MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Rehabilitation of:

MUDDY RIVER BRIDGE PIER
FORESIDE ROAD OVER THE MUDDY RIVER
TOPSHAM, MAINE

Prepared by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Reviewed by:
Kathleen Maguire, P.E.
Geotechnical Engineer

Sagadahoc County
PIN 16756.00

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the substructure rehabilitation of Muddy River Bridge which carries Foreside Road over Muddy River located in Topsham, Maine. The bridge is a two span, painted steel beam structure and a total length of 100 feet. The pier consists of two timber bents joined with timber and steel channel cross bracing. The pier timber piles have moderate to severe deterioration, rotting and ice damage. The proposed bridge substructure rehabilitation project will consist of replacing the existing pier with a new pier and repair to the bridge approach slopes. No rehabilitation of the existing abutments will be undertaken.

Pier Foundation Alternatives - The following foundations may be considered for the replacement pier substructure:

- A mass pier supported on a spread footing constructed on bedrock
- A pier bent consisting of concrete filled pipe piles, with rock-socketed internal W or H-sections, or rock anchors, for fixity
- A pier bent consisting of steel H-piles, with rock-socketed H-sections to provide a fixed or pinned pile end
- A pier bent consisting of steel H-piles without rock sockets if computer models demonstrate pier bent stability against failure by sliding, overturning or excessive movement, for all relevant load combinations
- A pier bent consisting of two small diameter drilled shafts with rock sockets.

All of these foundation types are viable, to varying degrees, as foundation alternatives for this site, however a H-pile pier bent was determined in the Preliminary Design Report to be the most economical and practicable foundation type. This report addresses only recommendations for a pier bent consisting of steel H-piles.

Replacement Pier Bent – Strength and extreme limit state design of the pier foundation shall consider:

- compressive pile axial loads
- eccentricity (overturning failure) of the bent system
- uplift resistance of piles in tension
- failure by sliding of individual piles
- failure by sliding of the pile group
- structural failure of individual bent piles
- geotechnical failure of individual bent piles bearing on or within bedrock
- pullout failure of any rock anchors or piles installed in bedrock sockets.

Extreme event design checks are those relating to ice load, debris load, vessel collision, the check flood for scour, and certain hydraulic events.

Strength and extreme limit state checks for sliding and overturning stability of a pier bent consisting of rock-socketed H-piles should be conducted with 3-dimensional (3-D) structural frame software or soil-structure interaction engineering software assuming a pinned connection at the pile/bedrock interface. If the H-piles bear only upon bedrock, a free boundary condition shall be assumed and a frictional nodal force applied, calculated using a sliding resistance factor, φ_τ, of 0.90, and a maximum frictional coefficient of 0.31 at the steel toe-bedrock interface.

Service limit state design checks shall be used to assess pier bent settlement, transverse and longitudinal movement, overall global stability and scour at the design flood. We anticipate that a stability evaluation will be performed using structural software with a pile-to-bedrock boundary condition of free, pinned or fixed, that is appropriate with the type of construction at the pile tip (i.e. pinned for rock-anchored or rock-socketed tips and free for piles bearing on bedrock.) The computer analysis may not converge with a free boundary condition at the pile tips, in which case a resisting force may be estimated using the frictional resistance coefficients provided in this report, pile contact surface area and the nominal axial pile normal force.

Piles should be 50 ksi, Grade A572 steel H-piles. For the strength, service and extreme limit states, the calculated factored axial compressive structural, geotechnical and drivability resistances of five H-piles sections are provided in Section 7.1 of this report. The selected pile section will depend on the factored design loads.

If computer models demonstrate a pile bent is adequate with a free or “friction” condition at the pile tip, the piles should be fitted with special pile tips such as Rock Injector HP-80500 Pile Point manufactured by Associated Pile and Fitting (APF) or equivalent.

Site conditions may warrant that the nose of the pier bent be designed to effectively break up or deflect floating ice.

Scour and Riprap - For the scour protection of the pier foundation, we recommend that a fixed or pinned condition be designed at the base of the steel H-piles by extending pier bent steel beams into bedrock sockets. The adequacy of a free, pinned, fixed condition should be assessed with appropriate 3-D structural frame analysis software during final design.

Bridge rehabilitation may require repair to the approach slopes. Riprap may also need to be placed at the toes of abutments. Rehabilitated slopes should be armored with 3 feet of riprap in accordance with Section 2.3.11.3 of the MaineDOT Bridge Guide (BDG). Riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1-foot thick layer of bedding material.

Seismic Design Considerations – Seismic analysis is not required for multiple-span bridges in Seismic Zone 1, however superstructure connections and bridge seat dimensions shall be designed in accordance with LRFD requirements.

Construction Considerations - Construction activities will include pile driving and possibly bedrock drilling. There is a potential that old timber piles, cobbles, and/or boulders may obstruct pile installation operations, including constructing rock sockets and driving piles. Obstructions may be cleared by conventional excavation methods, preaugering, predrilling, or down-hole hammers.

The nature, slope and degree of fracturing in the bedrock surface will not be evident until the piles are installed. Highly sloping bedrock and loose fractured bedrock may be encountered at the location of the proposed pier bent and may not be suitable for end-bearing piles. If during pile driving pile behavior indicates that any driven pile is not seated in bedrock, the pile shall be re-driven.

Due to the presence of sloping bedrock and thin overburden at the site, pile ‘walking’ during driving is possible, and the contractor will be responsible for removing and re-driving pile that are out of position. The Contractor shall be required to support all piles laterally in their final positions until the connection of the pier bent cap to the superstructure is complete and in place.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the pier rehabilitation of Muddy River Bridge which carries Foreside Road over Muddy River located in Topsham, Maine. This report presents the soils information obtained at the site during the subsurface investigation, foundation recommendations and geotechnical design parameters for pier rehabilitation.

Muddy River Bridge was built in 1952 and is a two span, painted steel beam structure with two 50-foot spans. The north abutment is a concrete stub abutment bearing on shallow bedrock. The south abutment consists of a transverse timber sill supported on five timber crib members oriented at 90° to the sill, each of which is founded on two battered timber piles, for a total of 10 piles. The south timber abutment is undermined and the piles show moderate deterioration. The center pier bent consists of two rows of timber piles joined with timber and steel channel cross bracing. Originally the bent consisted of two rows of seven piles, with the two upstream and downstream piles battered. The two upstream battered piles are now missing. The remaining pier timber piles have moderate to severe deterioration, splitting, rotting and ice damage.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports from 2010 assign the substructures a condition rating of 4 – poor, and indicate a Bridge Sufficiency Rating of 39.1. The bridge pier has been assessed to be in poor condition and in need of complete replacement.

The MaineDOT Bridge Program’s objective is to rehabilitate the bridge for a 10-year target life. The proposed bridge substructure rehabilitation project will consist of replacing the existing pier with a new pier. No rehabilitation of the abutments is proposed as a part of this project. Some repair to the bridge approach slopes may be conducted. The project will require jacking up the bridge at the pier, removing the existing timber pier bent and constructing the replacement pier. The bridge will be closed to traffic during rehabilitation activities.

2.0 GEOLOGIC SETTING

Muddy River Bridge carries Foreside Road over the Muddy River in Topsham, Maine, as shown on Sheet 1 - Location Map, presented at the end of this report.

The Maine Geologic Survey (MGS) Surficial Geology of Brunswick Quadrangle, Maine, Open-file No. 01-484 (2001) indicates that the surficial soil unit at the north bank of the Muddy River is massive to laminated silty clay of the Presumpscot Formation. The silty clay unit typically overlies bedrock and till, and is interbedded with and overlies end moraines and marine fan deposits. The south fill extension to the Muddy River Bridge is mapped as artificial fill, bounded by freshwater wetlands comprised as muck, peat, silt and sand.

The Bedrock Geologic Map of Maine, MGS, (1985), cites the bedrock at the bridge site as the Cape Elizabeth Formation with contacts to the Cushing Formation. The Cape Elizabeth Formation consists of quartz-plagioclase-biotite schist. The Bedrock Geology of the Bath 1:100,000 Quadrangle, Maine, Geologic Map No. 02-152, MGS (2002), cites the bedrock at the site as Nehumkeag Pond Formation of the Falmouth-Brunswick Sequence which is described as light to medium gray plagioclase-quartz-biotite granofels and gneiss. Bedrock cores obtained during the project subsurface investigation consist of quartz-feldspar-biotite, banded gneiss.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two test borings, both of which were advanced to bedrock and were terminated with bedrock cores. Test borings BB-TMR-101 and BB-TMR-102 were drilled on either side of the existing pier. It has not been determined on which side of the existing pier the proposed replacement pier will be located. The borings were drilled on October 12, 2010 using the MaineDOT drill rig. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report.

The borings were drilled using cased wash boring techniques. Soil samples were typically obtained at 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 a Central Mine Equipment (CME) automatic hammer. The hammer was calibrated by MaineDOT in March of 2010 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. The N-values presented for borings drilled with the MaineDOT hammer are corrected values computed by applying average energy transfer factors of 0.84 to the raw field N-values. The hammer efficiency factor of 0.84 and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the borings using an NQ-2-inch core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Subsurface Inspector certified by the New England Transportation Technician Certification Program (NETTCP) logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling programs.

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, found at the end of this report.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site.

Laboratory testing consisted of one standard grain size analyses, two grain size analyses with hydrometer, and three natural water content tests. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of soil laboratory tests are included as Appendix B – Laboratory Test Results. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs, on Sheet 3- Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the two test borings generally consisted of river bottom sediments and stream alluvium, underlain by metamorphic bedrock. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A brief summary description of the strata encountered follows:

5.1 River Bottom Sediments

A layer of peat and muck was encountered in BB-TMR-102. The encountered deposit is approximately 4 feet thick. The river bottom soils generally consisted of dark brown muck and wood.

Corrected SPT N-value in the muck was 3 blows per foot (bpf) indicating that the river bottom sediments are soft in consistency.

