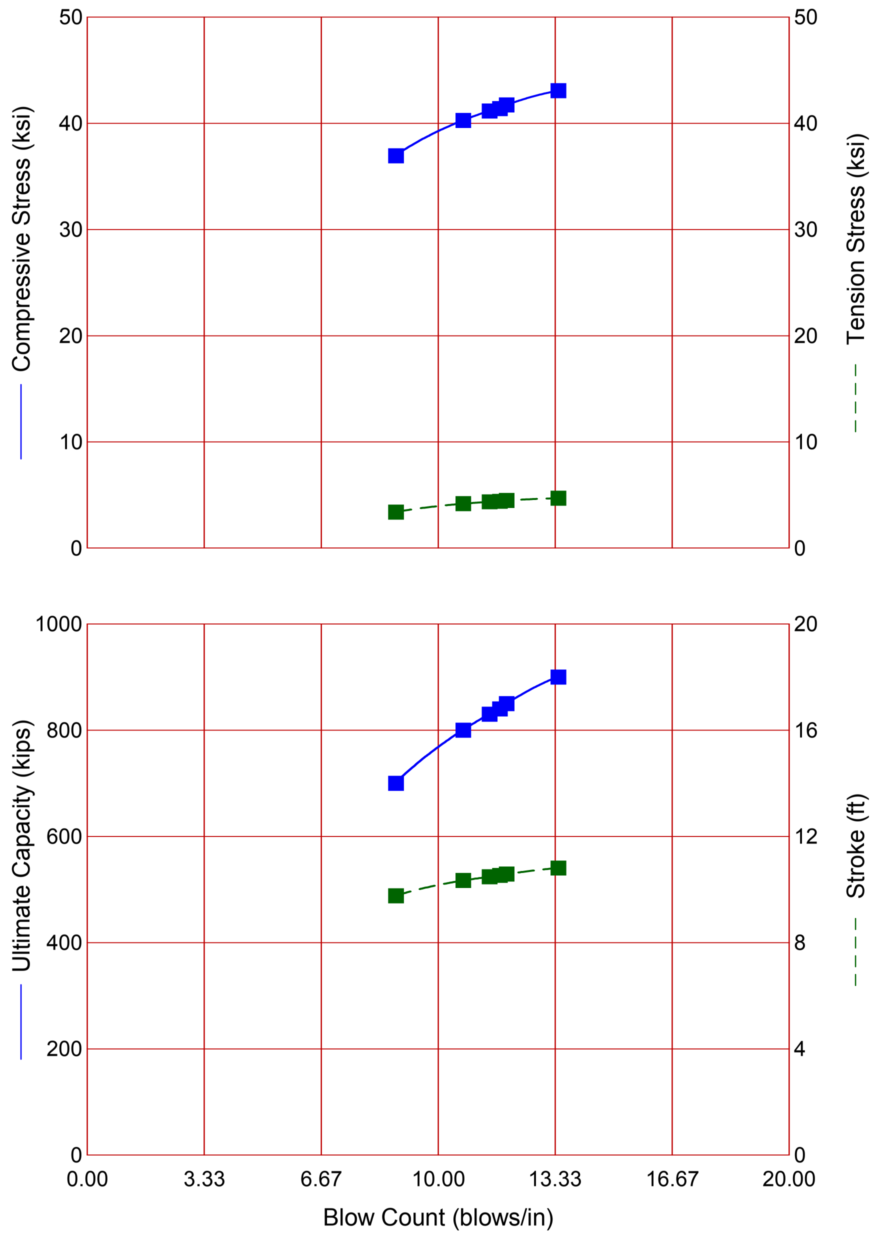
Service and Extreme Limit State

$\varphi := 1.0$

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \varphi = 840 \cdot kip$

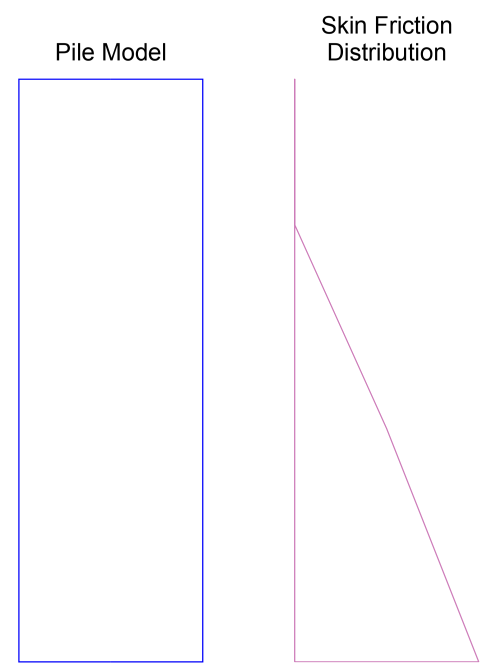
Maine DOT
Auburn 22224, HP 14x117 D 19-42, set 1

09-Jan-2019
GRLWEAP Version 2010



DELMAG D 19-42

Ram Weight	4.00	kips
Efficiency	0.800	
Pressure	1600 (100%)	psi
Helmet Weight	3.20	kips
Hammer Cushion	109975	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	20.00	ft
Pile Penetration	15.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 84.0 kips
(Constant Res. Shaft)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
700.0	36.93	3.40	8.8	9.76	17.84
800.0	40.26	4.19	10.7	10.34	18.70
830.0	41.15	4.36	11.5	10.48	18.87
840.0	41.38	4.40	11.8	10.53	18.90
850.0	41.72	4.49	11.9	10.58	19.03
900.0	43.07	4.71	13.4	10.81	19.31

Limit bpi to less than 12.

$R_{dr\_14x117} := 850kip$

Strength Limit State

$\varphi_{dyn} := 0.65$

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \varphi_{dyn} = 553 \cdot kip$

Service and Extreme Limit State

$\varphi := 1.0$

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \varphi = 850 \cdot kip$

Frost Depth

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: Auburn, Maine
Case 1 - coarse grained granular fill soils W=10%

$DFI_1 := 1400$ $d_1 := 79.2 \cdot in$

$DFI_2 := 1500$ $d_2 := 82.1in$

Approximate DFI at project = 1775 find frost depth by interpolation:

$DFI_3 := 1420$

$$d_3 := d_1 + \frac{(DFI_3 - DFI_1) \cdot (d_2 - d_1)}{(DFI_2 - DFI_1)} \qquad d_3 = 79.8 \cdot in$$

Depth of Frost Penetration $d_3 = 6.6 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils
Lewiston lies near the 1400 F-days isoline

```
------------------------
--- ModBerg Results ---
-----------------------

Project Location: Lewiston, Maine

Air Design Freezing Index      = 1224 F-days
N-Factor                       = 0.80
Surface Design Freezing Index  =  979 F-days
Mean Annual Temperature        = 46.4 deg F
Design Length of Freezing Season = 118 days

---------------------------------------------------------
Layer
#:Type      t   w%    d    Cf  Cu  Kf  Ku   L
---------------------------------------------------------
1-Coarse   63.5 10.0 125.0  28  34  2.0 1.6 1,800
---------------------------------------------------------

t  = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d  = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L  = Latent heat of fusion, in BTU / cubic ft.

*********************************************************
 Total Depth of Frost Penetration = 5.29 ft = 63.5 in.
*********************************************************
```