5.2 Stream Alluvium

A shallow layer of alluvial soils were encountered in borings BB-TMR-101 and BB-TMR-102. The encountered thickness ranged from approximately 5.0 to 6.9 feet thick at the boring locations. The stream alluvium consisted of grey, wet, gravelly, fine to coarse sand, little silt and grey, wet, fine to coarse sand, some silt, little to some gravel, trace clay.

Corrected SPT N-values in alluvium ranged from 4 to >50 blows per foot (bpf) indicating that the stream alluvium is loose to very dense in consistency. The SPT interval with the N-value >50 bpf is assumed to have been influenced by a cobble in the alluvial deposit.

Three grain size analysis resulted in the alluvium being classified as A-1-b and A-2-4 under the AASHTO Soil Classification System and SM and SC-SM under the Unified Soil Classification System (USCS). The measured water contents of the samples tested ranged from approximately 5 to 18 percent.

5.3 Bedrock

Below the streambed, bedrock was encountered and cored at depths ranging from approximately 5 feet below ground surface (bgs) and approximate Elevation -15.70 feet in boring BB-TMR-101 to a depth of approximately 10.9 feet bgs and approximate Elevation -22.20 feet in boring BB-TMR-102.

Based on field observations, and information inferred from State Highway Commission Bridge Division Plans from 1941 and 1952, bedrock surface at the site slopes downward from north to south. Bedrock is close to the ground surface on the north bank of the Muddy River, and at least 20 feet below the streambed at the toe of the South Abutment (Abutment 1).

The bedrock at the site is identified as grey and white, medium grained, Quartz, Feldspar, Biotite, banded Gneiss, hard to moderately hard, moderately weathered to fresh, joints along banding at chaotic to low angles, surfaces fresh with no infilling, to stained and oxidized. The RQD of the bedrock was determined to range from 33 to 80 percent, correlating to a Rock Mass Quality of “poor” to “good”.

5.4 Groundwater

The water levels observed in two borings were consistent with the tidal water elevations in the Muddy River.

6.0 SUBSTRUCTURE REHABILITATION ALTERNATIVES

The proposed bridge substructure rehabilitation project will consist of replacing the existing timber pier with a new pier. No rehabilitation of the abutments is proposed as a part of this project. Some repair to the bridge approach slopes may be conducted.

The following foundations may be considered for the replacement pier substructure:

- A mass pier supported on a spread footing constructed on bedrock
- A pier bent consisting of concrete filled pipe piles, with rock-socketed internal W- or H-sections, to provide fixity or a pinned connection at the ends of each pile
- A pier bent consisting of concrete filled pipe piles with internal rock anchors to provide fixity or a pinned connection at the end of each pile
- A pier bent consisting of steel H-piles, with the lower portion of the H-section installed in bedrock sockets for a fixed or pinned end condition
- A pier bent consisting of steel H-piles without the pile tips socketed in bedrock if computer models show pier bent stability against failure by sliding, overturning or excessive transverse and longitudinal movement
- A pier bent consisting of two small diameter drilled shafts with rock sockets.

All of these foundation alternatives are viable with varying degrees of risk for this site, however, an H-pile pier bent was identified in the Preliminary Design Report as the most economical and practicable foundation type. Therefore, this report addresses only recommendations for a replacement pier bent consisting of steel H-piles.

Design recommendations for the steel H-pile pier bent alternative are discussed in detail in Section 7.0 - Geotechnical Design Recommendations.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Design of H-Pile Pier Bent

The H-pile pier bent system may consist of:

- A pier bent consisting of steel H-piles, with the lower H-pile section installed in a bedrock socket to provide a fixed or pinned condition at the pile tip,
- A pier bent consisting of steel H-piles without rock sockets if 3D frame computer models demonstrate stability against failure by sliding, overturning and excessive transverse and longitudinal movement with a free or frictional end condition at the pile tip.

The new pier foundation shall be proportioned for all applicable load combinations specified in AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 5th Edition, 2010, Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, extreme and service limit states. The design of pier bents bearing on or within the bedrock at the ***strength limit state*** and ***extreme limit state*** shall consider:

- compressive axial resistance of individual piles
- eccentricity (overturning failure) of the bent system
- uplift resistance of piles in tension
- failure by sliding of individual piles
- failure by sliding at the base of the pile bent
- structural failure of individual bent piles
- geotechnical failure of individual piles bearing on or within bedrock
- pullout/uplift failure of any rock anchors or piles installed in bedrock.

Strength and extreme event limit state design of the new pier should consider foundation capacity and demand after scour due to the design and check floods for scour, respectively. For all practical purposes, the scour checks should demonstrate that lateral stability is adequate when the streambed soils are removed in their entirety.

A modified Strength Limit State analysis should be performed that includes the ice pressures specified in BDG Section 3.9 – Ice Loads.

Extreme event load combinations are those related to ice loads, vessel collision and certain hydraulic events. Resistance factors, φ, for the extreme event limit state shall be taken as 1.0. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For the ***service limit state***, a resistance factor (φ) of 1.0 shall be used to assess pier bent design for:

- settlement of individual piles
- excessive longitudinal movement of the bent
- excessive transverse movement of the bent
- bearing resistance of individual piles
- global stability of the overall bent

For sliding analyses of a bent with individual piles bearing on bedrock, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of individual piles founded on or within bedrock. Estimates of sliding resistance of the pier pile tips to lateral loads may assume a maximum frictional coefficient of 0.31 at the interface of steel piles-to-level bedrock.

Anchorage of the pier bent H-piles to bedrock by socketing the piles in bedrock may be required to resist sliding forces and provide stability.

Site conditions may warrant that the nose of the pier bent be designed to effectively break up or deflect floating ice or debris. Strengthening the nose of the bent with additional steel crossbracing may be considered.

7.1.1 Pile Material and Lengths

Bedrock was encountered below the steam bottom of the Muddy River at depths of approximately 5 to 11 feet. Based on field observations, and information inferred from State Highway Commission Bridge Division Plans from 1941 and 1952, bedrock slopes steeply from close to the ground surface on the north bank of the Muddy River, to more than 20 feet below the streambed at the toe of the South Abutment (Abutment 1).

Estimated pier bent pile lengths are summarized in the Table 7-1 below.

Pier Location Alternative	Approximate Bedrock Elevation (feet)	Estimated Pile Cap Bottom Elevation (feet)	Estimated Pile Lengths w/out rock sockets (feet)	Estimated Pile Lengths with 2- foot rock sockets (feet)
Pier Bent Location Alternate 1 Sta. 16+60	-15.70	9.0	25	27
Pier Bent Location Alternate 2 Sta. 16+75	-22.20	8.0	31	33

Table 7-1. Estimated Pile Lengths

To accommodate a pile bent pier at Muddy River Bridge, the following design features are recommended:

- The selected pile section will depend on the factored design loads. Piles should be 50 ksi, Grade A572 steel H-piles.

- In consideration of (a) the consequences of scour and ice loading, (b) the need to limit pile tip movement, and (c) providing adequate global stability of the bent, a minimum pile bedrock socket of 2 feet is recommended. The piles should be fitted with driving points to protect the tips, improve penetration and improve friction at the pile tip.

- Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. The analysis shall assign a free, pinned or fixed condition at the pile tip that is consistent with the proposed pile tip condition. As the proposed piles will have significant unsupported lengths and will not achieve fixity if not socketed, the resistance for the piles should be determined for compliance with the interaction equation and checked for buckling.

- In accordance with LRFD 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 10.7.3.9. A 3-D structural frame analysis and soil-structure analysis is recommended to evaluate the pile-bedrock interaction for combined axial and lateral loading. Assumptions regarding a fixed, pinned or free condition at the pile tip should be also confirmed with soil-structure interaction analyses.

- If 3-D structural frame analyses indicate the H-piles do not need to be installed in bedrock sockets, the piles should be driven to the required resistance on bedrock. The piles should be fitted with Rock Injector HP-80500 Pile Points, manufactured by Associated Pile and Fitting, LLC, to protect the tips, improve penetration, and improve friction at the pile tips.

7.1.2 Strength Limit State Design

Geotechnical Resistance. The nominal and factored axial geotechnical resistance in the strength limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ_{stat}, of 0.45. The calculated factored geotechnical resistances of five H-pile sections and three pile tip conditions are provided in Table 7-2, below.

	Factored Axial Geotechnical Pile Resistance		
	Pile end-bearing on bedrock φ_{stat} = 0.45 (kips)	Pile with 1-foot bedrock socket φ_{stat} = 0.45 (kips)	Pile with 2-foot bedrock socket φ_{stat} = 0.45 (kips)
HP 12 x 53	95	132	170
HP 12 x 74	133	186	239
HP 14 x 73	128	180	231
HP 14 x 89	156	219	282
HP 14 x 117	206	288	371

Table 7-2 Factored Axial Geotechnical Resistances for H-Pile Sections with three Pile Tip Conditions for Strength Limit State Design

Structural Resistance. The nominal compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD 6.9.4.1. It is the responsibility of the structural designer to calculate the nominal and factored pile structural compressive resistance (P_n) based on the “actual unbraced pile length (ℓ) and effective length factor (K)” or “on the actual elastic critical buckling resistance, P_e.”

Preliminary estimates of the factored structural axial compressive resistance of five H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (for good driving conditions) an unbraced length (ℓ) of 15 feet (which assumes one level of crossbracing), and an effective length factor (K) of 2.0.

Drivability Resistance. As the piles may be driven to refusal on bedrock a drivability analysis was performed to determine the 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 resistance factor for a single pile in axial compression when a dynamic test is performed given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn} = 0.65.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of five H-piles sections for the strength limit state is provided in Table 7-3 below, assuming no bedrock socket. If the H-pile section is socketed into bedrock, Table 7-2 can be used for an appropriate geotechnical resistance. Supporting calculations are provided in Appendix C – Calculations.

	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance ϕ_c=0.60[1] (kips)	Geotechnical Resistance, rock socket = 0.0 feet φ_{stat} = 0.45 (kips)	Drivability Resistance φ_{stat} = 0.45 (kips)	Governing Pile Axial Resistance (kips)
HP 12 x 53	147	95	241	95
HP 12 x 74	216	133	362	133
HP 14 x 73	295	128	354	128
HP 14 x 89	366	156	435	156
HP 14 x 117	495	206	390	206

Table 7-3. Factored Axial Pile Resistances for H-Pile Sections for Strength Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, for these site conditions the factored axial geotechnical resistance is less than the factored axial structural resistance and the estimated factored resistance from the drivability analyses. Therefore, the recommended governing resistance for pile design should be the factored geotechnical resistance in Table 7-3 if the piles are end bearing on rock. If the H-piles are installed in 1.0-foot deep bedrock sockets, the recommended governing resistance for pile design is geotechnical resistance provided in Table 7-2. If the H-piles are installed in 2.0-foot deep rock sockets, the governing resistance for the pile design is the structural resistance for 12x53 and 12x74 sections, and the geotechnical resistances for 14x73, 14x89 and 14x117 sections provided in Table 7-2. The maximum applied factored axial pile load should not exceed the governing factored pile resistance.

The unbraced portion of the piles should be checked for resistance against combined axial load and flexure, per LRFD Article 6.15. This axial load may govern the design. Per LRFD 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural

[1] Calculated using a resistance factor, ϕ_c, for good driving conditions, an unbraced length (ℓ) of 15 feet (which assumes one level of crossbracing), and a K of 2.0. The piling may not achieve fixity, therefore the factored structural resistance may be controlled by combined the axial and flexural resistance of the pile.

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). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2.

7.1.3 Service and Extreme Limit State Design

The design of piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall global stability of the pile pier bent and pile pier bent movements/stability considering changes in soil conditions after scour due to the design flood event.

Extreme limit state design checks for the pier bent shall include pile bearing resistance, failure of the bent by overturning (eccentricity), failure of the bent or individual piles by sliding, pile failure by uplift and structural failure. The extreme event load combinations are those related to ice loads, debris loads, vessel collision, the check flood for scour and certain hydraulic events. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For the service and extreme limit states, resistance factors, φ, of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.1 and 10.5.5.3.

The nominal and factored axial geotechnical resistance in the service and extreme limit state was calculated using the Canadian Geotechnical Society method and a resistance factor, φ, of 1.0. The calculated factored geotechnical resistances of five H-pile sections and three pile tip conditions are provided in Table 7-4, below.

	Extreme and **Service** Limit State Axial Geotechnical Pile Resistance		
	Pile end-bearing on bedrock $\varphi = 1.0$ (kips)	Pile with 1-foot bedrock socket $\varphi = 1.0$ (kips)	Pile with 2-foot bedrock socket $\varphi = 1.0$ (kips)
HP 12 x 53	210	294	378
HP 12 x 74	295	413	532
HP 14 x 73	285	399	513
HP 14 x 89	348	487	626
HP 14 x 117	458	641	824

Table 7-4 Axial Geotechnical Resistances for H-Pile Sections with 3 Pile Tip Conditions for Extreme and Service Limit State Design

The calculated factored axial structural, geotechnical and drivability resistances of five H-pile sections were calculated for the service and extreme limit states and are provided below in Table 7-5, assuming no bedrock socket. If the H-pile section is socketed into bedrock, Table 7-4 can be used for an appropriate geotechnical resistance. Supporting documentation is provided in Appendix C – Calculations.

	Service and Extreme Limit State Factored Axial Pile Resistance			
	Structural Resistance [2] ϕ=1.0 (kips)	Geotechnical Resistance, bedrock socket = 0.0 feet, φ= 1.0 (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Pile Resistance φ = 1.0 (kips)
HP 12 x 53	246	210	371	210
HP 12 x 74	360	295	557	295
HP 14 x 73	491	285	544	285
HP 14 x 89	610	348	670	348
HP 14 x 117	825	458	600	458

Table 7-5 Factored Axial Pile Resistance for H-Piles for Service and Extreme Limit State Design

LRFD Article 10.7.3.2.2 states that the factored axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance. However, at this site the factored geotechnical pile resistance of piles without the lower section installed in a bedrock socket is less than the factored axial structural resistance and the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the governing resistance used in service/extreme limit state design be the factored geotechnical resistance in the Table 7-5 if the H-piles are not installed in bedrock sockets.

If the H-piles are installed in 1 or 2-foot deep bedrock sockets, the governing pile axial resistances for pile design for the service and extreme limit states will be the lesser of the geotechnical resistances provided in Table 7-4, and the structural and drivability resistances provided in Table 7-5.

7.1.4 Driven Pile Resistance and Pile Quality Control

The contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with signal matching at the proposed replacement pier bent. (The pile test requirement may be waived if the piles are installed in bedrock sockets, but a wave equation is still recommended for selection of a properly sized hammer that will not

[2] Calculated using a resistance factor of ϕ_c=1.0, an unbraced length (ℓ) of 15 feet (which assumes one level of crossbracing), and a K of 2.0. Short pile may not achieve fixity, therefore the factored structural resistance will be controlled by combined the axial and flexural resistance of the pile.

damage the piles.) The first pile driven should be dynamically tested to confirm capacity and verify the stopping criteria developed by the contractor in the wave equation analysis. Restrikes will be not be required as part of the pile field quality control program unless pile behavior indicates the pile is not seated firmly on bedrock or if piles "walk" out of position.

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 Resident. Driving stresses in the pile determined in the drivability analysis shall be less than $0.90\phi_{da}\ F_y$ or 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 5 to 10 blows per inch (bpi), which is the optimal range for diesel hammers. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.1.5 H-Pile Unsupported Length – Design Considerations

For all practical purposes, design analyses should demonstrate that lateral stability is adequate when the streambed soils are removed in their entirety. Stability of the piles comprising the proposed pier bent shall be evaluated in accordance with the provisions of LRFD Article 6.9 using an equivalent length of the pile that accounts for the laterally unsupported length of the exposed pile extending through the air, water and the river bottom sediments and alluvium.

The design scour depth was estimated to be equal to the depth of the river bottom sediments and alluvial soils encountered in the borings; therefore to satisfy requirements for fixity, the design of the H-pile pier bent systems may need to specify that the lower H-pile section be installed in a bedrock socket to provide a fixed condition at the pile tip. A pier bent consisting of steel H-piles without rock sockets may be feasible if computer models, such as 3D structural frame analysis and soil-structure interaction programs, indicate pier bent stability against failure by sliding or overturning or excessive transverse and longitudinal movement.

7.1.6 H-Pile Pier Bent Construction

If 3-D structural frame or soil-structure models indicate the pile bent pier is not stable with free or "friction" conditions at the pile tip (i.e. the piles are seated upon bedrock), the H-piles sections should be placed installed in minimum 2-foot deep rock sockets. The H-piles should be fixed in the bedrock sockets with concrete and supported laterally in their final positions until the connection of the pier bent cap to the superstructure is complete and in place.

Bedrock sockets may be drilled using rotary percussive methods, rotary duplex methods with down-the-hole hammers, or solid coring methods. The rock socket should have a diameter of at least 2 inches greater than the diagonal H-pile section dimension. The rock socket shall be constructed so as to have a planar bottom. Once the rock socket is drilled, cleaned and the HP section installed, the H-pile should be driven to the required nominal resistance, or practical refusal, and the rock socket filled with filled with Class A concrete. If there is indication that the piles are not seated well on bedrock, the piles shall be re-driven to the stopping criteria established by the wave equation analysis or practical refusal. To protect the pile tips and improve penetration, contract documents may specify conventional or special pile tips such as Rock Injector HP-80500 Pile Point manufactured by APF.

Alternatively, if computer models demonstrate a pile bent is adequate with a free or “friction” condition at the pile tips (i.e. a fixed or pinned condition with a rock socket is not required), the piles should be fitted with special pile tips such as Rock Injector HP-80500 Pile Point manufactured by APF, or equivalent, and driven to the required nominal resistance on bedrock.

7.2 Scour and Riprap

For all practical purposes, design analyses should demonstrate that lateral stability is adequate when the streambed soils are removed in their entirety. For the scour protection of pile pier bents, the piles may need to be constructed with fixed or pinned conditions at the pile tips by installing the lower section of the H-piles in shallow, drilled rock sockets, or fitting the piles with special pile tips such as Rock Injector HP-80500 Pile Point manufactured by APF or equivalent. If augers or cleaning core buckets are available, auger through overburden soils and install piles on rock surfaces cleaned of all weathered, loose and potentially erodible rock.

Bridge rehabilitation may require repair to the approach slopes. Riprap may also need to be placed at the toes of abutments. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour design and riprap. Approach slope repairs should consist of armoring the slopes with 3 feet of riprap conforming to Special Provisions 610 and 703. Stone riprap shall conform to item number Special Provision 703.26 Plain and Hand Laid Riprap and be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. Per Standard Details 610(2) through 610(04), the riprap section shall be underlain by a Class 1 nonwoven Erosion Control Geotextile and a 1 foot thick layer of bedding material conforming to item number 703.19, of the Standard Specification.

7.3 Seismic Design Considerations

In conformance with LRFD Table 4.7.4.3.1-1, seismic analysis is not required for multiple-span bridges in Seismic Zone 1. However, superstructure connections and bridge seat dimensions shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak ground acceleration coefficient (PGA) = 0.079g

- Design spectral acceleration coefficient at 0.2-second period, $S_{DS} = 0.163g$
- Design spectral acceleration coefficient at 1.0-second period, $S_{D1} = 0.044g$
- Site Class D (based on an average shear wave velocity (v_s), between 600 ft/s and 1,200 ft/sec, for the upper 100 ft of the soil profile)
- Seismic Zone 1, based on a $S_{D1} < 0.15g$

7.4 Construction Considerations

Construction activities will include pile driving and potentially bedrock drilling. There is a potential that timber obstructions, cobbles and/or boulders may obstruct pile installation operations, including constructing rock sockets and driving piles. Obstructions may be cleared by conventional excavation methods, preaugering, predrilling, core buckets, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident.