Recommendation: 6.6 feet for design of foundations constructed on coarse grained soils

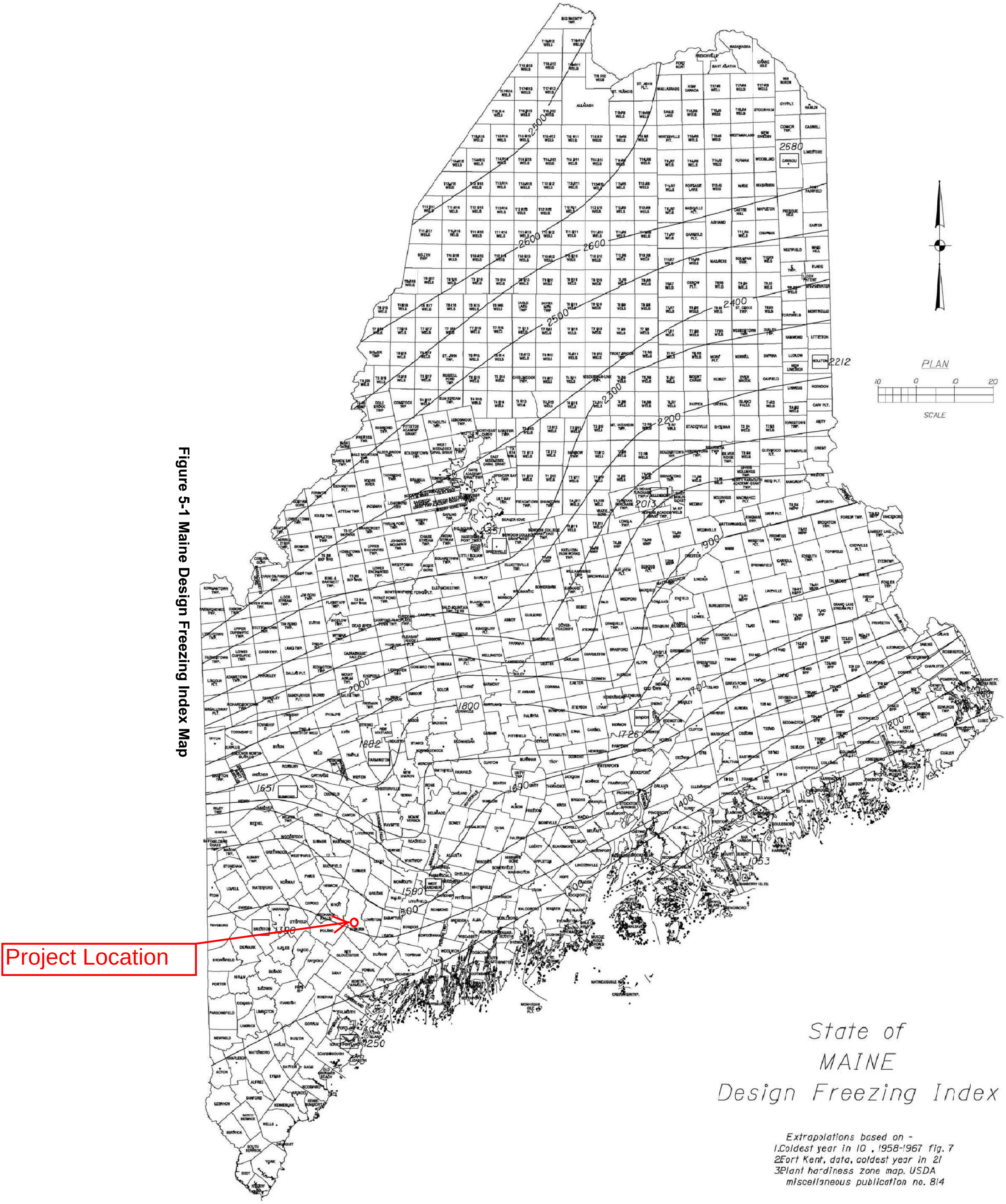


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Interpolate

Seismic Parameters

BB-ATB-101			
Depth	**101 N_{60}**	**di**	**di/N**
6	45	6	0.13
12	3	6	2.00
16	39	5.5	0.14
21	90	6	0.07
100	100	76.5	0.77
SUM		**100**	**2.97**
		di/di/N	**33.64**

BB-ATB-103			
Depth	**102 N_{60}**	**di**	**di/N**
6	31	10	0.32
11	2	5	2.50
16	14	5	0.36
21	3	5	1.67
26	29	6	0.21
100	100	69	0.69
SUM		**100**	**5.42**
		di/di/N	**18.45**

SUM	**Nav.**	**26.04**

15 < N_{av}. < 50

Conclusion: Site Class D

Site Classification per LRFD Table C3.10.3.1-1 - Method B

Auburn
WIN 22224.00
Taylor Brook Bridge
3/14/2019

Calculations by AVB
Checked by LK 3-19-2019

2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.099434
Longitude = -070.267021
Site Class B
Data are based on a 0.05 deg grid spacing.

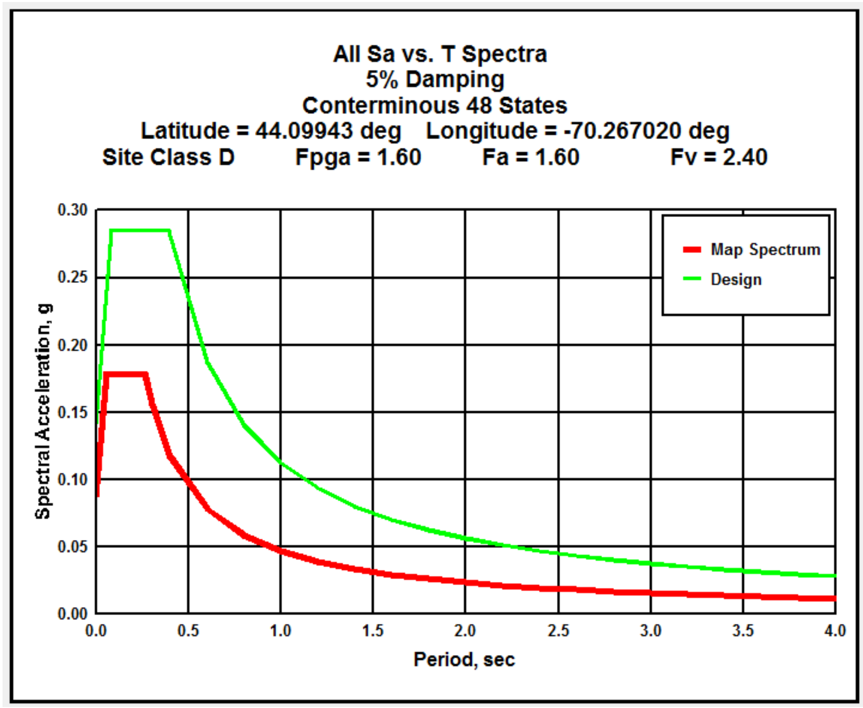
Period (sec)	Sa (g)	
0.0	0.089	PGA - Site Class B
0.2	0.178	Ss - Site Class B
1.0	0.047	S1 - Site Class B