The nature slope and degree of fracturing in the bedrock surface will not be evident until the piles are installed. Loose fractured bedrock, loose decomposed bedrock and soil may be encountered at the location of the proposed pier bent and may not be suitable for end bearing piles.

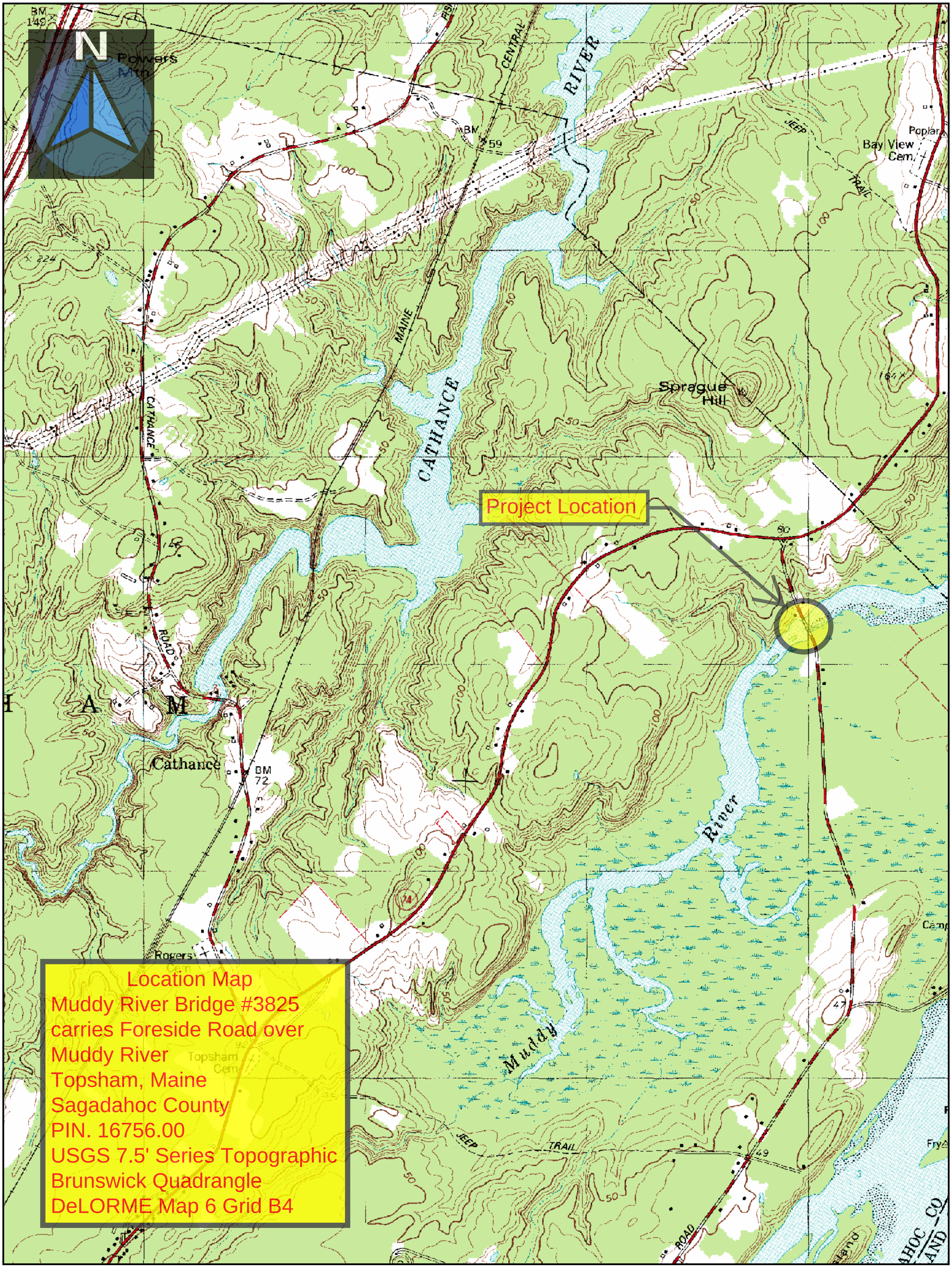
If during pile driving, pile behavior indicates that any driven pile is not seated in bedrock, the pile shall be re-driven. Due to the presence of sloping bedrock and thin overburden at the site, pile "walking" during driving is possible, and the contractor will be responsible for removing and re-driving any pile that are out of position. The contractor shall be required to support all piles laterally in their final positions until the connection of the pier bent cap to the superstructure is complete and in place.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed pier rehabilitation of Muddy River Bridge in Topsham, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is 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. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete 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.

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



Map Scale 1:24000

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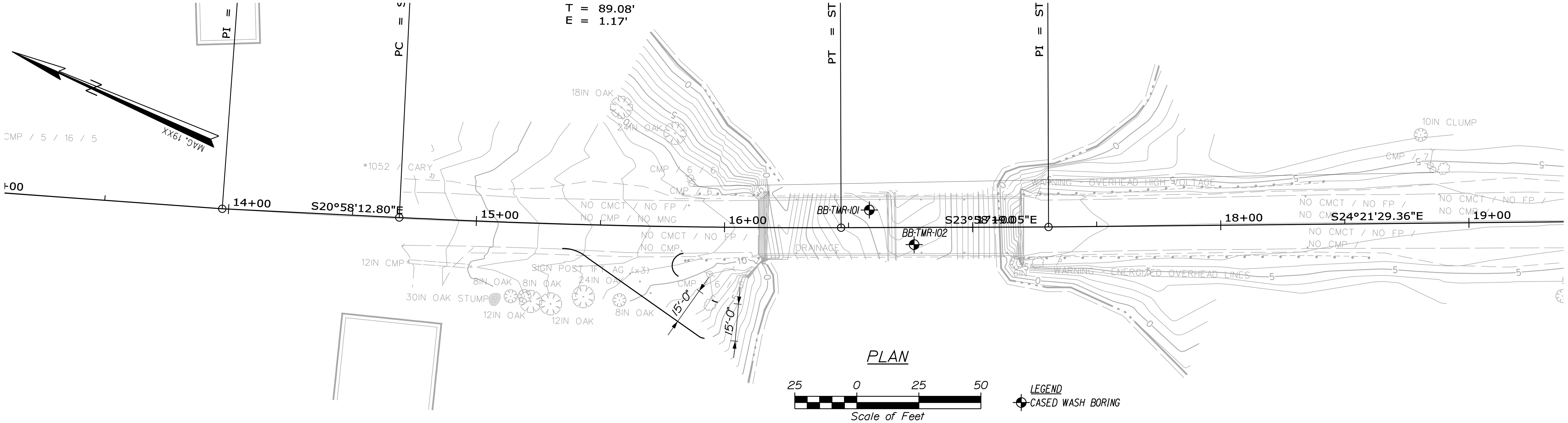

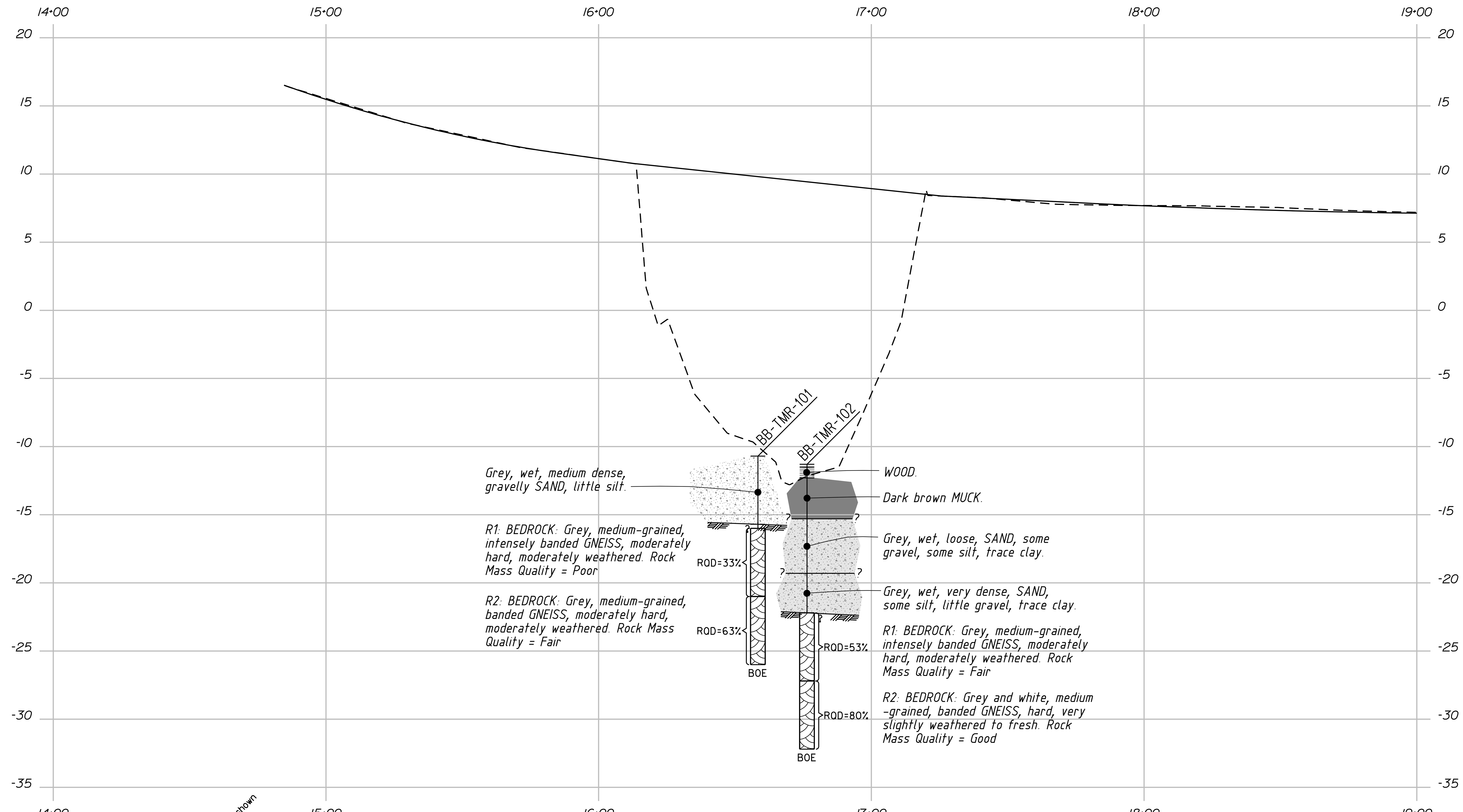

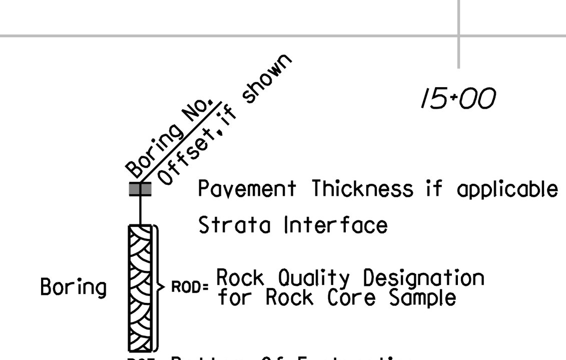

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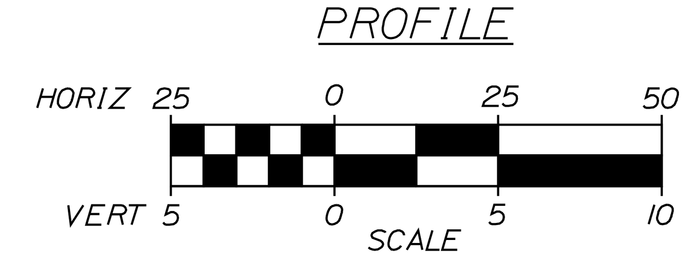

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1675(600)X

PIN 16756.00

BRIDGE PLANS

BRIDGE NO. 3825

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	L.KRUSINSKI	
CHECKED-REVIEWED	T.WHITE	JAN 2011
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MUDDY RIVER BRIDGE
MUDDY RIVER
TOPSHAM SAGADAHOC COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Muddy River Bridge #3825 carries Foreside Road over Muddy River
Location: Topsham, Maine

Boring No.: BB-TMR-101

PIN: 16756.00

Driller:	MaineDOT	Elevation (ft.)	-10.7	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/12/2010; 08:30-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	16+58.4, 7.1 Lt.	Casing ID/OD:	NW	Water Level*:	Water Boring (Tidal)
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/17	0.00 - 2.00	10/12/7/12	19	27	4		Grey, wet, medium dense, gravelly fine to coarse SAND, little silt.	G#240086 A-1-b, SM WC=11.5%
							17			
							53			
							39			
							108	-15.70	5.00	
	R1	60/60	5.30 - 10.30	RQD = 33%			NQ-2		Top of Bedrock at Elev. -15.7 ft. Roller Coned ahead to 5.3 ft bgs. R1:Bedrock: Grey, intensely banded, medium grained, quartz, feldspar, biotite GNEISS, moderately hard, moderately weathered, discontinuities and joints along banding at chaotic angles to predominantly low angles, 2nd joint set at steep angles, surfaces tight to open, stained and oxidized, no infilling. Nehumkeag Pond Formation. Rock Mass Quality: Poor. R1:Core Times (min:sec) 5.3-6.3 ft (3:00) 6.3-7.3 ft (2:30) 7.3-8.3 ft (2:15) 8.3-9.3 ft (2:30) 9.3-10.3 ft (2:40) 100% Recovery	
6										
	R2	60/60	10.30 - 15.30	RQD = 63%					R2:Bedrock: Grey, medium grained, quartz, feldspar, biotite, banded GNEISS, moderately hard, moderately weathered, highly banded to massive joint set along banding at moderately dipping angles, very stained and oxidized, biotite bands relatively soft. Nehumkeag Pond Formation. Rock Mass Quality: Fair. R2:Core Times (min:sec) 10.3-11.3 ft (2:00) 11.3-12.3 ft (2:40) 12.3-13.3 ft (4:45) 13.3-14.3 ft (5:22) 14.3-15.3 ft (3:00) 100% Recovery	
12										
								-26.00	15.30 Bottom of Exploration at 15.30 feet below ground surface.	
18										
24										
30										

Remarks:

20.0 ft from Bridge Deck to Ground.

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-TMR-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Muddy River Bridge #3825 carries Foreside Road over Muddy River
Location: Topsham, Maine

Boring No.: BB-TMR-102

PIN: 16756.00

Driller:	MaineDOT	Elevation (ft.)	-11.3	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/12/2010; 12:30-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	16+76.4, 7.0 Rt.	Casing ID/OD:	NW	Water Level*:	Water Boring (Tidal)
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/3	0.00 - 2.00	3/2/WOH/WOH	2	3	12	-11.50	0.20 WOOD from 0.2-1.0 ft bgs.	
							WOC	-12.30	1.00 Dark brown muck in wash water from 1.0- 3.0 ft bgs.	
							WOC			
							16	-15.30	4.00	
	2D	24/18	4.00 - 6.00	9/2/1/3	3	4	16		Grey, saturated, very loose, fine to coarse SAND, some gravel, some silt, trace clay.	G#240087 A-2-4, SC-SM WC=17.8%
6							5			
							9			
							5	-19.30	8.00	
							12			
	3D	22.8/17	9.00 - 10.90	5/42/32/50	74	104	5		Grey, wet, very dense, fine to coarse SAND, some silt, little gravel, trace clay, rock fragments in tip of spoon. a64 blows for 0.9 ft.	G#240088 A-2-4, SC-SM WC=4.9%
	R1	60/58	10.90 - 15.90	RQD = 53%			a64 NQ-2	-22.20	10.90 Top of Bedrock at Elev. -22.2 ft. R1:Bedrock: Grey, medium grained, Quartz, Feldspar, Biotite, intensely banded GNEISS, moderately hard, moderately weathered, joint set along banding at moderately dipping angles, tight to healed, stained and oxidized, no infiling. Nehumkeag Pond Formation. Rock Mass Quality: Fair. R1:Core Times (min:sec) 10.9-11.9 ft (2:15) 11.9-12.9 ft (2:40) 12.9-13.9 ft (2:25) 13.9-14.9 ft (2:00) 14.9-15.9 ft (2:00) 97% Recovery	
12										
	R2	60/60	15.90 - 20.90	RQD = 80%					R2:Bedrock: Grey and white, banded, medium grained, Quartz, Feldspar, Biotite, GNEISS, hard, very slightly weathered to fresh, banding at low to moderate angles, joints widely spaced, tight, stained, no infilling. Nehumkeag Pond Formation. Rock Mass Quality: Good. R2:Core Times (min:sec) 15.9-16.9 ft (1:50) 16.9-17.9 ft (2:10) 17.9-18.9 ft (2:40) 18.9-19.9 ft (3:25) 19.9-20.9 ft (4:10) 100% Recovery	
18										
								-32.20	20.90 Bottom of Exploration at 20.90 feet below ground surface.	
24										
30										

Remarks:

20.3 ft from Bridge Deck to Ground.

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-TMR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1675(600)X

BRIDGE NO. 3825

PIN 16756.00

BRIDGE PLANS

PROJ. MANAGER		BY	DATE
DESIGN-DETAILED	L.KRUSINSKI	T.WHITE	MAR 2011
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

MUDDY RIVER BRIDGE
MUDDY RIVER
TOPSHAM SAGADAHOC COUNTY

BORING LOGS

SHEET NUMBER

3

OF 3

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Muddy River Bridge #3825 carries Foreside Road over Muddy River
Location: Topsham, Maine

Boring No.: BB-TMR-101
PIN: 16756.00

Driller:	MaineDOT	**Elevation (ft.)**	-10.7	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/12/2010; 08:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+58.4, 7.1 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	Water Boring (Tidal)
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.00 - 2.00	10/12/7/12	19	27	4			Grey, wet, medium dense, gravelly fine to coarse SAND, little silt.	G#240086 A-1-b, SM WC=11.5%
							17				
							53				
							39				
							108	-15.70		5.00	
	R1	60/60	5.30 - 10.30	RQD = 33%			NQ-2			Top of Bedrock at Elev. -15.7 ft. Roller Coned ahead to 5.3 ft bgs. R1:Bedrock: Grey, intensely banded, medium grained, quartz, feldspar, biotite GNEISS, moderately hard, moderately weathered, discontinuities and joints along banding at chaotic angles to predominantly low angles, 2nd joint set at steep angles, surfaces tight to open, stained and oxidized, no infilling. Nehumkeag Pond Formation. Rock Mass Quality: Poor. R1:Core Times (min:sec) 5.3-6.3 ft (3:00) 6.3-7.3 ft (2:30) 7.3-8.3 ft (2:15) 8.3-9.3 ft (2:30) 9.3-10.3 ft (2:40) 100% Recovery	
6											
	R2	60/60	10.30 - 15.30	RQD = 63%						R2:Bedrock: Grey, medium grained, quartz, feldspar, biotite, banded GNEISS, moderately hard, moderately weathered, highly banded to massive joint set along banding at moderately dipping angles, very stained and oxidized, biotite bands relatively soft. Nehumkeag Pond Formation. Rock Mass Quality: Fair. R2:Core Times (min:sec) 10.3-11.3 ft (2:00) 11.3-12.3 ft (2:40) 12.3-13.3 ft (4:45) 13.3-14.3 ft (5:22) 14.3-15.3 ft (3:00) 100% Recovery	
12											
								-26.00		15.30	
										Bottom of Exploration at 15.30 feet below ground surface.	
18											
24											
30											

Remarks:

20.0 ft from Bridge Deck to Ground.