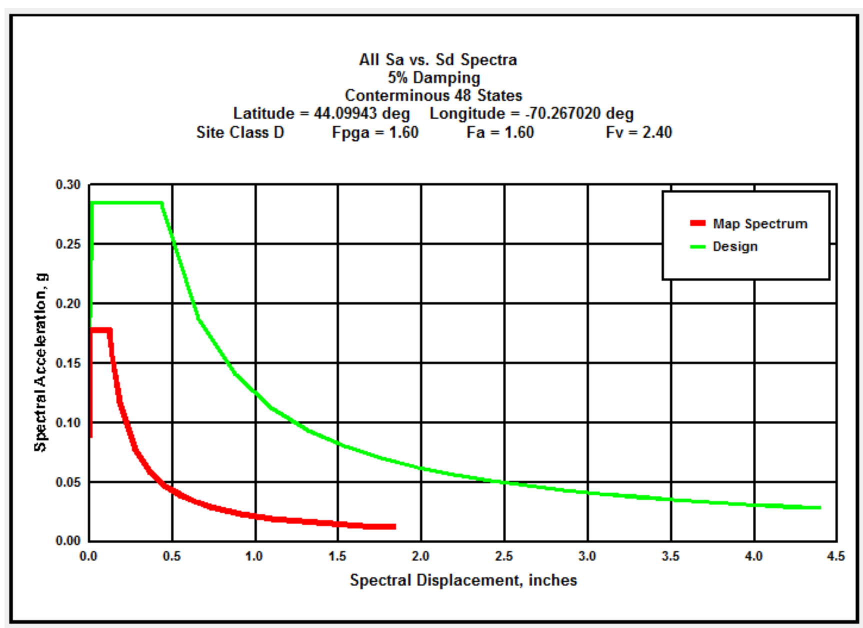
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.099434
Longitude = -070.267021
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.142	As - Site Class D
0.2	0.285	SDs - Site Class D
1.0	0.112	SD1 - Site Class D

Auburn
WIN 22224.00
Taylor Brook Bridge
3/14/2019

Calculations by AVB
Checked by LK 3-19-2019





Appendix E

Historical Soils Report

Maine Department of Transportation

Materials and Research Division

Soils Section

PRELIMINARY

SUBSURFACE INVESTIGATION FOR THE PROPOSED WIDENING OF TAYLOR BROOK BRIDGE OVER TAYLOR BROOK ON HOTEL ROAD IN THE CITY OF AUBURN

Androscoggin County   Bridge No. 3225   Soils Report 80-105

April 1980

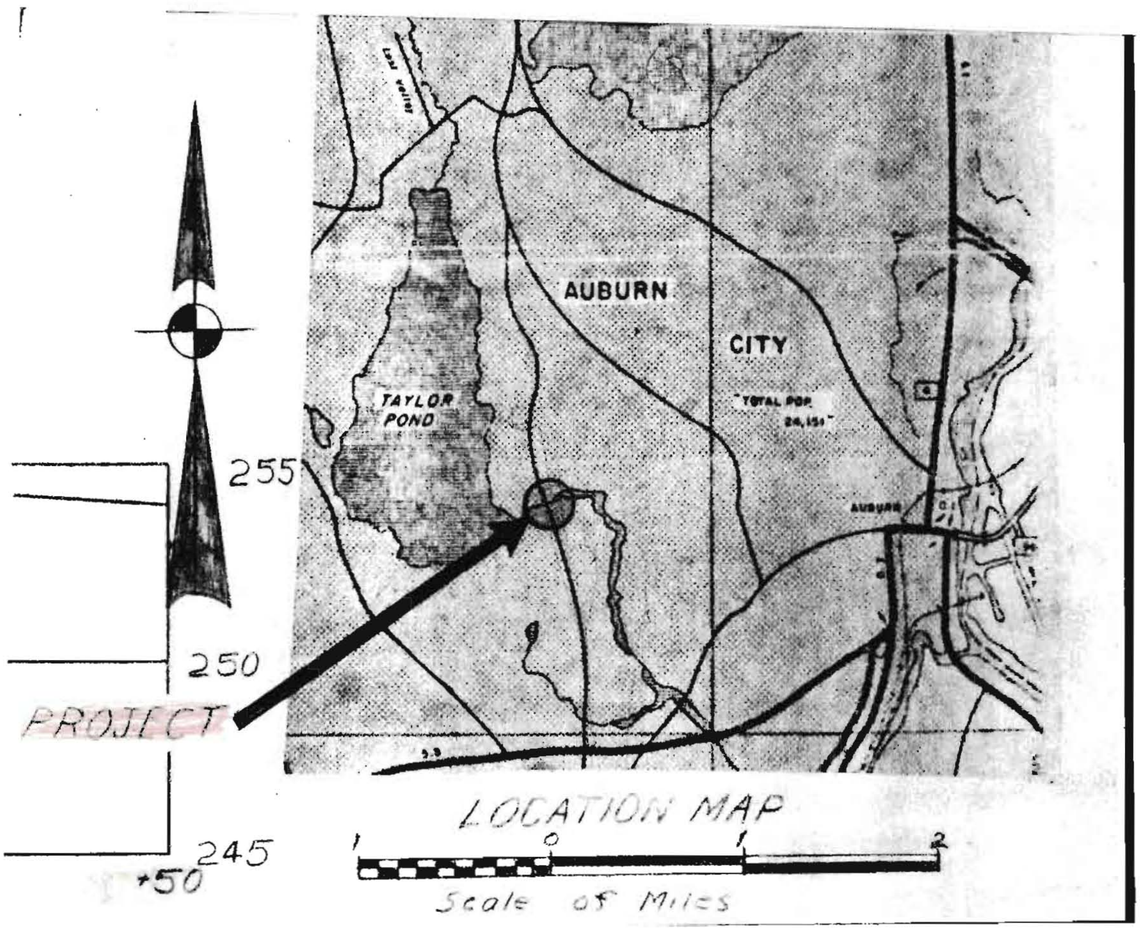

AUBURN
CITY
TAYLOR POND
"TOTAL POP. 24,151"
AUBURN
255
250
245
+50
PROJECT
LOCATION MAP
1
0
1
2
Scale of Miles

TABLE OF CONTENTS

INTRODUCTION

A preliminary subsurface soils investigation has been completed for Taylor Brook Bridge over Taylor Brook on Hotel Road in the City of Auburn. It is proposed to widen the existing structure by extending the abutments approximately twenty feet at each end for a total additional width of forty feet, or by replacing the existing structure with a culvert. It is also proposed to raise the roadway grade in the vicinity of the bridge.