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: Muddy River Bridge #3825 carries Foreside Road over Muddy River
Location: Topsham, Maine

Boring No.: BB-TMR-102
PIN: 16756.00

Driller:	MaineDOT	**Elevation (ft.)**	-11.3	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/12/2010; 12:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+76.4, 7.0 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	Water Boring (Tidal)
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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Hammer Efficiency Factor = Annual Calibration Value
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/3	0.00 - 2.00	3/2/WOH/WOH	2	3	12	-11.50		0.20 WOOD from 0.2-1.0 ft bgs.	
							WOC	-12.30		1.00 Dark brown muck in wash water from 1.0- 3.0 ft bgs.	
							WOC				
							16				
	2D	24/18	4.00 - 6.00	9/2/1/3	3	4	16	-15.30		4.00 Grey, saturated, very loose, fine to coarse SAND, some gravel, some silt, trace clay.	G#240087 A-2-4, SC-SM WC=17.8%
							5				
6							9				
							5				
							12	-19.30		8.00	
	3D	22.8/17	9.00 - 10.90	5/42/32/50	74	104	5			Grey, wet, very dense, fine to coarse SAND, some silt, little gravel, trace clay, rock fragments in tip of spoon. a64 blows for 0.9 ft.	G#240088 A-2-4, SC-SM WC=4.9%
	R1	60/58	10.90 - 15.90	RQD = 53%			a64 NQ-2	-22.20		10.90 Top of Bedrock at Elev. -22.2 ft. R1:Bedrock: Grey, medium grained, Quartz, Feldspar, Biotite, intensely banded GNEISS, moderately hard, moderately weathered, joint set along banding at moderately dipping angles, tight to healed, stained and oxidized, no infiling. Nehumkeag Pond Formation. Rock Mass Quality: Fair. R1:Core Times (min:sec) 10.9-11.9 ft (2:15) 11.9-12.9 ft (2:40) 12.9-13.9 ft (2:25) 13.9-14.9 ft (2:00) 14.9-15.9 ft (2:00) 97% Recovery	
12											
	R2	60/60	15.90 - 20.90	RQD = 80%						R2:Bedrock: Grey and white, banded, medium grained, Quartz, Feldspar, Biotite, GNEISS, hard, very slightly weathered to fresh, banding at low to moderate angles, joints widely spaced, tight, stained, no infilling. Nehumkeag Pond Formation. Rock Mass Quality: Good. R2:Core Times (min:sec) 15.9-16.9 ft (1:50) 16.9-17.9 ft (2:10) 17.9-18.9 ft (2;40) 18.9-19.9 ft (3:25) 19.9-20.9 ft (4:10) 100% Recovery	
18											
								-32.20		20.90 **Bottom of Exploration at 20.90 feet below ground surface.**	
24											
30											

Remarks:

20.3 ft from Bridge Deck to Ground.

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.

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)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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 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):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*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)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

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

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

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Topsham — Project Number: 16756.00

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-TMR-101, 1D	16+58.4	7.1 Lt.	0.0-2.0	240086	1	11.5			SM	A-1-b	II
BB-TMR-102, 2D	16+76.4	7.0 Rt.	4.0-6.0	240087	1	17.8			SC-SM	A-2-4	II
BB-TMR-102, 3D	16+76.4	7.0 Rt.	9.0-10.9	240088	1	4.9			SC-SM	A-2-4	III

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

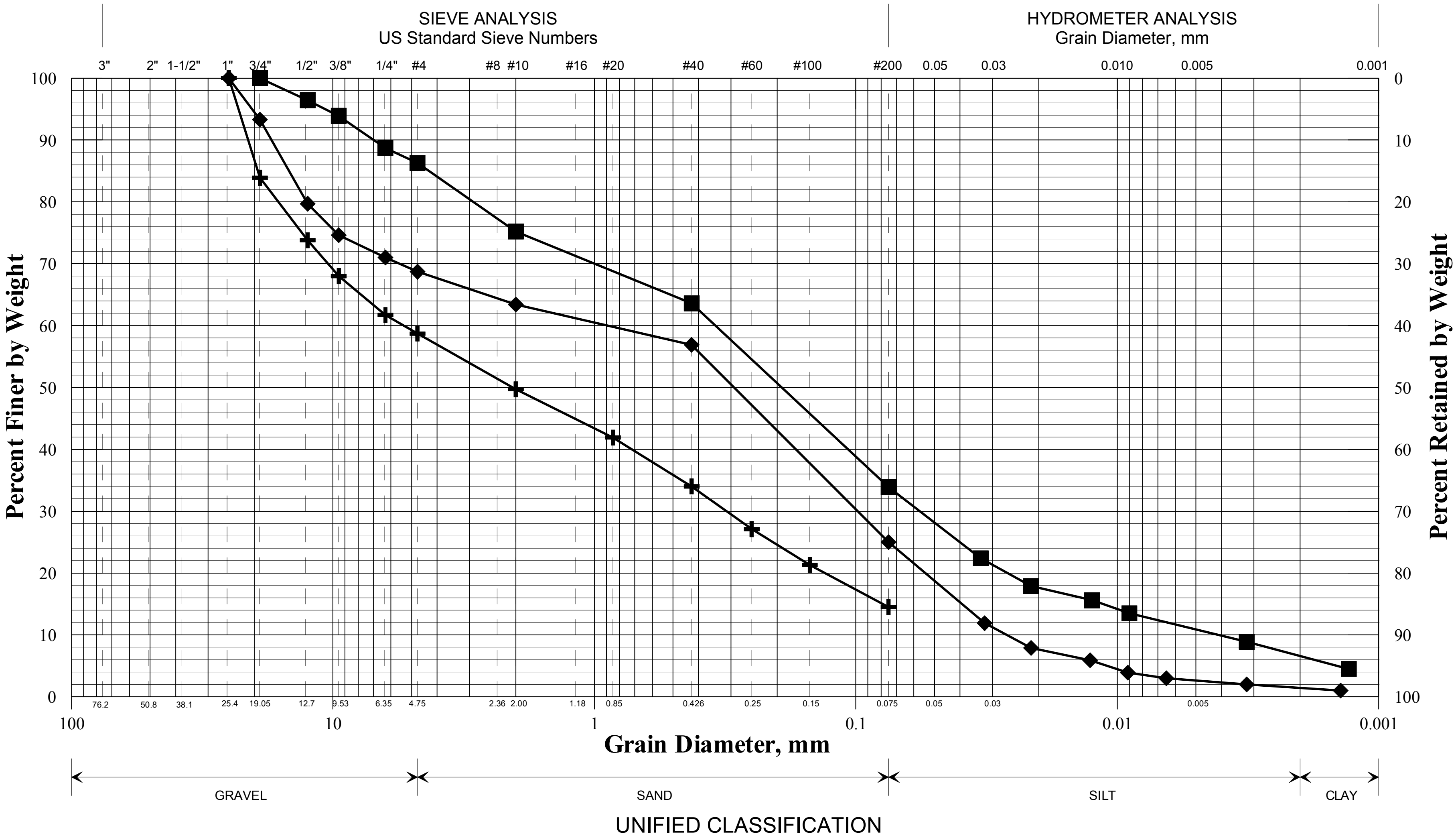
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-TMR-101/1D	16+58.4	7.1 LT	0.0-2.0	Gravelly SAND, little silt.	11.5			
◆	BB-TMR-102/2D	16+76.4	7.0 RT	4.0-6.0	SAND, some gravel, some silt, trace clay.	17.8			
■	BB-TMR-102/3D	16+76.4	7.0 RT	9.0-10.9	SAND, some silt, little gravel, trace clay.	4.9			
●									
▲									
✖									

PIN
016756.00
Town
Topsham
Reported by/Date
WHITE, TERRY A 11/9/2010

SHEET 1

Appendix C

Calculations

Bedrock Properties at the Site

RQD of bedrock cores
33% in BB-TMR-101
53% in BB-TMR-102

Rock Type: Sedimentary GNEISS

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);

uniaxial compressive strength = Co= 3500 to 45,000 psi - use **20,000 psi** for design AASHTO TABLE 4.4.8.1.2.B

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

$$A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2 \qquad d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$$

$$A_{box} := \overrightarrow{(d \cdot b)} \qquad A_{box} = \begin{pmatrix} 141.89 \\ 148.168 \\ 198.356 \\ 203.232 \\ 211.516 \end{pmatrix} \cdot in^2$$

Nominal and Factored Structural Compressive Resistance of HP piles

Axial pile resistance may be controlled by structural resistance if driven to sound bedrock
Use LRFD Equation 6.9.2.1-1

$$F_y := 50 \cdot ksi$$

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QFyAs (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$$P_o := Q \cdot F_y \cdot A_s \qquad P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Determine elastic critical buckling resistance Pe, LRFD eq. 6.9.4.1.2-1

E = Elastic Modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 (assume rotation free at pile tip

l = unbraced length $l_{unbraced} := 15 \cdot ft$ Assume cross bracing reduces the total unbraced length from 25-30 feet to 15 feet

r s = radius of gyration

$$r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$$

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 279.999 \\ 410.502 \\ 575.648 \\ 718.262 \\ 979.129 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 0.361 \\ 0.377 \\ 0.538 \\ 0.55 \\ 0.569 \end{pmatrix}$$

If Pe/Po > or = 0.44, then:

$$P_n := \overrightarrow{\left(0.658^{\frac{P_o}{P_e}} \cdot P_o \right)}$$

LRFD Eq. 6.9.4.1.1-1

then

$$P_n = \begin{pmatrix} 243 \\ 359 \\ 491 \\ 610 \\ 825 \end{pmatrix} \cdot kip$$

this applies to the 3 larger pile sizes

If Pe/Po < 0.44, then:

$$P_{n1} := \overrightarrow{\left(0.877 \cdot P_e \right)}$$

$$P_{n1} = \begin{pmatrix} 246 \\ 360 \\ 505 \\ 630 \\ 859 \end{pmatrix} \cdot kip$$

This applies to
the 2 smaller pile sizes

Factored Axial Structural Resistance of single H pile

Resistance factor or H-pile in compression, good driving condtions LRFD 6.5.4.2

$$\phi_c := 0.6$$

For the 3 larger pile sizes the Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$$P_r := \phi_c \cdot P_n$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 146 \\ 215 \\ 295 \\ 366 \\ 495 \end{pmatrix} \cdot kip$$

For the 2 smallest pile sizes the Factored Structural Resistance (Pr) per LRFD 6.9.2.1-1 is

$$P_r := \phi_c \cdot P_{n1}$$

Factored structural compressive resistance, Pr

$$P_r = \begin{pmatrix} 147 \\ 216 \\ 303 \\ 378 \\ 515 \end{pmatrix} \cdot kip$$

Assume crossbracing reduces Le to 15 feet: the factored structural resistances are 147, 216, 295, 366, 495.