Soils investigation consisted of two washborings, one approximately twenty feet beyond the right end of the existing south abutment and one approximately twenty feet beyond the left end of the existing north abutment, which were made during February 1980 by a crew under the supervision of Mr. Christian Bark. The locations of the two washborings are shown on a plan of the existing bridge on Sheet 6 of the illustrations following the text of this report. Also shown on this sheet is a transverse section depicting the soils stratification encountered by the washborings and the details of the two borings.

All soil samples were transported to the Central Laboratory in Bangor where standard identification and direct shear tests were performed. Grain size distribution curves for several samples are shown on Sheet 1. Direct shear diagrams for three samples are shown on Sheets 2 through 4.

GENERAL CONDITIONS

The proposed project is located in the City of Auburn, Androscoggin County. The existing bridge, which carries Hotel Road over Taylor Brook, is a single-span reinforced concrete structure, with a length between abutments of approximately seventeen feet. It is proposed to widen the existing structure by extending each end of the abutments approximately twenty feet, or by replacing the bridge with a culvert. It is also proposed to raise the roadway grade in the area of the bridge approximately seven feet.

Soils at this bridge site generally consist of brown sand and gravel fill over loose brown sand with rocks underlain by medium density gray silty fine to medium sand with rocks over ledge. The boring made beyond the left end of the north abutment encountered loose brown and gray fine sand and silt with rocks under the fill and a thin layer of very soft gray silty sandy clay above the gray sand layer. Ledge cores were described by a geologist as biotitic, chloritic and/or granitic gneiss.

WASHBORING DETAILS

South Abutment:

Washboring CB-11-80 was made beyond the right end of the existing south abutment at Station 71+95, thirty feet right, at a ground surface elevation of 241.31. This boring encountered three feet of brown sand and gravel fill over five feet of loose brown medium to coarse sand with rocks underlain by 11.5 feet of medium density gray silty fine to medium sand with rocks over ledge. Ledge, described as biotitic and granitic gneiss, was encountered at an elevation of 221.8. Direct shear tests were performed on Samples 2D and 3D representative of the gray sand layer and the resulting direct shear diagrams are shown on Sheets 2 and 3. Grain size distribution curves for Samples 1D and 3D are shown on Sheet 1. The detail sheet for this boring is shown on Sheet 6.

North Abutment:

Washboring CB-10-80 was made beyond the left end of the existing north abutment at Station 72+19, thirty-two feet left, at a ground surface elevation of 241.96. This boring encountered three feet of gravel fill with rocks, the top one foot of which was frozen, over six feet of loose brown and gray fine sand and silt with brick fragments and rocks underlain by 3.3 feet of very soft gray silty sandy clay over 12.2 feet of medium density gray silty fine to medium sand with rocks over ledge. Ledge described as biotitic and chloritic gneiss was encountered at Elevation 217.5. A direct shear test was made on Sample 2D representing the gray silty sand and the resulting direct shear diagram is shown on Sheet 4. Grain size distribution curves for Samples 1D and 3D are shown on Sheet 1. The detail sheet for this boring is shown on Sheet 6.

FOUNDATION DESIGN

Two alternatives are under consideration for widening the Taylor Brook Bridge. One is to extend each end of the existing abutments approximately twenty feet and the other is to replace the bridge with a culvert. In the case of widening the existing bridge, the abutment extensions could be supported on spread footings. The bottom of the extension footings would have to be located below the loose sand and soft clay layers. Additional borings should be made to define the extent of the soft clay layer. Allowable design pressures for a spread footing design within the gray sand layer are indicated on Sheet 5. The loads on the existing abutment footings would be increased due to the proposed higher roadway grade. This increased pressure of the existing footings would have to be within the limits shown on Sheet 5 in order to avoid a bearing capacity failure.

The culvert design would involve replacing the existing structure with one or more culverts or pipe arches which would be supported on the streambed. Very little information was obtained on the extent of the clay layer or the properties of the clay encountered by boring CB-10-80, but based on the available information, a settlement of approximately six inches could be expected for a culvert design.

It is also proposed to raise the roadway grade in the vicinity of the bridge approximately seven feet. No significant settlement or stability problems are anticipated for the proposed bridge approaches, but additional explorations should be made to determine the extent of the soft clay layer.

SUMMARY

Two washborings were made for the proposed project. The locations of these explorations are shown on a plan on Sheet 6. Also shown on this sheet is a transverse section depicting the soil stratification encountered by the borings and the details of the two borings.

Soils near the right end of the existing south abutment consist of brown sand and gravel fill over loose brown sand with rocks underlain by medium density gray sand with rocks over ledge. Soils near the left end of the existing north abutment consist of gravel fill over loose brown and gray fine sand and silt with rocks underlain by a thin layer of very soft gray silty sandy clay over the gray sand layer and ledge.

It is proposed to widen and raise the grade of the roadway in the area of the Taylor Brook Bridge. One proposal is to widen the existing bridge by extending each end of the existing abutments approximately twenty feet. In this case, the extensions could be supported on spread footings as long as the new footing elevations were below the loose sand and clay layers (Elevation 230±). Allowable spread footing pressures are shown on Sheet 5. The proposed higher roadway grade would result in an additional load on the existing abutment footings. The resulting bearing pressures must also be within the limits shown on Sheet 5.

A second alternative being considered is to replace the existing structure with a culvert. Approximately six inches of settlement is possible with this design.

No significant settlement or stability problems are anticipated for the bridge approaches, but additional explorations should be made to determine the extent of the clay layer encountered by washboring CB-10-80. A sewer line crosses the road in the vicintiy of Station 71+50 and parallels the roadway on the west side throughout this project site. Since clay was encountered in the boring made on this side, this sewer line is likely to experience some settlement as a result of the higher roadway grade.

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Guy L. Baker
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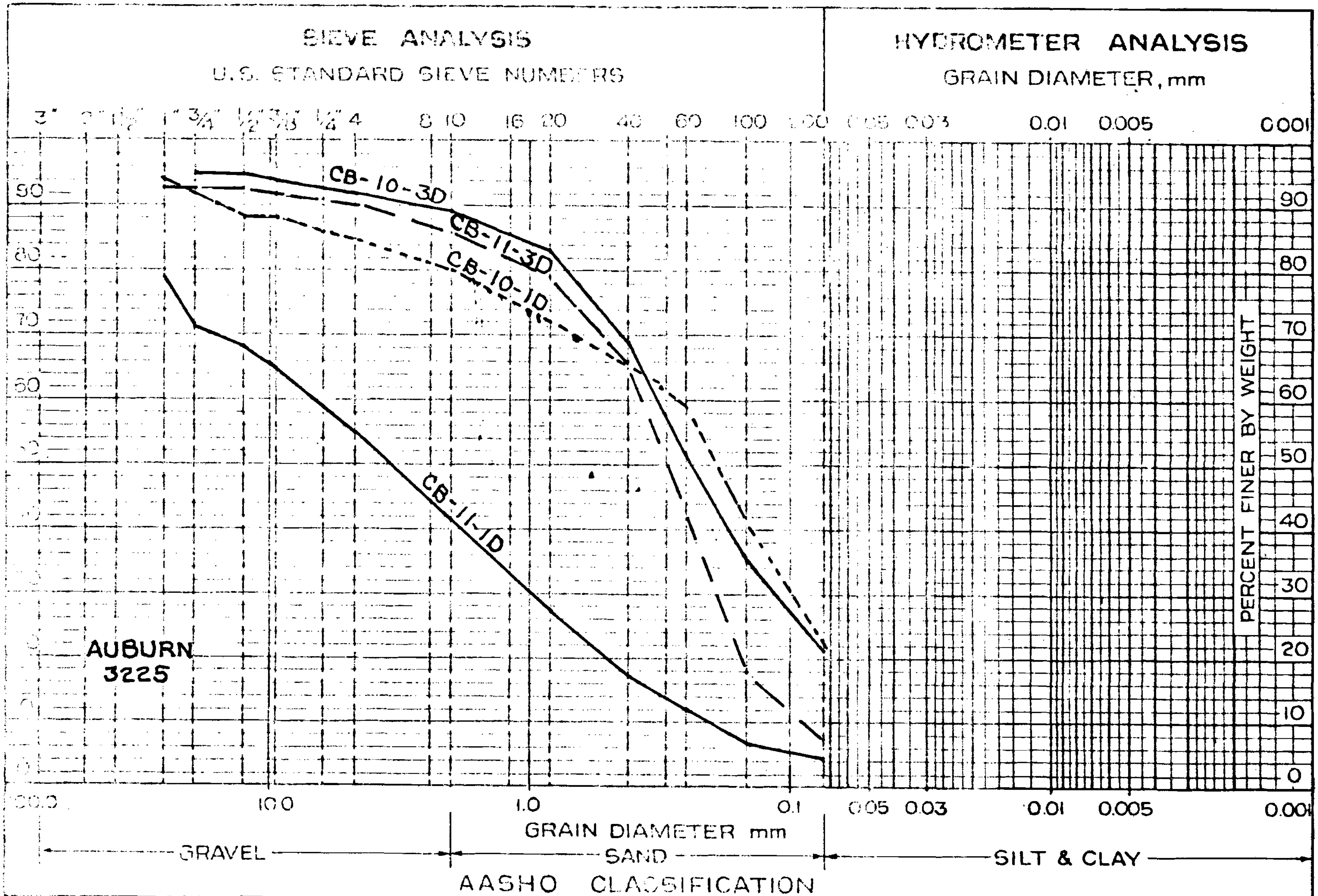
SML-10 (12/65)