Nominal and Factored Axial Geotechnical Resistance of HP piles

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, qp

Design value of compressive strength of rock core

Banded Gneiss $q_{u\_1} := 20000 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor $dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$ and dd < 3

$dd = 1$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd \qquad q_{p\_1} = \begin{pmatrix} 1953 \\ 1951 \\ 1920 \\ 1918 \\ 1916 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)} \qquad R_{p\_1} = \begin{pmatrix} 210 \\ 295 \\ 285 \\ 348 \\ 458 \end{pmatrix} \cdot kip$$

with no
rock socket

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1} \qquad R_{r\_p1} = \begin{pmatrix} 95 \\ 133 \\ 128 \\ 156 \\ 206 \end{pmatrix} \cdot kip$$

with no
rock socket

Factored Structural Resistance

Assume crossbracing reduces Le to 15 feet: the factored structural resistances are 147, 216, 295, 366, 495.

Nominal and Factored Axial Geotechnical Resistance of HP piles with 1-foot or 2-ft bedrock socket

Geotechncial axial pile resistance for pile end bearing on rock is determined by CGS method (LRFD Table 10.5.5.2.3-1) and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, qp, for a 1-foot deep rock socket

Design value of compressive strength of rock core

Banded Gneiss $q_{u\_1} := 20000 \cdot psi$

Spacing of discontinuities $s_d := 4 \cdot in$

Width of discontinuities. Joints are open to tight per boring logs $t_d := \frac{1}{64} \cdot in$

Pile width is b - matrix $D := b$

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 1.0 \cdot ft$

Diameter of socket: $D_s := 12 \cdot in$

Depth factor

$$dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$$ and dd < 3

$$dd = 1.4$$ OK

Ksp

$$K_{sp} := \frac{3 + \frac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \frac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.226 \\ 0.226 \\ 0.222 \\ 0.222 \\ 0.222 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 2735 \\ 2731 \\ 2687 \\ 2686 \\ 2683 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 294 \\ 413 \\ 399 \\ 487 \\ 641 \end{pmatrix} \cdot kip$$

with 1-ft rock socket

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Candadian Geotechnical Society method

$$\phi_{stat} := 0.45$$

LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 132 \\ 186 \\ 180 \\ 219 \\ 288 \end{pmatrix} \cdot kip$$

with 1 ft rock socket

Nominal unit bearing resistance of pile point, qp, for a 2-foot deep rock socket

Embedment depth of pile in socket - pile is end bearing on rock $H_s := 2.0 \cdot ft$

Depth factor

$$dd := 1 + 0.4 \cdot \frac{H_s}{D_s}$$

and dd < 3

$$dd = 1.8$$

OK

Geotechnical tip resistance.

$$q_{p\_1} := 3 \cdot q_{u\_1} \cdot K_{sp} \cdot dd$$

$$q_{p\_1} = \begin{pmatrix} 3516 \\ 3511 \\ 3455 \\ 3453 \\ 3449 \end{pmatrix} \cdot ksf$$

Nominal geotechnical tip resistance, Rp - *Extreme Limit States and Service Limit States*

Case I

$$R_{p\_1} := \overrightarrow{\left(q_{p\_1} \cdot A_s\right)}$$

$$R_{p\_1} = \begin{pmatrix} 378 \\ 532 \\ 513 \\ 626 \\ 824 \end{pmatrix} \cdot kip$$

with 2-ft rock socket

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Factored Geotechnical Tip Resistance (Rr)

$$R_{r\_p1} := \phi_{stat} \cdot R_{p\_1}$$

$$R_{r\_p1} = \begin{pmatrix} 170 \\ 239 \\ 231 \\ 282 \\ 371 \end{pmatrix} \cdot kip$$

with 2-ft rock socket

Drivability Analysis

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 6-10 bpi which is 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.

$\phi_{dyn} := 0.65$

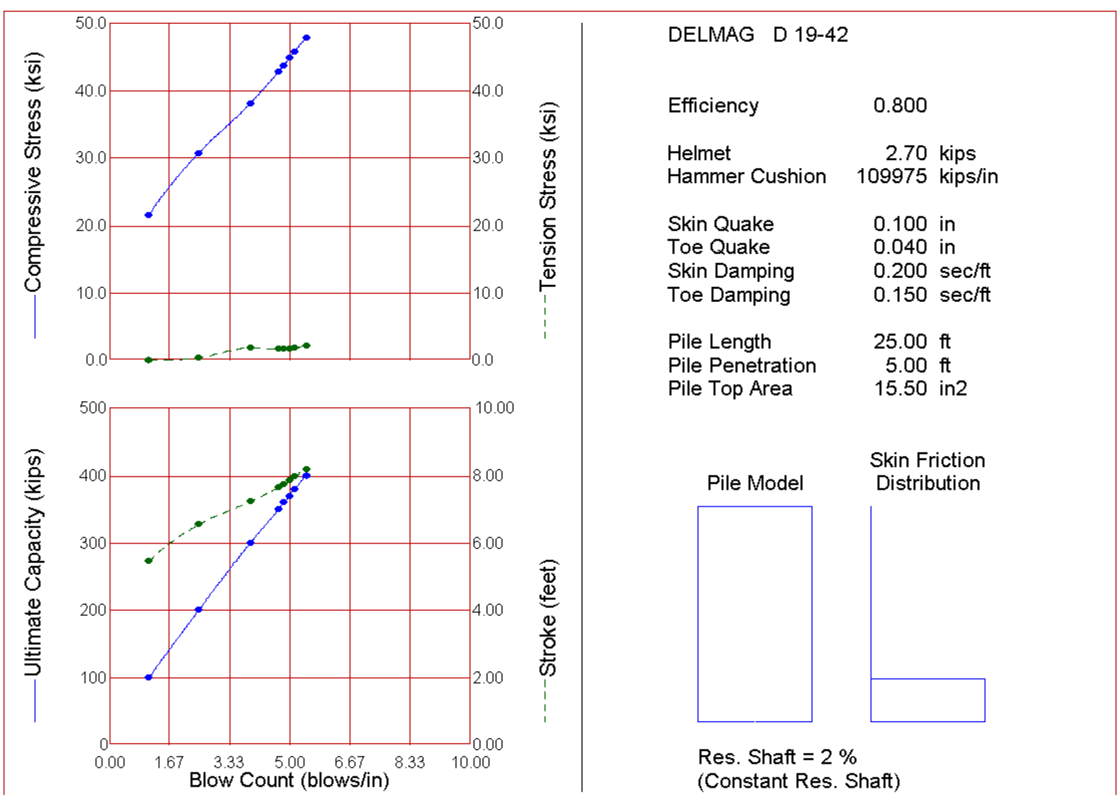
Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation | 14-Mar-2011
12 x 53 Delmag D19-42 | GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	21.57	0.05	1.1	5.45	14.86
200.0	30.77	0.38	2.5	6.55	13.95
300.0	38.13	1.85	3.9	7.24	14.68
350.0	42.83	1.76	4.7	7.65	15.39
360.0	43.78	1.76	4.8	7.74	15.62
370.0	44.85	1.79	5.0	7.85	15.82
380.0	45.87	1.83	5.1	7.98	16.08
400.0	47.77	2.22	5.5	8.19	16.52

State of Maine Dept. Of Transportation | 14-Mar-2011
12 x 53 Delmag D19-42 | GRLWEAP (TM) Version 2003



Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.85}{45.87 - 44.85}\right)\cdot(380\cdot kip - 370\cdot kip) + 370\cdot kip$$

$$R_{ndr} = 371.5\cdot kip$$

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$\boxed{R_{fdr} = 241\cdot kip}$ use this

Examine using a Delmag D 16-32

State of Maine Dept. Of Transportation 14-Mar-2011
12 x 53 Delmag D19-32 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	32.77	0.41	2.8	7.07	12.92
340.0	44.50	1.88	5.0	8.24	14.35
360.0	45.98	2.76	5.3	8.44	14.66
380.0	47.50	3.45	5.7	8.65	14.97
400.0	48.66	3.90	6.2	8.77	15.12
420.0	49.99	4.31	6.6	8.96	15.42
440.0	51.69	4.68	7.0	9.27	15.95

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.5}{45.98 - 44.5}\right)\cdot(360\cdot kip - 340\cdot kip) + 340\cdot kip$$

$$R_{ndr} = 346.8\cdot kip$$

For a resistance factor for dynamic test of 0.65:

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

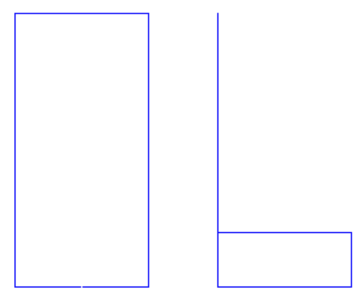
$$\boxed{R_{fdr} = 225\cdot kip}$$

DELMAG D 16-32

Efficiency	0.800	
Helmet	1.90	kips
Hammer Cushic	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	15.50	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

Pile Size is 12 x 74

The 12x 74 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 16-Mar-2011
12 x 74 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	25.18	0.18	2.6	6.57	13.62
400.0	36.73	2.69	5.5	7.65	14.44
450.0	39.66	2.65	6.2	8.07	15.17
500.0	42.44	2.65	7.1	8.50	15.88
550.0	44.65	2.82	8.5	8.78	16.19
600.0	47.00	3.31	9.8	9.15	16.84
650.0	49.28	3.69	11.4	9.55	17.56