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

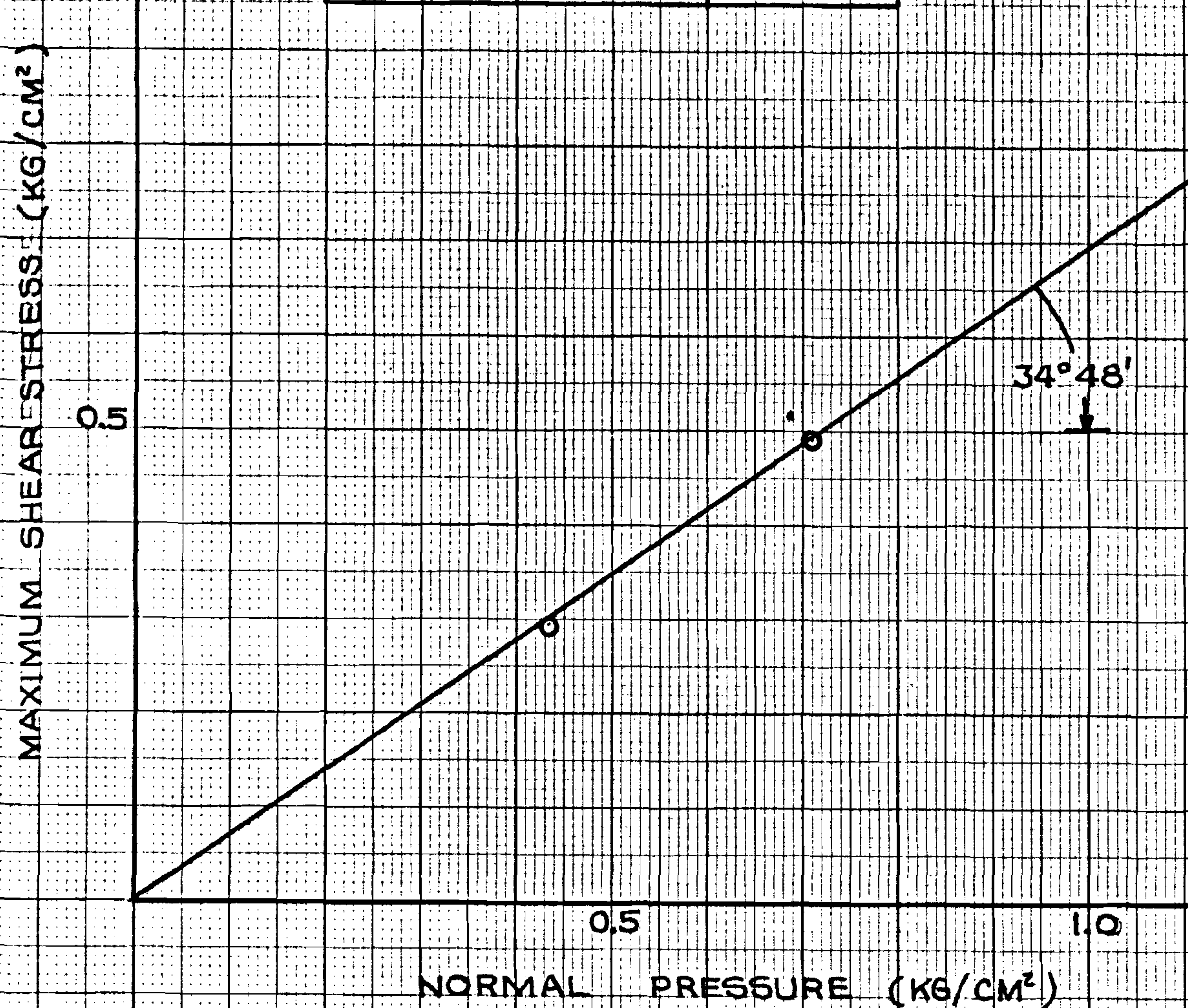
GRAIN SIZE DISTRIBUTION CURVES

BUREAU OF HIGHWAYS
MATERIALS & RESEARCH DIVISION

SHEET NO. 1

LDS-3

FRAME & APPLIED WEIGHT	NORMAL PRESSURE KG/CM²	MAXIMUM SHEAR
A 14.16	.432	.295
B 23.25	.710	.491
C 41.43	1.265	.884

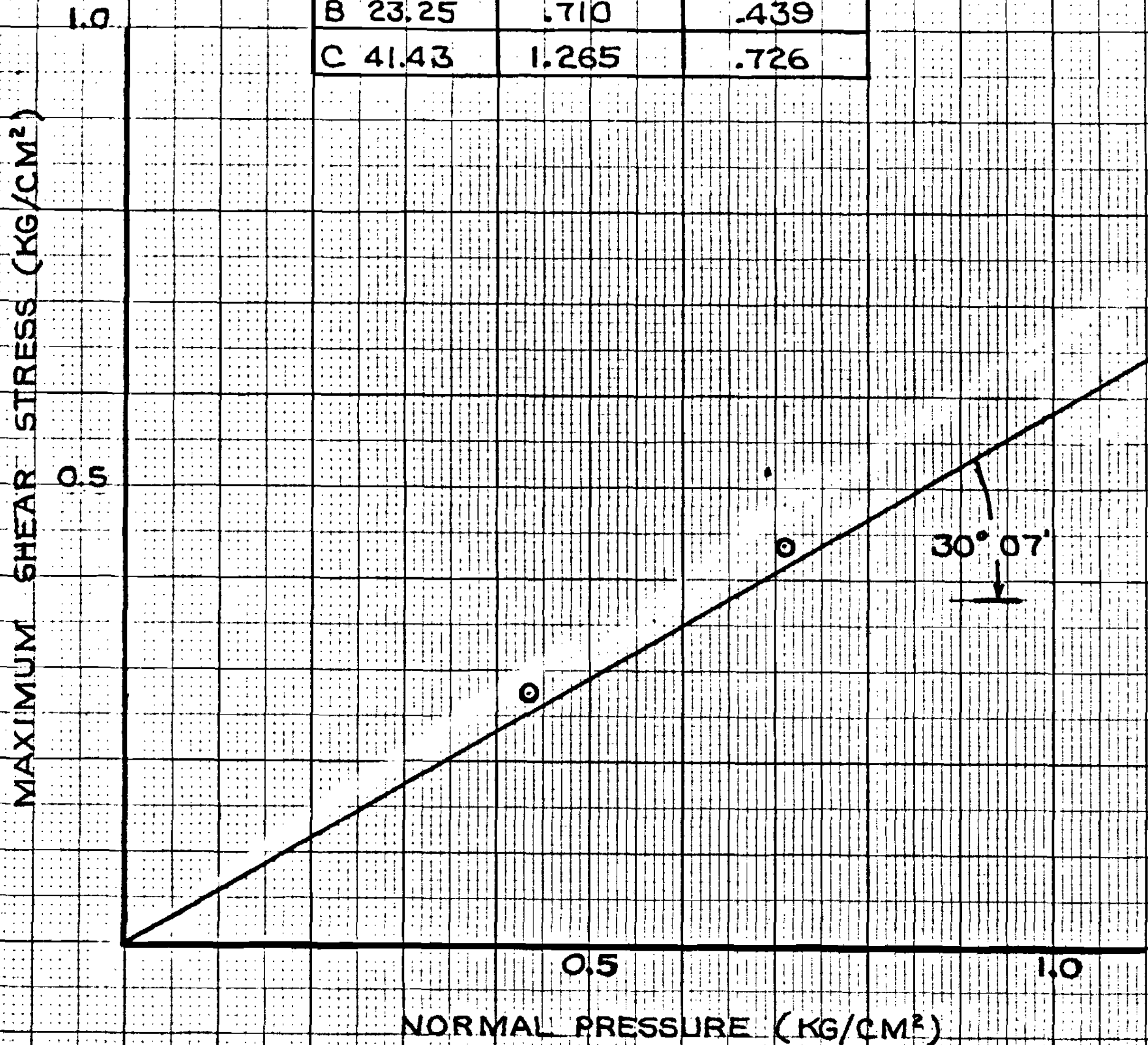


DIRECT SHEAR DIAGRAM
AUBURN
BR # 3225
BORING CB-11-80 SAMPLE 2D
MARCH 1980

SHEET NO. 2

LOS- 4

FRAME & APPLIED WEIGHT	NORMAL PRESSURE KG/CM^2	MAXIMUM SHEAR
A 14.16	.432	.277
B 23.25	.710	.439
C 41.43	1.265	.726



DIRECT SHEAR DIAGRAM

AUBURN

BR # 3225

BORING CB-11-80 SAMPLE 3D

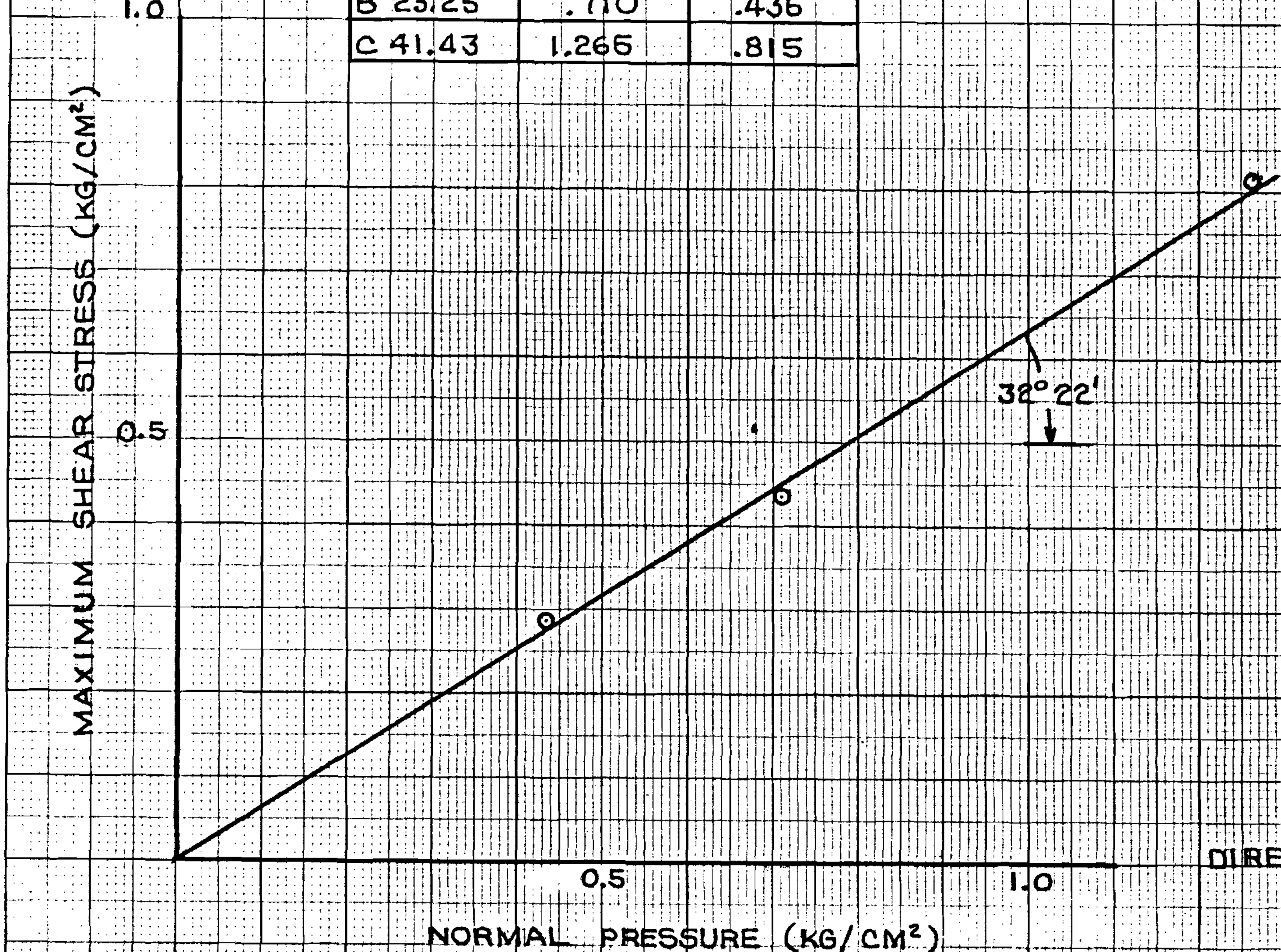
1980

SHEET NO. 3

46 1242

LDS - 2

FRAME & APPLIED WEIGHT	NORMAL PRESSURE KG/CM²	MAXIMUM SHEAR STRESS
A 14.16	.432	.288
B 23.25	.710	.436
C 41.43	1.266	.815