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.65}{47. - 44.65}\right)\cdot(600\cdot kip - 550\cdot kip) + 550\cdot kip$$

$$R_{ndr} = 557.4\cdot kip$$

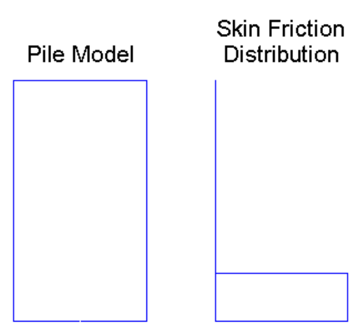
For a resistance factor for dynamic test of 0.65:

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$R_{fdr} = 362\cdot kip$$ use this

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	21.80	in2



Res. Shaft = 2 %
(Constant Res. Shaft)

Delmag 16-32

State of Maine Dept. Of Transportation — 14-Mar-2011
12 x 74 Delmag D16-32 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	33.89	0.41	4.5	7.69	12.87
350.0	36.84	1.64	5.4	7.96	13.06
400.0	40.01	2.10	6.3	8.35	13.51
440.0	42.45	2.28	7.0	8.68	13.93
480.0	44.54	2.48	8.0	8.96	14.26
490.0	44.98	2.36	8.3	9.02	14.37
500.0	45.45	2.50	8.6	9.08	14.49
510.0	45.95	2.51	8.9	9.15	14.61

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.98}{45.95 - 44.98}\right)\cdot(500\cdot kip - 490\cdot kip) + 490\cdot kip$$

$$R_{ndr} = 490.2\cdot kip$$

For a resistance factor for dynamic test of 0.65:

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

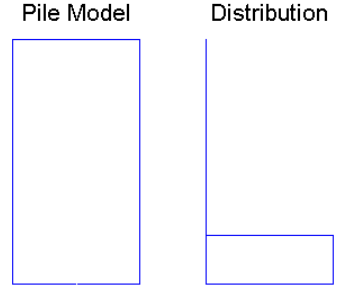
$$R_{fdr} = 319\cdot kip$$

14-Mar-2011
GRLWEAP (TM) Version 2003

DELMAG D 16-32

Efficiency	0.800	
Helmet	1.90	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

Pile Size is 14 x 73

The 14x 73 pile can be driven to the resistances below with a D 16-32 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation 15-Mar-2011
14 x 73 Delmag D16-32 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
200.0	27.19	0.14	3.0	7.11	12.65
300.0	34.37	0.40	4.5	7.68	12.91
350.0	37.47	1.77	5.3	7.98	13.12
400.0	40.54	2.33	6.2	8.37	13.56
420.0	41.84	2.40	6.6	8.54	13.79
440.0	43.04	2.40	7.0	8.71	13.97
460.0	44.23	2.64	7.4	8.92	14.31
480.0	45.08	2.56	8.0	8.98	14.29

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.23}{45.08 - 44.23}\right) \cdot (480 \cdot kip - 460 \cdot kip) + 460 \cdot kip$$

$$R_{ndr} = 478 \cdot kip$$

For a resistance factor for dynamic test of 0.65:

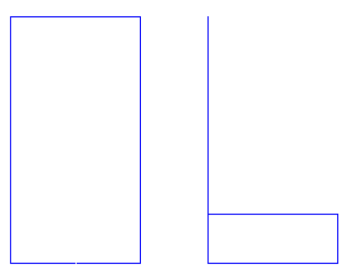
$$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$$

$$\boxed{R_{fdr} = 311 \cdot kip}$$

DELMAG D 16-32

Efficiency	0.800	
Helmet	1.90	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	21.40	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

14 x 73 with Delmag 19-42 and 2.7 kip helmet

State of Maine Dept. Of Transportation — 15-Mar-2011
14 x 73 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
100.0	19.73	0.05	1.1	5.49	14.78
300.0	31.54	0.84	3.9	7.10	13.82
400.0	37.23	2.82	5.5	7.67	14.51
480.0	41.93	2.75	6.7	8.37	15.73
500.0	43.05	2.69	7.1	8.54	15.98
520.0	43.82	2.71	7.7	8.61	16.03
540.0	44.79	2.77	8.2	8.75	16.24
580.0	46.67	3.07	9.2	9.04	16.72

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.79}{46.67 - 44.79}\right)\cdot(580\cdot kip - 540\cdot kip) + 540\cdot kip$$

$$R_{ndr} = 544\cdot kip$$

For a resistance factor for dynamic test of 0.65:

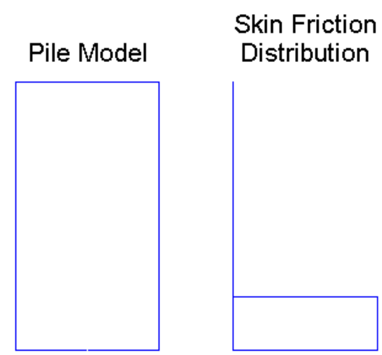
$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 354\cdot kip}$$

Use this result

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	21.40	in2



Res. Shaft = 2 %
(Constant Res. Shaft)

Pile Size is 14 x 89

The 14 x 89 pile can be driven to the resistances below with a D 19-42 at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation — 15-Mar-2011
14 x 73 Delmag D19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	27.79	0.50	4.1	7.05	13.39
500.0	37.81	2.30	7.3	8.19	14.76
600.0	42.06	4.07	9.7	8.78	15.66
650.0	44.16	4.40	11.1	9.14	16.29
660.0	44.60	4.43	11.4	9.23	16.43
680.0	45.43	4.41	12.1	9.37	16.71
700.0	46.27	4.09	12.7	9.52	17.04
800.0	49.90	2.66	16.8	10.17	18.46

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.60}{45.43 - 44.60}\right)\cdot(680\cdot kip - 660\cdot kip) + 660\cdot kip$$

$$R_{ndr} = 669.6\cdot kip$$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	26.10	in2

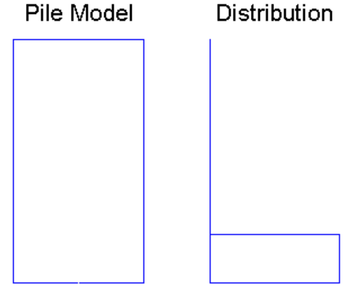
For a resistance factor for dynamic test of 0.65:

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 435\cdot kip}$$

Blow count is 12 bpi which is out of the optimal range! Caution.

Pile Model — Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14 x 117 pile can be driven to the resistances below with a D 36-32 at Fuel Setting 3 and a 2.7 kip helmet, at a reasonable blow count and level of driving stress. See GRLWEAP results below:

State of Maine Dept. Of Transportation
14 x 117 Delmag 36-32

15-Mar-2011
GRLWEAP (TM) Version 2003

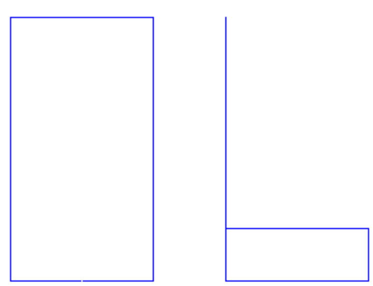
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	29.35	0.11	1.7	6.64	31.14
600.0	44.80	1.04	3.8	7.97	31.14
650.0	46.96	1.44	4.1	8.21	31.97
700.0	48.85	1.97	4.5	8.34	32.16
750.0	50.73	2.73	4.9	8.54	32.58
800.0	52.63	2.74	5.4	8.75	33.06

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

Limiting driving stress to 45 ksi:

$$R_{ndr} := \left(\frac{45 - 44.80}{46.96 - 44.80}\right)\cdot(650\cdot kip - 600\cdot kip) + 600\cdot kip$$

$$R_{ndr} = 604.6\cdot kip$$

For a resistance factor for dynamic test of 0.65:

$$R_{fdr} := R_{ndr}\cdot\phi_{dyn}$$

$$\boxed{R_{fdr} = 393\cdot kip}$$

Blow count is 4 bpi and out of the optimal range (5-10).
Use result from 19-42.

State of Maine Dept. Of Transportation 15-Mar-2011
14 x 117 Delmag 19-42 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	23.32	0.43	4.4	7.05	13.15
600.0	35.51	1.72	9.9	8.46	15.20
650.0	37.20	1.62	11.3	8.70	15.75
700.0	38.92	1.37	12.7	8.98	16.41
750.0	40.51	1.50	14.4	9.22	16.95
800.0	41.84	2.46	16.5	9.41	17.39

Limiting driving to 10 bpi (10 bpi is optimal for diesel hammer):

$R_{ndr} := 600 \cdot kip$

$R_{ndr} = 600 \cdot kip$

For a resistance factor for dynamic test of 0.65:

$R_{fdr} := R_{ndr} \cdot \phi_{dyn}$

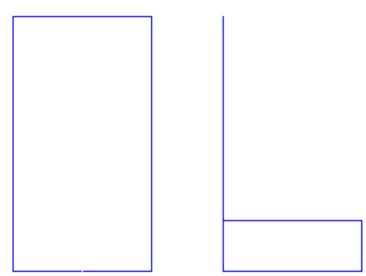
$\boxed{R_{fdr} = 390 \cdot kip}$

Use this result based on a critieria of 10 bpi

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetration	5.00	ft
Pile Top Area	34.40	in2

Pile Model Skin Friction Distribution



Res. Shaft = 2 %
(Constant Res. Shaft)

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04086
Zip Code Latitude = 43.940400
Zip Code Longitude = -069.968200
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.079	PGA - Site Class B
0.2	0.163	Ss - Site Class B
1.0	0.044	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04086
Zip Code Latitude = 43.940400
Zip Code Longitude = -069.968200
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.079	As - Site Class B
0.2	0.163	SDs - Site Class B
1.0	0.044	SD1 - Site Class B