DIRECT SHEAR DIAGRAM

AUBURN

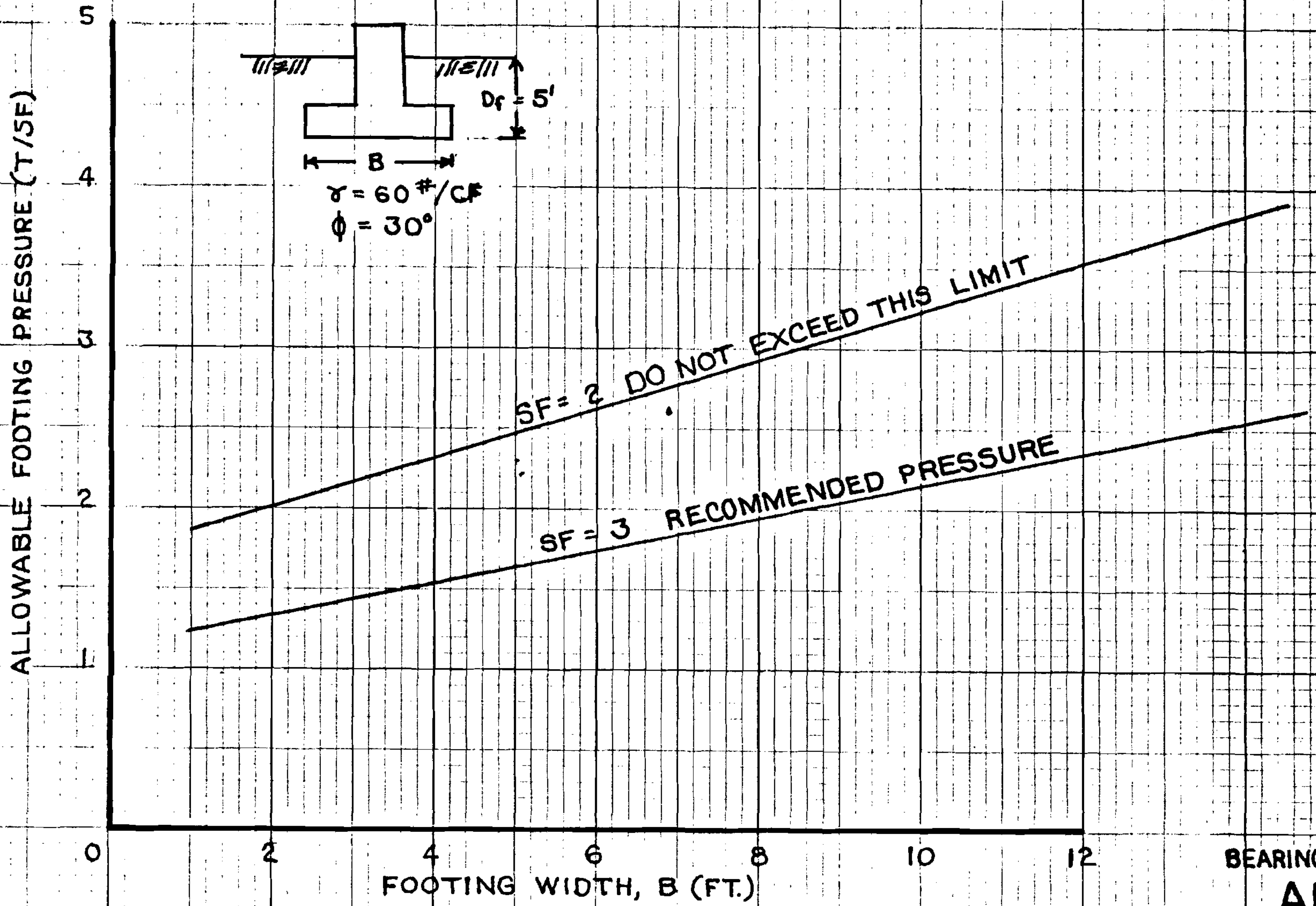
BR. # 3225

BORING CB-10-80 SAMPLE 2D

MARCH 1980

SHEET NO. 4

46 0782



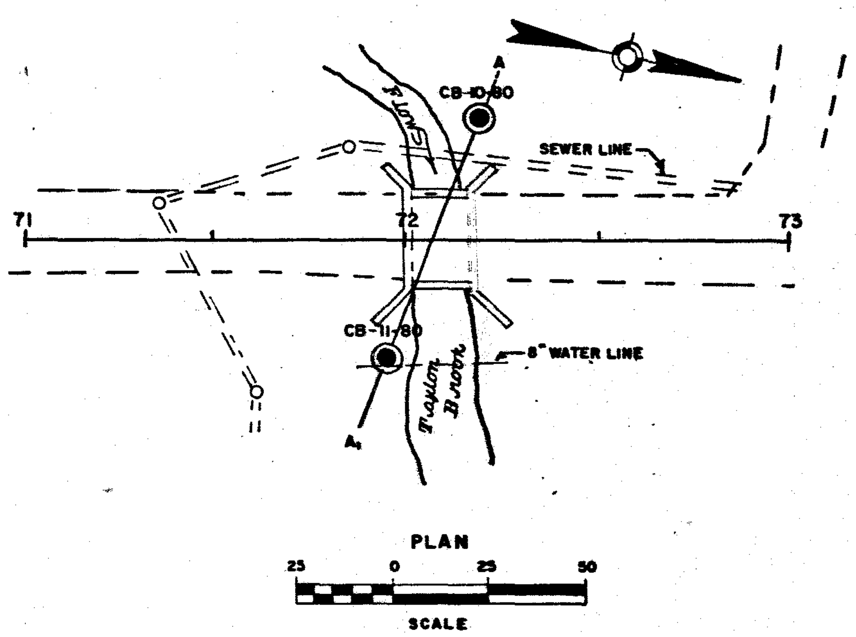
BEARING CAPACITY CHART

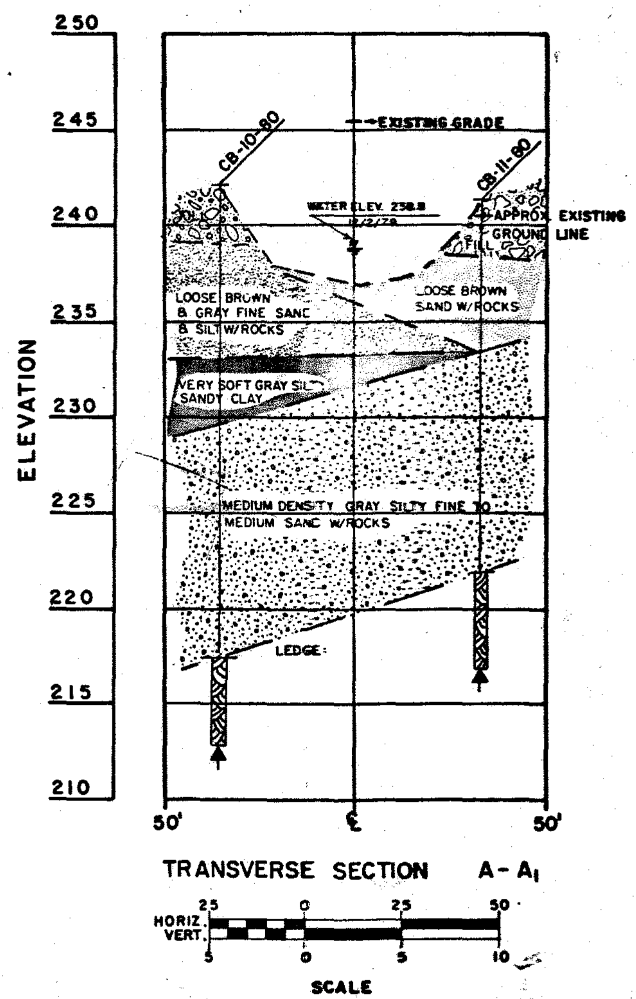
AUBURN

BR# 3225

SHEET NO. 5

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE		6	





BORING NOTES

2½" Casing used

All samples are made ahead of casing

Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow

Location of sample or sample attempt

ID S & H Sampler #1290's

IC 2" O.D. 16 ga. seamless tubing

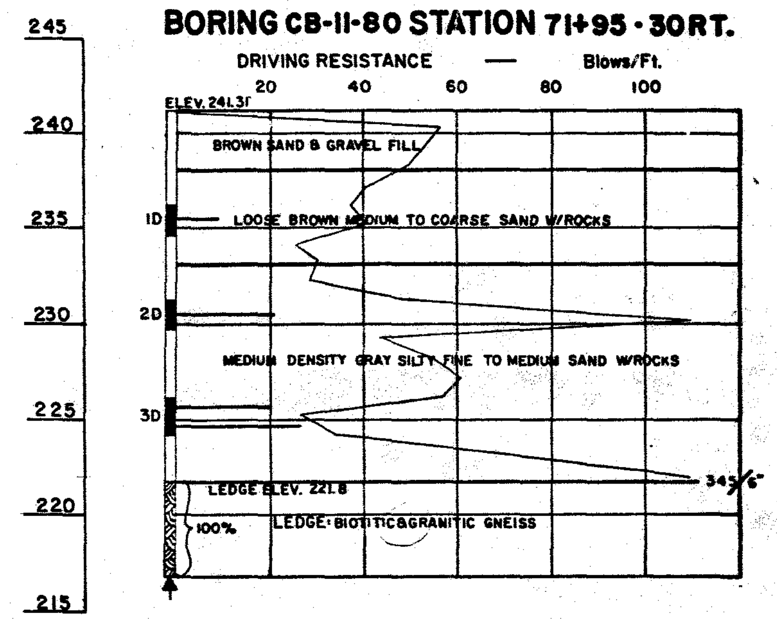
MD Unsuccessful sample attempt and type sampler

Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow

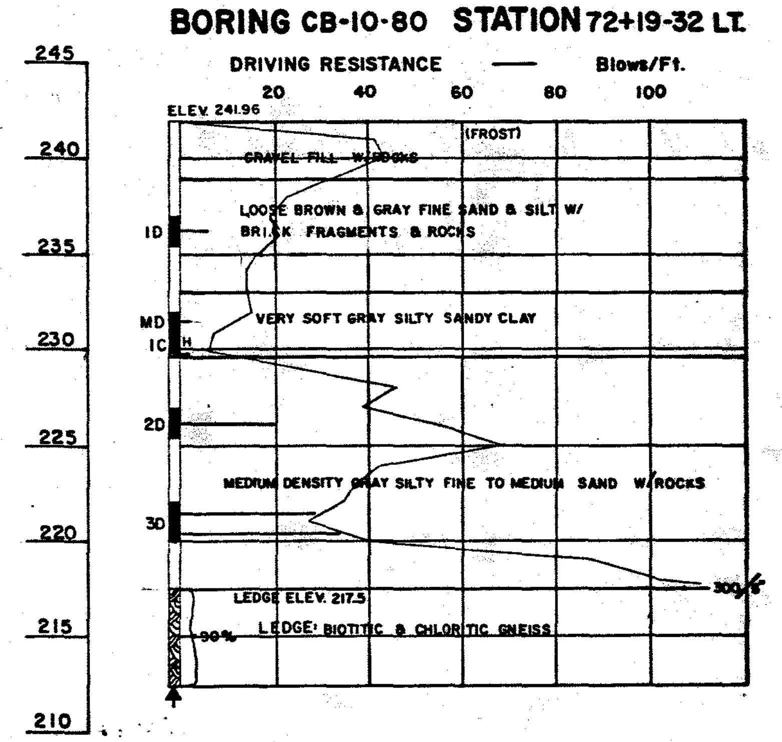
Bottom of boring (may not be bottom of soil strata)

71% Location cored by diamond bit and percent recovery of rock

H Sampling spoon or seamless tubing driven by static weight of drill rods and hammer



ELEVATION



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

TAYLOR BROOK BRIDGE

OVER

TAYLOR BROOK

IN THE CITY OF

AUBURN

ANDROSCOGGIN COUNTY

FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE