State of Maine

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Bridge Program – Geotechnical Section
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Addendum #1

To:	Jack Burgess, P.E. Becker Structural Engineers, Inc. 75 York Street #3 Portland, Maine 04101
Cc:	Laura Krusinski, Terry White, TeDocs
From:	Brandon Slaven
Subject:	Soils Report No. 2014- 26 H-pile Resistances and Integral Abutment Design Recommendations
Date:	September 3, 2015
Bridge #:	3736
Street:	Route 191 over Denny's River
PIN:	20506.00
Town:	Meddybemps

The purpose of this addendum is to transmit estimated H-pile resistances and pile design recommendations for the replacement of Meddybemps Bridge in Meddybemps, Maine. A geotechnical design report was published for this project in November 2014 (MaineDOT Soils Report No. 2014-26). The design information included in the report provided geotechnical design recommendations for spread footings bearing on bedrock. It is now anticipated that the proposed structure will have an increased span length of 70 feet. Economic factors dictate spread footings are no longer the preferred substructure type. Integral abutments, founded on a single row of H-piles, have been identified as the preferred foundation alternative at the new abutment locations. This addendum should be used as a supplement to the original geotechnical report to aid in the design of the replacement structure.

INTEGRAL ABUTMENT H-PILES:

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of H-piles. The piles should be end bearing and driven to the required resistance on bedrock or within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles should be 50 ksi, Grade A572 steel. The piles should be oriented for weak axis bending. Abutment No. 1 and Abutment No. 2 piles should be fitted with Rock Injector Pile Points (HP-80500 for 14-inch sections or APF-R-12 for 12-inch sections), manufactured by Associated Pile and Fitting (APF), LLC, or equivalent. These points protect pile tips, improve penetration and friction at the pile tip, and support pinned pile tip assumptions. Special Provision 501 Foundation Piles – Rock Injector Pile Tip is provided at the end of this addendum.

Pile lengths at the centerline of proposed abutments may be estimated based on Table 1:

Location	Estimated Bottom Elevation of Proposed Abutment (feet)	Interpolated Top of Bedrock Elevation at Proposed Centerline (feet)	Estimated Pile Lengths at Proposed Centerline (feet)
Abutment No. 1	162.4	151.5	10.9
Abutment No. 2	161.2	149.3	11.9

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

A comprehensive range of estimated pile lengths (as measured from estimated bottom of abutment) considering all top of bedrock elevations encountered in the borings and assumed top of bedrock elevations encountered in power auger probes is provided at the end of this addendum.

The estimated pile lengths in Table 1 and comprehensive pile length table at the end of this addendum 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 (5) 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 or pile cap.

To accommodate integral abutment piles at proposed Abutments No. 1 and No. 2, the following are recommended:

- Piles supporting integral abutments should be designed in accordance with AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD); the design example in *Integral Abutment Bridge Design Guidelines,* VTrans Structures Section, 2008; the design example found in Appendix B of Technical Report ME-01-7, University of Maine, June 2005, *Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock – Phase I;* and Chapter 5 of Technical Report ME-01-7

- A series of lateral pile resistance analyses should be performed by the geotechnical engineer to evaluate pile behavior at both abutments using L-Pile® Plus 5.0 (L-Pile) software with pile head deflections, moments, and axial loads supplied by the structural engineer. The designer should utilize the results of the L-Pile analyses to recalculate axial compressive structural pile resistances based on unbraced pile segments and verify pile bending stresses do not exceed allowable stresses.

- The abutment slopes must provide lateral support to the piles supporting the abutments and will require special scour countermeasures, consisting of thicker riprap, heavy riprap, concrete cable mats, or toe berms to satisfy the need for foundation stability after the Q_{100} and Q_{500} flood events.

STRENGTH LIMIT STATE DESIGN:

The design of pile foundations bearing on or within 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 LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$ (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 $\phi_c = 0.70$ and the flexural resistance factor $\phi_f = 1.0$ 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).

Abutment H-piles should be analyzed by the geotechnical engineer for determination of unbraced lengths and fixity using L-Pile software. The calculated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2.

Structural Resistance. 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 five (5) H-pile sections were calculated using a resistance factor, $\phi_c = 0.50$, for a pile segment subject to severe driving conditions and are provided in Table 2. The unbraced pile lengths (ℓ) and effective length factors (K) in these evaluations have been assumed. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (ℓ) and effective length factors (K) determined from L-Pile.

Geotechnical Resistance. 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 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 2.

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 five (5) H-piles sections for the strength limit state is provided in Table 2. Supporting calculations are provided at the end of this addendum.

Pile Section	Strength Limit State Factored Axial Pile Resistance			
	Structural Resistance[1] ϕ_c=0.50 (kips)	Controlling Geotechnical Resistance[2] ϕ_c=0.50 (kips)	Drivability Resistance φ_{dyn} = 0.65 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	386[3]	386[3]	273	273
HP 12 x 74	542	542	403	403
HP 14 x 73	533[3]	533[3]	400	400
HP 14 x 89	650	650	445	445
HP 14 x 117	857	857	523	523

Table 2 – 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 driven with a Delmag D19-42 hammer are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3. Therefore, the recommended governing resistances for pile design are the drivability resistances provided in the rightmost column "Governing Axial Pile Resistance (kips)" in Table 2. The maximum applied factored axial pile load should not exceed the governing factored pile resistance shown in Table 2.

[1]Structural resistances were calculated for approximated normal conditions (no scour). Controlling value shown here is for a segment in pure compression using a resistance factor, ϕ_c=0.50, for severe driving conditions. Factored structural resistances should be calculated for upper and lower unbraced segments based on L-Pile results using a resistance factor, ϕ_c=0.70, for combined axial loading and bending.
[2]Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock*
[3] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions based upon structural performance considerations.

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 in 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 five (5) H-pile sections for the extreme and service limit states are provided in Table 3. Supporting calculations are provided at the end of this addendum.

Pile Section	Extreme and Service Limit State Factored Axial Pile Resistance			
	Structural Resistance (normal conditions)[4] ϕ_c=1.0 (kips)	Controlling Geotechnical Resistance ϕ_c=1.0[5] (kips)	Drivability Resistance φ = 1.0 (kips)	Governing Axial Pile Resistance (kips)
HP 12 x 53	771[6]	771[6]	420	420
HP 12 x 74	1085	1085	620	620
HP 14 x 73	1066[6]	1066[6]	615	615
HP 14 x 89	1301	1301	685	685
HP 14 x 117	1714	1714	805	805

Table 3 – Factored Axial Compressive Resistances for H-Piles for Extreme and Service 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 driven with a Delmag D19-42 hammer are less than the controlling factored axial structural resistance per LRFD Article 10.7.3.2.3 and the nominal structural resistances. 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 3. The maximum applied factored axial pile load for the service and extreme limit states should not exceed the governing factored pile resistance shown in Table 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.

A series of lateral pile resistance analyses should be performed by the geotechnical engineer to evaluate pile behavior at both abutments using L-Pile software with pile head deflections, moments, and axial loads supplied by the structural engineer. The designer should utilize the

[4] Normal conditions consider no soil loss due to scour. Nominal structural resistances were calculated for a braced pile segment using a resistance factor, ϕ = 1.0. Factored structural resistances should be calculated for upper and lower unbraced pile segments determined by L-Pile analyses.
[5] Based on guidance in LRFD Article 10.7.3.2.3., *Piles Driven to Hard Rock.*
[6] Does not consider resistance factors of slender elements. 12x53 and 14x73 H-pile sections may require additional reductions based upon structural performance considerations.

results of the L-Pile analyses to recalculate axial compressive structural pile resistances based on unbraced pile segments and verify pile bending stresses do not exceed allowable stresses.

Geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses should emulate the soil at the site by using the soil layers and use appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

DRIVEN PILE RESISTANCE AND PILE QUALITY CONTROL:

The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test 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. Restrikes or additional dynamic tests may be required as part of the pile field quality control program should; pile behavior vary radically between adjacent piles, pile behavior indicate a pile is refusing on a boulder or in a cobble layer above bedrock, the special pile tip be not firmly embedded in bedrock, or if the pile "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 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 could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

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 4:

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

Table 4 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The vertical profile shows the bridge to be in a sag vertical curve; therefore, abutment design shall include a drainage system behind the abutment to intercept any groundwater. Weep holes, if required, should be constructed approximately 6 inches above the riprap shelf to prevent drainage into the underside of slope protection. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.9.

Backfill within 10 feet of the abutments and side slope fill shall conform to Granular Borrow for Underwater Backfill – MaineDOT Specification 703.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

Slopes in front of the pile supported integral abutments should be set back from the riverbank and 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).

WINGWALLS:

In-line "butterfly" wingwalls, or return "U-shape" wingwalls, may be constructed monolithically with the integral abutments. The monolithic 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, collision loads, creep and temperature, and shrinkage deformations. The design of monolithic wingwalls shall account for the additional bending stresses resulting from the wingwall being cantilevered off the abutment. For monolithic U-shaped wingwalls a chamfer, typically 1 foot, should be used between the abutment and the wingwalls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection.

The design of the "butterfly" wingwalls shall at a minimum consider a load case at the service limit state 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. 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 design of U-shaped monolithic wingwalls shall at a minimum also consider a second load case where the wingwall is subjected to active pressure and to collision loads on wall mounted bridge rail under the extreme limit state. Calculation of active earth pressure shall use the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope and 0.52 for a 2H:1V backslope. See the end of this addendum for calculations and supporting documentation.
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.

There are no bearing resistance considerations or special foundation supports needed for wingwalls that are cantilevered off the abutment. However, it is recommended that the geotechnical engineer be consulted should other earth retaining systems not provided within this report be considered for design. Independently supported wingwalls that are not pile supported are required to meet the embedment requirements of Section 7.7 of Soils Report No. 2014-26.

CONSTRUCTION CONSIDERATIONS:

Construction of the abutments will require pile driving. Temporary lateral earth support systems may be required to support the existing bridge approach fills and to permit construction of driven pile foundations at the proposed abutments.

The new integral abutments will be constructed downstream of the existing abutments. There is a potential that the existing substructures or wingwalls, if not removed entirely, may impact pile driving operations. The contractor shall be responsible for excavating those portions of the existing abutments, wingwalls, and footings that conflict with piles by; conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be made incidental to related pay items. It is assumed that the existing substructures will be removed to the streambed or slightly below. Care should be taken to ensure suitable materials are not disturbed unnecessarily.

Obstructions (cobbles and boulders) were encountered in the native sand and glacial till layers. As a result, drilling equipment incurred damage while driving casing at Abutment No. 2. There is potential for these obstructions to impact construction activities. Impacts include but are not limited to impeding the driving of sheet piles for temporary earth support systems and driving H-piles for abutment foundations. Obstructions may be cleared by; conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident. Care should be taken to drive piles within allowable tolerances.

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.

A special provision for Rock Injector Pile Points, or equivalent, shall be included in contract documents and is provided at the end of this addendum to support the fixed, or pinned, pile head assumption. This item may also require detailing on construction plans and require construction notes.

CLOSURE:

This addendum has been prepared to provide H-pile resistances and geotechnical design recommendations for the design of a replacement structure for Meddybemps Bridge in Meddybemps, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

This addendum should be used as a supplement to Soils Report No. 2014-26. In the event that any changes in the nature, design, or location of the proposed project are planned, this addendum and Soils Report No. 2014-26 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. Further, the analyses and recommendations in this addendum are based in part upon limited soil explorations at discrete locations completed at the project site. If variations from the conditions encountered during the investigation appear evident during construction, it may become necessary to reevaluate the recommendations made in this addendum.

It is also recommended that the geotechnical engineer 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.

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock Injector Pile Tip)

Subsection 501.048 Prefabricated Pile Tips of the Standard Specifications is amended as follows:

Pile tips for use on all piles shall be Rock Injector HP-80500 Pile Point for 14-inch H-pile sections or APF-R-12 Pile Point for 12-inch H-pile sections, manufactured by Associated Pile and Fitting or approved equal. Material specifications, attachment of pile tips and seating of the piles shall be in accordance with Manufacturer's recommendations and in accordance with the Standard Specifications.

Payment will be made under:

Pay Item	Description	Pay Unit
501.903	Pile Tips – Rock Injector Point	Each

20506 Meddybemps Bridge #
Possible Pile Lengths
By: B.Slaven 8/2015
Check: LK 9/2015

Purpose: Investigate possible pile lengths for drivability analyses. Use length that provides lowest resistance for conservative design.

Abut. 1					
Boring/Probe	**Estimated Finish Grade EL.**	**Estimated Abut. Depth**	**Refusal El.**	**Estimated Min Pile L**	**Estimated Max Pile L**
BB-MDR-201	172.4	8-10	151.5	10.9	12.9
PA-206	172.4	8-10	150.6	11.8	13.8
PA-201	172.4	8-10	150.9	11.5	13.5
BB-MDR-101	172.4	8-10	149.5	12.9	14.9
BB-MDR-102	172.4	8-10	147.2	15.2	17.2
PA-202	172.4	8-10	151.9	10.5	12.5

Abut 1 Summary	
Min Pile L	10.5
Ave	13.1
Max Pile L	17.2

Abut. 2					
Boring/Probe	**Estimated Finish Grade EL.**	**Estimated Abut. Depth**	**Refusal El.**	**Estimated Min Pile L**	**Estimated Max Pile L**
PA-205	171.2	8-10	150.5	10.7	12.7
PA-203	171.2	8-10	149	12.2	14.2
BB-MDR-202	171.2	8-10	149.3	11.9	13.9
BB-MDR-103	171.2	8-10	147.5	13.7	15.7
BB-MDR-103A	171.2	8-10	143.9	17.3	19.3
BB-MDR-203A	171.2	8-10	143.4	17.8	19.8
PA-204	171.2	8-10	145.7	15.5	17.5

Abut 2 Summary	
Min Pile L	10.7
Ave	15.2
Max Pile L	19.8

Result: Short pile segments control. Use 11' embedment with contractor driving total length of 25'

Design of H-piles

Reference: AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014

Bedrock Properties

BB-MDR-101, R_1=53%, R_2=72%
BB-MDR-102, R_1=67%, R_2=85%
BB-MDR-103, R_1=62%
BB-MDR-103A, R_1=17%, R_2=0%, R_3=29%
BB-MDR-201, R_1=33%, R_2=56%, R_3=50%
BB-MDR-202, R_1=87%, R_2=62%
BB-MDR-203A, R_1=57%, R_2=48%

Rock Type: Diorite and Granodiorite, hard to moderately hard, fresh to very weathered, joint sets low to moderately dipping angles, closely spaced, tight to open, some silt infiling.

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
Granite C_o = 2,100 - 49,000 psi, Quartzdiorite C_o = 1,400 - 14,000 psi (AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B)

For Design Purposes, use bedrock data from BB-MDR-103A: RQD = 17% and an assumed Unconfined Compressive Strength of 13,000 psi.

Pile Properties

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

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

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

12x53
12x74
14x73
14x89
14x117

Note: All matrices set up in this order

Pile yield strength $F_y := 50 \cdot ksi$

1. Nominal and Factored Structural Compressive Resistance of H-piles

Use LRFD Equation 6.9.2.1-1 Pr = ϕ Pn

Nominal Axial Structural Resistance

Determine equivalent yield resistance Po=QF_yA_s (LRFD 6.9.4.1.1)

$Q := 1.0$ LRFD Article 6.9.4.2

$P_o := Q \cdot F_y \cdot A_s$

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Slender element reduction factor, Q, may be required to reduce resistance for 12x53 and 14x73 H-pile sections per LFRD 6.9.4.2.

Assume a 1 foot unbraced section of pile due to settlement or scour, L=1. Assume one end subject to translation but not rotation and one end rotation free but translation fixed (pinned), K=2.0,Ref. LRFD Table C4.6.2.5-1

A. Structural Resistance of unbraced segment of pile

Determine elastic critical buckling resistance P_e, 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
Design value for ideal conditions when one end rotation fixed and translation free and one end rotation free and translation fixed.

l = unbraced length $l_{unbraced_top} := 1.0 \cdot ft$

r_s= radius of gyration

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

radius of gyration about the Y-Y or weak axis per LRFD Article C6.9.4.1.2.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced_top}}{r_s} \right)^2} \cdot A_s \right]} \qquad P_e = \begin{pmatrix} 63000 \\ 92363 \\ 129521 \\ 161609 \\ 220304 \end{pmatrix} \cdot kip$$

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 81.29 \\ 84.737 \\ 121.048 \\ 123.838 \\ 128.084 \end{pmatrix}$$

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:

this applies to all pile sizes

$$P_n = \begin{pmatrix} 771 \\ 1085 \\ 1066 \\ 1301 \\ 1714 \end{pmatrix} \cdot kip$$

Factored Axial Structural Resistance for the Strength Limit State

Resistance factor for lower portion of H-pile in pure compression, severe driving conditions, per LRFD 6.5.4.2 for the case where pile tip is necessary

$\phi_c := 0.5$

The Factored Structural Resistance (P_r) per LRFD 6.9.2.1-1 is $P_r := \phi_c \cdot P_n$

Factored structural compressive resistance, P_r

$$P_r = \begin{pmatrix} 386 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip$$

LRFD 10.7.3.2.3 - Piles Driven to Hard Rock -

Article 10.7.3.2.3 states "The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage."

Therefore limit the nominal axial geotechnical pile resistance to the nominal structural resistance with a resistance factor for severe driving conditions of 0.50 applied per 10.7.3.2.3.

Nominal Structural Resistance Previously Calculated:

$$P_n = \begin{pmatrix} 771 \\ 1085 \\ 1066 \\ 1301 \\ 1714 \end{pmatrix} \cdot kip$$

The factored geotechnical compressive resistance (P_r) for the **Strength Limit State**, per LRFD 6.9.2.1-1 is

$$\phi_c := 0.5$$

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

$$\boxed{P_r = \begin{pmatrix} 386 \\ 542 \\ 533 \\ 650 \\ 857 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

The factored geotechnical compressive resistance (P_r) for the **Extreme and Service Limit States**, per LRFD 6.9.2.1-1 is

$$\phi_c := 1.0$$

$$P_{r_ee} := \phi_c \cdot P_n$$

$$\boxed{P_{r_ee} = \begin{pmatrix} 771 \\ 1085 \\ 1066 \\ 1301 \\ 1714 \end{pmatrix} \cdot kip}$$

12x53
12x74
14x73
14x89
14x117

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 5-15 blows per inch (bpi) per Section 501 (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 addendum, estimated pile lengths at Abutment 1 will be approx. 11 ft.

Assume contractor drives pile lengths of 25 ft. (5' testing + 2' cap + 7' contingency)

Use constant shaft resistances so that GRLWeap will assign 30 kip shaft resistance acting on embedded length to all ultimate capacities analyzed.

Pile Size is 12 x 53

The 12x53 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: 73% of Max (-3 Setting)

State of Maine Dept. Of Transportation
20506 Meddybemps D19-42 12x53

03-Sep-2015
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	32.99	1.46	5.2	6.03	10.85
350.0	37.15	2.21	6.5	6.32	11.35
400.0	41.39	3.71	8.0	6.72	12.13
410.0	42.17	4.01	8.3	6.79	12.26
420.0	42.91	4.30	8.7	6.86	12.38
430.0	43.63	4.57	9.1	6.93	12.51
440.0	44.34	4.82	9.5	7.00	12.63
445.0	44.67	4.93	9.7	7.03	12.72
450.0	45.02	5.04	10.0	7.06	12.75
500.0	48.42	5.97	12.5	7.44	13.49

Rounding up blow counts >9 will overstress the piles

$$R_{ndr} := 420 \cdot kip$$

Strength Limit State

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

$$R_{fdr} = 273 \cdot kip$$

Extreme and Service Limit States

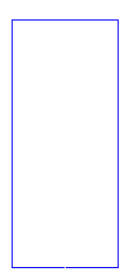
$$R_{dr} := R_{ndr} \cdot \phi$$

$$R_{dr} = 420 \cdot kip$$

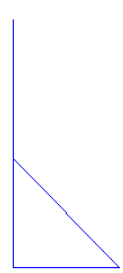
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrat	11.00	ft
Pile Top Are	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 12 x 74

The 12x74 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: 81 % of Max (-2 Setting)

State of Maine Dept. Of Transportation
20506 Meddybemps D19-42 12x74

31-Aug-2015
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	28.57	0.63	4.4	6.52	12.02
400.0	33.73	2.60	6.2	7.05	12.72
450.0	36.21	2.65	7.5	7.31	13.03
500.0	39.08	2.86	8.5	7.75	13.82
550.0	41.24	3.13	10.3	8.01	14.17
600.0	43.57	3.45	12.2	8.37	14.83
610.0	44.03	3.60	12.6	8.45	14.95
620.0	44.49	3.82	13.0	8.53	15.10
630.0	44.99	4.06	13.4	8.62	15.27
640.0	45.45	4.29	13.8	8.70	15.41

Limit driving stress < 45 ksi stress

$R_{ndr} := 620 \cdot kip$

Strength Limit State

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

$R_{fdr} = 403 \cdot kip$

Extreme and Service Limit States

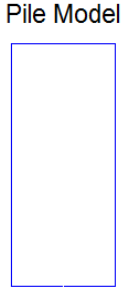
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 620 \cdot kip$

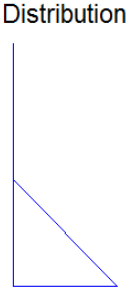
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrat	11.00	ft
Pile Top Are	21.80	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 73

The 14x73 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: 81% of Max (-2 setting)

State of Maine Dept. Of Transportation
20506 Meddybemps D19-42 14x73

31-Aug-2015
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	28.90	0.62	4.4	6.51	12.04
400.0	34.07	2.75	6.3	7.06	12.75
450.0	36.61	2.84	7.5	7.33	13.12
500.0	39.26	2.99	8.7	7.70	13.73
550.0	41.69	3.17	10.3	8.04	14.27
600.0	44.03	3.29	12.3	8.40	14.91
610.0	44.49	3.38	12.7	8.48	15.04
615.0	44.74	3.43	12.8	8.53	15.15
620.0	44.96	3.52	13.1	8.56	15.20
630.0	45.45	3.73	13.4	8.65	15.38

Blow counts >13 bpi result in a driving stresses that exceed 45 ksi. Use bpi of 13.

$R_{ndr} := 615 \cdot kip$

Strength Limit State

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

$R_{fdr} = 400 \cdot kip$

Extreme and Service Limit States

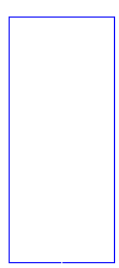
$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 615 \cdot kip$

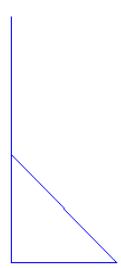
DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrat	11.00	ft
Pile Top Are	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 89

The 14x89 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: 90% of Max (-1 Setting)

State of Maine Dept. Of Transportation
20506 Meddybemps D19-42 14x89

31-Aug-2015
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	27.14	0.48	3.9	7.17	13.87
500.0	36.83	2.26	7.2	8.19	14.85
600.0	41.24	4.17	9.5	8.83	15.87
620.0	42.08	4.42	10.1	8.96	16.08
640.0	42.92	4.59	10.7	9.10	16.28
660.0	43.87	4.72	11.2	9.28	16.65
680.0	44.71	4.79	11.8	9.44	16.92
685.0	44.94	4.76	12.0	9.48	16.97
690.0	45.16	4.79	12.1	9.52	17.06
700.0	45.55	4.76	12.4	9.59	17.21

DELMAG D 19-42

Limit driving stress to 45 ksi

$R_{ndr} := 685 \cdot kip$

Strength Limit State

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

$R_{fdr} = 445 \cdot kip$

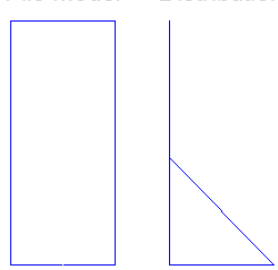
Extreme and Service Limit States

$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 685 \cdot kip$

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrat	11.00	ft
Pile Top Are	26.10	in2

Pile Model | Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Pile Size is 14 x 117

The 14x117 pile can be driven to the resistances below with a D 19-42 hammer and 2.7 kip helmet at a reasonable blow count and level of driving stress. See GRLWEAP results below: Fuel Setting: Max

State of Maine Dept. Of Transportation
20506 Meddybemps D19-42 14x117

31-Aug-2015
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	24.14	0.15	3.6	7.97	16.07
600.0	36.96	2.12	8.0	9.39	17.53
700.0	40.64	2.12	10.1	9.98	18.89
750.0	42.29	1.82	11.4	10.26	19.52
800.0	43.94	1.61	12.7	10.55	20.20
805.0	44.04	1.77	13.0	10.55	20.17
820.0	44.50	1.91	13.4	10.63	20.37
840.0	45.28	2.35	13.8	10.81	20.79
860.0	45.60	2.76	14.8	10.81	20.77
880.0	45.94	3.16	15.8	10.81	20.78

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	kips
Hammer C	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Dampin	0.050	sec/ft
Toe Dampin	0.150	sec/ft
Pile Length	25.00	ft
Pile Penetrat	11.00	ft
Pile Top Are	34.40	in2

Rounding up blow counts >13 overstresses piles

$R_{ndr} := 805 \cdot kip$

Strength Limit State

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

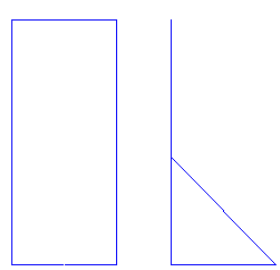
$R_{fdr} = 523 \cdot kip$

Extreme and Service Limit States

$R_{dr} := R_{ndr} \cdot \phi$

$R_{dr} = 805 \cdot kip$

Pile Model | Skin Friction Distribution



Res. Shaft = 10 %
(Constant Res. Shaft)

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight	$\gamma_1 := 125 \cdot pcf$
Internal friction angle	$\phi_1 := 32 \cdot deg$
Cohesion	$c_1 := 0 \cdot psf$

U-shaped and Butterfly Wingwalls - At-Rest

At-Rest Earth Pressure - Rankine Theory

Reference: LRFD 3.11.5.2 For walls less than 5 feet, or braced stem walls that prevent rotation.

$K_o := 1 - \sin(\phi_1)$

$K_o = 0.47$

U-shaped Wingwalls - Active Earth Pressure

Active pressure acting parallel to the travelway is assumed to be resisted by the superstructure and can be neglected for abutments and butterfly walls. Design of U-shaped wingwalls shall consider active pressure acting perpendicular to the travelway.

Active Earth Pressure - Rankine Theory

Rankine shall be used for walls where the failure surface is uninterrupted by the top of the wall stem (See LFRD C3.11.5.3-1). The lateral earth pressure is assumed to act at a height of H/3 above the base of the wall. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot deg - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill (2H:1V)

β = Angle of fill slope to the horizontal

$\beta := 26.6 \cdot deg$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.519$$

- Pa is oriented at an angle of β to the vertical plane

Integral Abutment - Passive Earth Pressure - Coulomb Theory

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

$\phi_1 := 32 \cdot deg$

θ = Angle of back face of wall to the horizontal $\theta := 90 \cdot deg$

For cases where interface friction is considered (such as gravity, semi gravity, and prefab modular walls with steep backfaces where the sliding surface is restricted by the top of the wall), use Coulomb.

For IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta := 19.5 \cdot deg$

$$K_{p_coul} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2}$$

$K_{p_coul} = 6.73$

U-shaped and Butterfly Wingwall - Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when β>0.

β = Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

$$K_{p_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}$$

$K_{p_rank} = 3.255$

Pp is oriented at an angle of β to the vertical plane

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

MEDDYBEMPS BRIDGE
ROUTE 191 OVER DENNY'S RIVER
MEDDYBEMPS, MAINE

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Washington County
WIN 20506.00

Soils Report No. 2014-26
Bridge No. 3736

Fed No. STP-20506(600)
November 26, 2014

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and geotechnical design recommendations for the replacement of Meddybemps Bridge which carries State Route 191 over Denny's River in Meddybemps, Maine. The proposed bridge will be a 40-foot simple-span precast, prestressed concrete voided slab superstructure on cantilever semi-gravity abutments founded on spread footings on bedrock. Wingwalls will consist of cantilever, semi-gravity return walls constructed on spread footings founded on bedrock. The bridge will be replaced on a new alignment located approximately 45 feet downstream of the existing bridge centerline. The following design recommendations are discussed in detail in this report:

General - Spread Footings on Bedrock. Based on test borings and power auger probes conducted at the proposed abutments and wingwalls, the recommended bedrock bearing elevations for design range from Elevation 147 to 152 at Abutment No. 1 and Elevation 143 to 150 at Abutment No. 2. It is essential to note that the top of intact rock cannot be known for the entire foundation area prior to excavation. Intact rock may be encountered above and below the recommended design bearing elevations provided in this report. The thickness of the abutment and wingwall footings may be designed to vary in thickness, or the top elevation of the footing may be stepped and the stem walls shortened to accommodate variations in the bedrock surface.

Design of Cantilever Semi-gravity Abutments and Wingwalls - Abutments and wingwalls shall be designed to resist all applicable load combinations and for all relevant strength, service and extreme limit states. The design of abutments and walls founded on spread footings at the strength and extreme limit states shall consider nominal bearing resistance, eccentricity, failure by sliding and structural failure. Service limit state design shall consider settlement, horizontal movement, bearing resistance, sliding and eccentricity. Shear failure along adversely oriented joint surfaces in the rock mass below the foundations is not anticipated, therefore a global stability evaluation at the service limit state may be waived.

Earth Pressures and Surcharge Forces - Earth loads shall be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory. Use soil properties for the structural backfill of ϕ = 32 degrees, γ = 125 pounds per cubic foot (pcf). Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. If a structural approach slab is specified, some reduction of surcharge loads is permitted.

Bearing Resistance - The factored bearing pressure at the strength limit state for spread footings on sound bedrock should not exceed a factored bearing resistance of 29 kips per square foot (ksf) at Abutment No. 1 and 20 ksf at Abutment No. 2. Based on presumptive bearing resistance values, a factored bearing resistance of 24 ksf may be used when analyzing the service limit state to control settlement. For extreme limit state load combinations factored bearing resistances of 52 ksf and 35 ksf may be used at Abutment No. 1 and No. 2, respectively.

Scour and Riprap - For the scour protection of abutments and wingwalls constructed on spread footings, cast footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock.

Riprap slope protection for slopes in front of abutments and wingwalls subject to hydraulic events shall be armored with 3 feet of plain riprap or 4 feet of heavy riprap in accordance the requirements of the MaineDOT Bridge Design Guide (BDG). Riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1-foot thick layer of bedding material.

Approach Embankment Considerations and Settlement – Approach fills of up to 15 feet are proposed above the existing grades immediately behind the proposed abutments. The maximum side slopes should be 2 horizontal to 1 vertical (2H:1V) or flatter. Steeper slopes of 1.75H:1V may be utilized in conjunction with riprap protection. Post-construction settlement of the foundation soils due to the proposed approach fills are anticipated to be negligible. Any settlement of bridge foundations will be due to elastic compression of the bedrock mass and is estimated to be 0.5-inch or less.

Frost Protection - Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for footings cast directly on sound bedrock. Any foundations placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

Seismic Design Considerations – Seismic analysis is not required for single-span bridges, regardless of seismic zone. However, superstructure connections and minimum bridge seat dimensions shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

Construction Considerations – Cofferdams to control stream flow and earth support systems to support the existing bridge approach fills may be required for the construction of spread footings. Construction activities will also include common and structural earth and rock excavation. Cobbles and boulders were encountered in the explorations at the locations of the proposed abutments and walls. Clearing cobbles and boulders shall be specified as incidental to related pay items.

Preparation of the bedrock subgrade for foundations may require excavation of bedrock to create level benches or flatten bedrock surfaces with existing slopes steeper than 4H:1V. All loose bedrock and soil debris should be removed from the final bedrock surface before tremie-seal or footing concrete is placed for abutment and wingwall foundations.

The cleanliness and condition of the bedrock surface should be confirmed and approved by the Resident prior to placing concrete.

Excavation of bedrock may be conducted using conventional equipment (clam buckets, excavators, hoe rams) but may require drilling and blasting methods. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre- and post-blast surveys, as well as blast

vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

In general, do not use excavated fill or native soils for fill anywhere beneath the new pavement structure, for dressing slopes or for new backfill. If the contractor wishes to reuse excavated granular material as embankment fill, it is recommended that the materials be stockpiled and tested for meeting MaineDOT requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Meddybemps Bridge which carries State Route 191 over Denny's River in Meddybemps, Maine. Two subsurface investigations have been completed at this site. The purpose of the investigations was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soil and bedrock information obtained at the site during the subsurface investigations, foundation recommendations and geotechnical design parameters for foundation design.

Meddybemps Bridge was built in 1940 and is a 25-foot reinforced concrete rigid frame bridge on a 15° skew. The historical bridge plans indicate the rigid frame walls are founded on strip footings cast in shallow keyways excavated in bedrock. The 2013 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports assign the substructures a condition rating of 6 – satisfactory, the deck and superstructure a condition rating of 4 – poor, and the bridge a Sufficiency Rating of 11.1. The structure has a channel protection rating of 6 – bank slumping. The structure exhibits areas of moderate to heavy deterioration, with significant cracking and efflorescence on the bottom of the superstructure slab and the south fascia. The wingwalls have moderate cracking; cracking is worse on the downstream wingwalls. The structure is classified as structurally deficient due to the poor condition of the deck and superstructure and is in need of complete replacement.

The project site is adjacent to the Eastern Surplus Company Superfund site which is on the north side of the bridge along Route 191. The Maine Department of Environmental Protection (MaineDEP) has installed monitoring wells for the superfund site on all corners of the bridge site. As part of the MaineDEP site clean-up, soils on the north side of Route 191 were removed and replaced. No soil contamination was identified on the south side of Route 191. Minor groundwater contamination has been found at the site. The MaineDEP investigation is ongoing.

The MaineDOT Bridge Program has identified the preferred replacement bridge alternative to be a 40-foot span precast, prestressed concrete voided slab superstructure with a 20° skew on cantilever semi-gravity abutments founded on spread footings on bedrock. Wingwalls will consist of cantilever, semi-gravity return wingwalls on the upstream side and flared wingwalls on the downstream side, all constructed on spread footings cast directly on bedrock. The bridge will be replaced on a new alignment located approximately 45 feet downstream of the existing bridge centerline. The new alignment will require approximately 350 feet of approach work on each end to match into the existing tangents. Bridge approach embankments up to approximately 15 feet high will be constructed at each approach. Traffic will be maintained on the existing bridge while the new bridge is constructed on the new alignment.

2.0 GEOLOGIC SETTING

Meddybemps Bridge on State Route 191 in Meddybemps, Maine, crosses Denny's River as shown on Sheet 1 - Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the Calais Quadrangle, Maine, Open-File No. 82-1 (1982) indicates that the surficial soils at the bridge site are glacial marine deposits, locally known as the Presumpscot Formation, in contact with glacial till. The glacial marine deposit is comprised of clay and silt that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor. The deposit was formed during late-glacial time, when the relative sea level was higher than at present. Glacial till is a heterogeneous mixture of sand, silt, clay and stones, and includes two varieties: basal till and ablation till. The till unit generally overlies bedrock.

According to the Bedrock Geologic Map of Maine, (1985) the bedrock at the site is identified as Devonian gabbro, diorite and ultramafic rocks. The Bedrock Geology map for the Calais Quadrangle, Maine, MGS, Open-File 90-27 (1990) cites the bedrock at the bridge location as a gabbro-diorite intrusive complex with contacts to Meddybemps granite.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling eight (8) test borings and six (6) power auger probes. Seven (7) of the eight (8) borings were advanced to bedrock and were terminated with bedrock cores. Borings BB-MDR-101, BB-MDR-102 and BB-MDR-201 were drilled at the approximate location of proposed Abutment No. 1 and its wingwalls. Borings drilled at proposed Abutment No. 1 were supplemented with power auger probes PA-201, PA-202 and PA-206. Borings BB-MDR-103, BB-MDR-103A, BB-MDR-202, BB-MDR-203 and BB-MDR-203A and power auger probes PA-203, PA-204 and PA-205 were drilled within the footprint of proposed Abutment No. 2 and its wingwalls.

The boring and probe locations and an interpretive subsurface profile across the site are shown on Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile.

The 100-series borings were drilled between January 13 and 15, 2014 by Northern Test Boring (NTB), Inc. of Gorham, Maine using a track mounted drill rig. The 200-series borings and power augers with drilled between July 14 and 16, 2014 also by NTB with a track mounted drill rig. The probes were drilled using solid stem auger techniques and were taken to refusal of the drilling tools. No soil samples were taken in the probes. All the borings were drilled using solid stem auger, cased wash boring and rock coring 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 NTB drill rig used for the exploration programs is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633-05 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in July 2013. The Diedrick D-50 hammer was found to deliver

approximately 34 percent more energy during driving than the standard rope and cathead system. The N-values discussed in this report for the NTB drill rig are corrected values computed by applying an average energy transfer of 0.801 to the raw field N-values. This hammer efficiency factor (0.801) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Where bedrock was encountered, the bedrock was cored 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, and identified field and laboratory testing requirements and reviewed field logs for accuracy. A MaineDOT Subsurface Inspector certified by the Northeast Transportation Technical Certification Program (NETTCP) logged the subsurface conditions encountered in the borings. The elevations and coordinates of the as-drilled borings were located after completion of the drilling program by MaineDOT Survey.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on the boring logs provided in Appendix A – Boring Logs and Power Auger Probes and graphically on Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile.

4.0 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.

Laboratory testing consisted of four (4) standard grain size analyses with natural water content and one (1) grain size analysis with hydrometer and natural water content. 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 and Power Auger Probes, and on Sheets 3 and 4 - Boring Logs.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the proposed bridge location consisted of topsoil, glaciomarine deposits, native sands, gravels and silts, with cobbles and boulders, and glacial till, all overlying igneous bedrock. The glaciomarine deposit was absent from the borings completed on the east side of Denny's River. Boulders and cobbles also dominated the soil units on the east side of the river.

An interpretive subsurface profile depicting the generalized soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile. The boring logs are provided in Appendix A – Boring Logs and Power Auger Probes, and on Sheets 3 and 4 – Boring Logs. A summary description of the strata encountered at each proposed substructure follows:

5.1 Abutment No. 1

At proposed Abutment No. 1, topsoil, glaciomarine deposits, native soils with cobbles and boulders, and glacial till were encountered overlying bedrock.

Topsoil. The thickness of the topsoil ranged from approximately 0.5 to 3.5 feet thick at the boring locations. The topsoil consisted of brown, damp to wet, fine to coarse sand, some silt, little gravel, trace of roots and organics and silty fine to medium sand, trace gravel. Corrected SPT V-values in the topsoil ranged from 5 to 17 blows per foot (bpf) indicating that the soil is loose to medium dense in consistency.

Glaciomarine Deposits. Weathered glaciomarine silt was encountered below the topsoil in boring BB-MDR-201. The silt deposit was approximately 5 feet thick in the boring. The deposit is characterized as olive brown, damp, weathered, silt, some fine to medium sand, trace gravel. One SPT N-value in the silt was 13 bpf, indicated the deposit is stiff in consistency. One (1) grain size analysis resulted in the silt being classified as A-4 under the AASHTO Soil Classification System and ML under the Unified Soil Classification System (USCS). The measured water content of the sample tested was approximately 19 percent.

Native Sands with Cobbles. Native sands were encountered below the topsoil in BB-MDR-101 and BB-MDR-102. The thickness of the layer ranged from approximately 7.1 to 8.9 feet at the boring locations and consisted of variable amounts of sand, silt, and gravel with frequent cobbles. Corrected SPT N-values in the layer ranged from 25 to greater than 50 bpf indicating the soil is medium dense to very dense in consistency.

Two (2) grain size analyses resulted in the sand layer being classified as A-2-4 and A-1-b 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 were approximately 15 and 18 percent.

Glacial Till. Glacial till was encountered below the glaciomarine silt in boring BB-MDR-201. The thickness of the glacial till encountered was approximately 2.7 feet in the boring. The deposit consisted of grey-brown, wet, gravelly, fine to coarse sand, little silt, with cobbles.

5.2 Abutment No. 2

At proposed Abutment No. 2, topsoil, native soils with cobbles and boulders, and glacial till were encountered overlying bedrock.

Topsoil. A layer of topsoil was encountered in boring BB-MDR-103. The thickness of the topsoil encountered was approximately 2.0 feet thick at the boring location. The topsoil consisted of dark brown, saturated, fine to coarse sand, little gravel, little silt, trace organics. One (1) SPT N-value in the topsoil was 5 bpf indicating that the soil is loose in consistency.

Native Sands with Cobbles and Boulders. A native sand deposit was encountered in all the borings at Abutment No. 2. The thickness of the sand deposit ranged from approximately 7.5 to 15.2 feet thick at the boring locations. The layer consisted of dark brown or brown, moist to wet, fine to coarse sand, little silt, little gravel and brown, moist to wet, gravelly, fine to coarse sand, little to trace silt. Isolated cobbles and nests of cobbles were encountered in all the borings. Boulders, defined as rocks greater than 12 inches in diameter, were encountered frequently from the ground surface to the bedrock surface in borings BB-MDR-203 and BB-MDR-203A.

Corrected SPT N-values in the deposit ranged from 5 to greater than 50 bpf indicating that the soil is loose to very dense in consistency. One (1) grain size analysis resulted in the soil being classified as A-4 under the AASHTO soil classification system and ML under the USCS. The measured water content of the sample tested was approximately 19 percent.

Glacial Till. Glacial till was encountered directly overlying bedrock in borings BB-MDR-103A and BB-MDR-203. The thickness of the till was approximately 0.8 to 5.5 feet at the boring locations. The deposit consisted of grey, wet, gravelly, fine to coarse sand, little silt. One (1) SPT N-value in the deposit was 67 bpf indicating that the soil is very dense in consistency. One (1) grain size analysis resulted in the soil being classified as A-1-b under the AASHTO Soil Classification System and SM under the USCS. The measured water content of the sample tested was approximately 8 percent.

5.3 Bedrock

Bedrock was encountered and cored in the three (3) borings drilled at Abutment No. 1 at depths ranging from approximately 9.1 to 11.8 feet bgs. Bedrock was encountered and cored in four (4) of six (6) borings drilled at Abutment No. 2 at depths ranging from approximately 9.5 to 15.2 feet bgs. Table 1 summarizes approximate depths to bedrock, corresponding approximate bedrock surface elevations and RQD at the boring locations.

Proposed Foundation	Boring	Approx. Depth to Bedrock (feet)	Approx. Elevation of Bedrock Surface (feet)	RQD
Abutment 1 - Right	BB-MDR-101	9.4	149.5	53 to 72%
Abutment 1 - Right	BB-MDR-102	9.1	147.2	67 to 85%
Abutment 1 - Left Wingwall	BB-MDR-201	11.8	151.5	33 to 56%
Abutment 2 - center	BB-MDR-103	9.5	147.5	62%
Abutment 2 – center	BB-MDR-103A	12.3	143.9	0 to 29%
Abutment 2 – center	BB-MDR-202	12.3	149.3	62 to 87%
Abutment 2 - right	BB-MDR-203A	15.2	143.4	48 to 57%

Table 1 - Summary of Approximate Bedrock Elevations and RQD

The bedrock at proposed Abutment No. 1 is identified as dark grey, fine to medium grained, diorite, hard, slightly weathered to fresh, with joint sets at low to moderately dipping angles, closely spaced and tight. The RQD of the bedrock was determined to range from 33 to 85% which correlates to a Rock Mass Quality of poor to good.

The bedrock at proposed Abutment No. 2 is identified as grey, coarse grained, granodiorite, hard to moderately hard, fresh to very weathered, with fractured zones, with joint sets at low to moderately dipping angles, closely spaced, open to tight, slightly weathered surfaces and some with silt infilling, and grey, fine to medium grained, diorite, fresh to very slightly weathered, joint set at low angles to horizontal, moderately closely spaced, tight. The RQD of the bedrock was determined to range from 0 to 87% which correlates to a Rock Mass Quality of very poor to good.

A series of six (6) probes were drilled to better define the approximate bedrock surface. Probes PA-203, PA-206 and PA-207 were drilled through the existing bridge approaches and probes PA-201, PA-202 and PA-204 were drilled on the slopes downstream of the bridge. The probes were taken to refusal depths ranging from approximately 6.2 to 19.4 feet below the ground surface. The power auger information is presented in the Power Auger Summary Sheet provided in Appendix A – Boring Logs and Power Auger Probes.

5.4 Groundwater

The groundwater levels observed in the borings drilled ranged from approximately 3.6 to 9.0 feet bgs. Water levels were measured upon completion of drilling and are indicated on the boring logs in Appendix A. Note that water was introduced into the borings during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Groundwater levels will fluctuate with changes in the water levels in the river, precipitation, seasonal changes, runoff, and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

Assessment of subsurface conditions indicates that due to the presence of shallow bedrock at the site the most effective foundation systems for this site are spread footings bearing on bedrock, or pile-supported integral abutments with the piles installed in bedrock sockets due to the limited overburden. Two replacement alternatives were considered during preliminary design: a 40-foot simple span, voided slab bridge with cantilever abutments on spread footings bearing on bedrock and a 70-foot single span, integral bridge on pile-supported integral abutments with piles installed in bedrock sockets. Upon evaluation of these two bridge alternatives and two alignment alternatives, a 40-foot span simple span bridge on full height, cantilever semi-gravity abutments bearing on spread footings on bedrock, on a new alignment downstream of the existing bridge was selected. The selected alignment will allow complete construction of the new bridge without affecting the use of the existing bridge, and will require 560 feet of approach work with 50-foot transitions into Route 191 on either side.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections provide foundation considerations and recommendations for cantilever semi-gravity abutments and wingwalls founded on spread footings on bedrock.

7.1 General Considerations for Spread Footings on Bedrock

Bedrock subgrade preparation should call for the bedrock surface to be cleared of any loose and highly fractured bedrock encountered during construction. The nature, slope and degree of fracturing in the bedrock bearing surface will not be evident until the foundation excavations for the abutments and wingwalls are made.

It is essential to note that the top of intact rock cannot be known for the entire foundation area prior to excavation during construction. Intact rock may be encountered below and above the recommended design bearing elevations.

It is likely that the degree of weathering and fracturing of the bedrock could vary over short distances, which could result in an irregular intact rock surface. The thickness of the abutment and wingwall footings may be designed to vary in thickness, or the top elevation of the footing may be stepped and the stem walls shortened to accommodate variations in the bedrock surface, subject to review and approval by the Designer.

7.1.1. Abutment No. 1

Bedrock at proposed Abutment No. 1 was encountered in borings BB-MDR-101, BB-MDR-102 and BB-MDR-201 at Elevations ranging from approximately 147.2 to 151.5. The borings indicate that igneous bedrock with a minimum RQD of approximately 50 percent would be encountered at the bedrock surface. However, up to a foot of loose, highly fractured rock may be encountered during construction.

Power auger probes PA-201, PA-202, PA-206 were also drilled in the footprint of proposed Abutment No. 1 and its wingwalls and terminated in refusal at Elevations ranging from approximately 150.6 to 151.9. The refusal surface is interpreted to be the bedrock surface. Table 2 below summarizes the interpreted bedrock surface based on the power auger probes and borings at proposed Abutment No. 1:

Foundation Element	Power Auger or Boring Designation	Approximate Elevation of Power Auger Refusal[1] (feet)	Approximate Elevation of Bedrock Surface based on Rock Cores (feet)
Abutment 1 – Left Wingwall	BB-MDR-201		151.5
	PA-206	150.6	
Abutment 1	PA-201	150.9	
	BB-MDR-101		149.5
	BB-MDR-102		147.2
Abutment 1 – Right Wingwall	PA-202	151.9	

[1] Assumed bedrock surface.

Table 2 – Approximate Bedrock Surface Elevation Data at Abutment No. 1

Based on the data obtained at proposed Abutment No. 1 the recommended bedrock bearing elevation range for design is Elevation 147 to 152, depending on the specific location within the foundation.

7.1.2. Abutment No. 2

Bedrock at proposed Abutment No. 2 was encountered in borings BB-MDR-103, BB-MDR-103A and BB-MDR-202 and BB-MDR-203A at Elevations ranging from approximately 143.4 to 149.3. The borings indicate that igneous bedrock with a minimum RQD of approximately 50 percent should be encountered at most of the bedrock surface. However, one bedrock core at the center of proposed Abutment No. 2 was determined to have an RQD value of 0%; therefore localized areas or seams of highly fractured bedrock may be encountered during construction.

Power augers PA-203, PA-204, and PA-205 were also drilled in the footprint of the proposed Abutment No. 2 and its wingwalls. The refusal surface is interpreted to be the bedrock surface. Table 3 below summarizes the interpreted bedrock surface based on the power auger probes and borings conducted at proposed Abutment No. 2:

Foundation Element	Power Auger or Boring Designation	Approximate Elevation of Power Auger Refusal[1] (feet)	Approximate Elevation of Bedrock Surface based on Rock Cores (feet)
Abutment 2 – Left Wingwall	PA-205	150.5	
	PA-203	149.0	
Abutment 2	BB-MDR-202		149.3
	BB-MDR-103		147.5
	BB-MDR-103A		143.9[2]
	BB-MDR-203A		143.4
Abutment 2 – Right Wingwall	PA-204	145.7	

[1] Assumed bedrock surface.
[2] Very poor quality rock with RQD ranging from 0 to 29% was encountered below the bedrock surface. Excavation of this material to expose intact rock will be required during construction.

Table 3 – Approximate Bedrock Surface Elevation Data at Abutment No. 2

Based on the data obtained at proposed Abutment No. 2 the recommended bedrock bearing elevation range for design is Elevation 143 to 150, depending on the specific area within the foundation.

7.2 Design of Cantilever Semi-Gravity Abutments and Wingwalls

The proposed abutments and wingwalls shall be proportioned for all applicable load combinations specified in AASHTO LRFD Bridge Design Specifications, Seventh Edition, 2014, (herein referred to as LRFD) Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, service and extreme limit states.

The design of abutments and wingwalls founded on spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity,
- failure by sliding,
- reinforced concrete structural failure.

For footings cast directly on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions, in either direction. This eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the footing width or length.

For the service limit state, a resistance factor, φ,of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, bearing resistance, sliding and eccentricity. The overall global stability of foundations is typically investigated at the Service I Load

Combination and a resistance factor, φ, of 0.65. Shear failure along adversely oriented joint surfaces in the rock mass below the foundations is not anticipated, and therefore a global stability evaluation may be waived.

Extreme limit state design checks shall include bearing resistance, eccentricity, failure by sliding and structural failure. Extreme limit states are those load combinations relating to certain hydraulic events, ice (if warranted by ice history or stream constriction by the abutment) and seismic forces. Resistance factors, φ, for the extreme event limit state shall be taken as 1.0 with the exception of bearing resistance for which a resistance factor of 0.8 shall be used.

For the scour protection of abutment and wingwall spread footings, construct footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock. With these precautions, strength and extreme event limit state designs do not need to consider rock scour due to abrasion or plucking due to the design or check floods for scour.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings or concrete seals on bedrock assuming the bedrock surface will be prepared in the wet and some amount of sediment will remain on the bedrock surface. A sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock if the bedrock surface is prepared in the dry and cleaned with high pressure water and air prior to placing footing concrete. LRFD Table 11.5.7-1 allows a sliding resistance factor, φ_τ, of 1.0 for semi-gravity retaining walls regardless of subgrade material.

When the rock subgrade is prepared in the wet, some amount of sediment is expected to remain on the rock surface and the sliding computations for resistance of abutment and wingwall spread footings shall assume a maximum frictional coefficient of 0.60 at the bedrock-concrete interface. If the bedrock is prepared in the dry and the surface cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of abutment and wingwall footing to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface. Anchorage of the abutment footings to bedrock may be required to resist sliding forces and improve stability.

If during construction bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. An alternative approach is to design reinforcing dowels to anchor the footing to the sloping bedrock.

7.3 Earth Pressures and Surcharge Forces

Cantilever semi-gravity abutments should be designed for active earth pressure over the abutment height. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.31 is recommended assuming level backfill. Earth loads for wingwalls shall also be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory and assuming a level backfill. Supporting calculations are provided in Appendix C – Calculations.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG)) Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 4 below:

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

Table 4 - Equivalent Height of Soil for Estimating Live Load Surcharge Of Abutments Perpendicular to Traffic

Abutment and wingwall designs shall include a drainage system to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 7 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.4 Bearing Resistance

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.6. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2.

The bearing resistance for project abutments and wingwall footings founded on hard, sound bedrock shall be investigated at the strength limit state using factored loads and the factored bearing resistances provided in Table 5, below. These calculated bearing resistances assume a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. Factored bearing resistances for the extreme limit state are also provided in Table 5. A factored bearing resistance of 24 ksf may be used when analyzing the service limit state load combination to control settlements. Supporting calculations are provided in Appendix C - Calculations.

	Factored Bearing Resistance (ksf)		
	Strength Limit State φ_b=0.45	Service Limit State φ_b=1.0	Extreme Limit State φ_b=0.8
Abutment No. 1 and Wingwalls	29	24	52
Abutment No. 2 and Wingwalls	20	24	35

Table 5. Factored Bearing Resistances for Abutments No. 1 and 2

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as 0.3 *f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

7.5 Scour and Riprap

For the scour protection of abutment and wingwall spread footings bearing on bedrock, cast footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible rock.

Riprap slope protection for slopes in front of abutments and wingwalls subject to hydraulic events shall be armored with 3 feet of plain riprap or 4 feet of heavy riprap in accordance the requirements of the MaineDOT BDG. Stone riprap shall conform to Supplemental Standard Provisions 703.26 Plain and Hand Laid Riprap and 703.28 Heavy Riprap and be placed at a maximum slope of 1.75H:1V. The toe of riprap sections shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. 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.

7.6 Approach Embankment Considerations and Settlement

To construct the proposed bridge on the new alignment, approach fills up to 15 feet above existing grades will be required immediately behind the proposed abutments. The maximum side slope should be 2H:1V or flatter. Steeper slopes of 1.75H:1V may be utilized in conjunction with riprap protection. Subsurface conditions at both approaches include loose to very dense granular soils and stiff glacial marine soils overlying bedrock. These existing materials are expected to compress as the proposed embankment fill is placed. Consequently, post-construction settlements are anticipated to be negligible.

Earth fill embankments that will be constructed per MaineDOT Standard Specifications and Standard Details, using engineered fill over relatively firm overburden soils. Experience has shown that embankments constructed in the manner over relatively dense soils meet minimum required safety factors for global stability.

Bridge foundations bearing on bedrock with an RMR-based quality of Fair or better will experience 0.5-inch or less of elastic settlement as indicated in LRFD Article 10.6.2.4.4.

7.7 Frost Protection

Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for footings cast directly on sound bedrock.

Any foundations placed on granular fill or native soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Meddybemps has a design freezing index of approximately 1450 F-degree days. An assumed water content of 10% was used for native fill soils above the water table. These components correlate to a frost depth of 6.7 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). Meddybemps was assigned a design freezing index of approximately 1489 F-degree days based on the ModBerg weather database information for Ellsworth, which is on a DFI contour similar to Meddybemps. An assumed water content of 10% was used for granular soils above the water table. These components correlate to a frost depth of 5.5 feet. We recommend that foundations constructed within granular native soils be founded a minimum of 6.0 feet below finished exterior grade for frost protection. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection. Supporting calculations are provided in Appendix C – Calculations.

7.8 Seismic Design Considerations

At the proposed abutment locations, the average SPT N-values from the soil materials encountered in the borings were considered in the determination of Seismic Site Class in accordance with Method B presented in LRFD Table C3.10.3.1-1. LRFD allows the assumption that bedrock in the upper 100 feet of the profile has an N-value equal to 100 bpf. However, the SPT N-values used to determine the site class was conservatively evaluated by

including only the blow counts and thickness of soil above bedrock, reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet. Based on this approach, the bridge site is assigned to Site Class D. Supporting calculations are provided in Appendix C - Calculations.

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) = 0.085g
- Site Class D (based on an average N-value for the upper 100 ft of the soil profile greater than 15 bpf and less than 50 bpf, using steps in LRFD Table C3.10.3.1-1)
- Acceleration coefficient (As) = 0.136g
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.264g
- Design spectral acceleration coefficient at 1.0-second period, (S_{D1})= 0.100g
- Seismic Zone 1, based on a $S_{D1} \leq 0.15$g

According to LRFD Article 4.7.4.2. no seismic analysis is required for single-span bridges, regardless of seismic zone. LRFD Article C3.10.9.1 further indicates that single-span bridges are not required to include acceleration-augmented soil pressures for design. However, superstructure connections and minimum bridge seat dimensions shall be designed to meet the requirements of LRFD Articles 3.10.9 and 4.7.4.4, respectively.

7.9 Construction Considerations

Construction activities may include construction of cofferdams and earth support systems to support the existing bridge approach fills and control river flow during construction of abutment and wingwall footings. Construction activities will include common and structural earth and rock excavation.

The contractor should maintain the abutment and wingwall excavations so that the foundations can be constructed in the dry. Preparation of the bedrock subgrade for all foundations may require excavation of bedrock to create level benches or flatten bedrock surfaces with existing slopes steeper than 4H:1V. All loose bedrock and soil debris should be removed from bearing surfaces and the final bedrock surface washed with high-pressure water and air before concrete is placed for the abutment and wingwall foundations. It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that abutment foundations are constructed in the dry.

Foundation subgrades should be confirmed to be relatively level. Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.

The cleanliness and overall condition of the final bedrock surface for all spread footings and tremie-seals shall be confirmed and approved by the Resident prior to placement of the footing concrete.

Concrete for subfootings and footings should consist of Class A Concrete in accordance with Maine Standard Specifications 502.05.

In the dry or underwater excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre- and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

Cobbles and boulders were encountered in the explorations at the location of proposed abutments and wingwalls. Clearing obstructions shall be specified as incidental to related pay items.

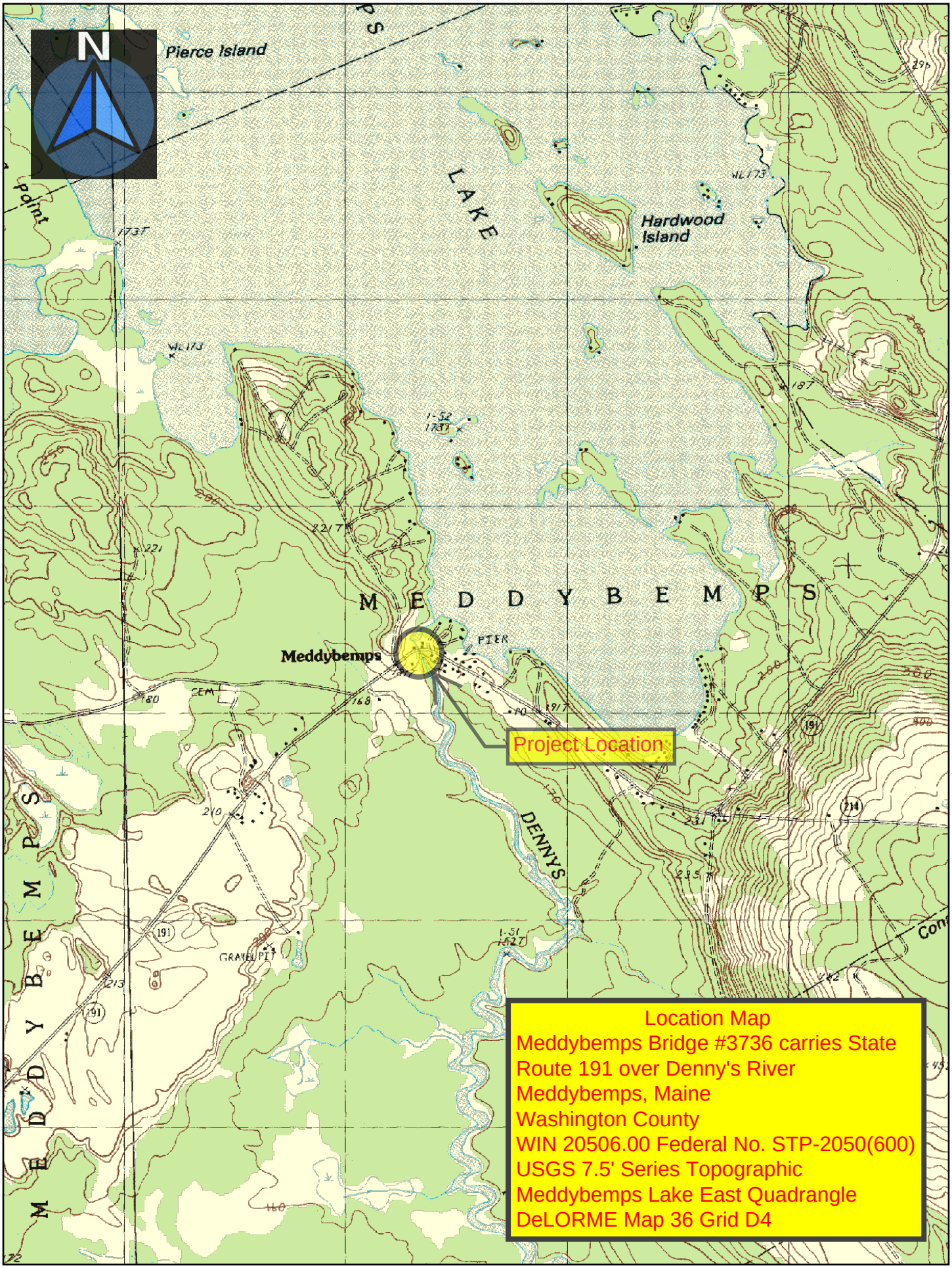
In general, do not use excavated fill or native soils for fill anywhere beneath the new pavement structure, for dressing slopes or for new backfill. If the contractor wishes to reuse excavated granular material as embankment fill, it is recommended that the materials be stockpiled and tested for meeting MaineDOT requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill. Stockpiled materials meeting appropriate MaineDOT Specifications may be reused in accordance with Standard Specification Section 203 Excavation and Embankment.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Meddybemps Bridge in Meddybemps, 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.

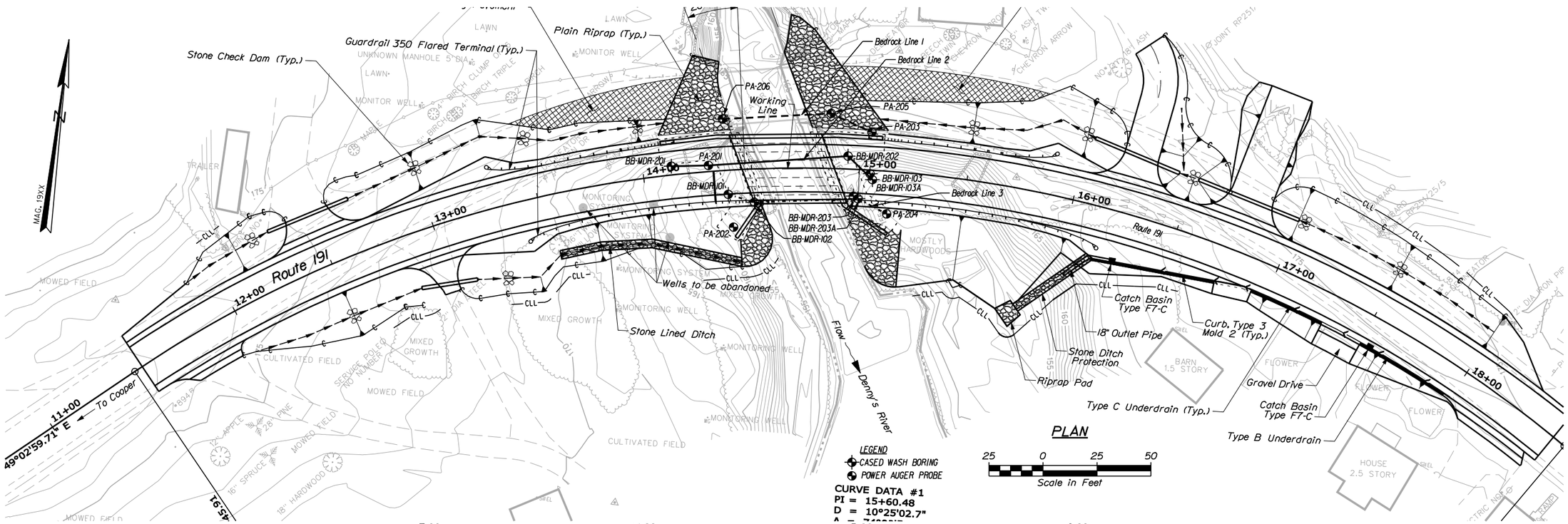
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

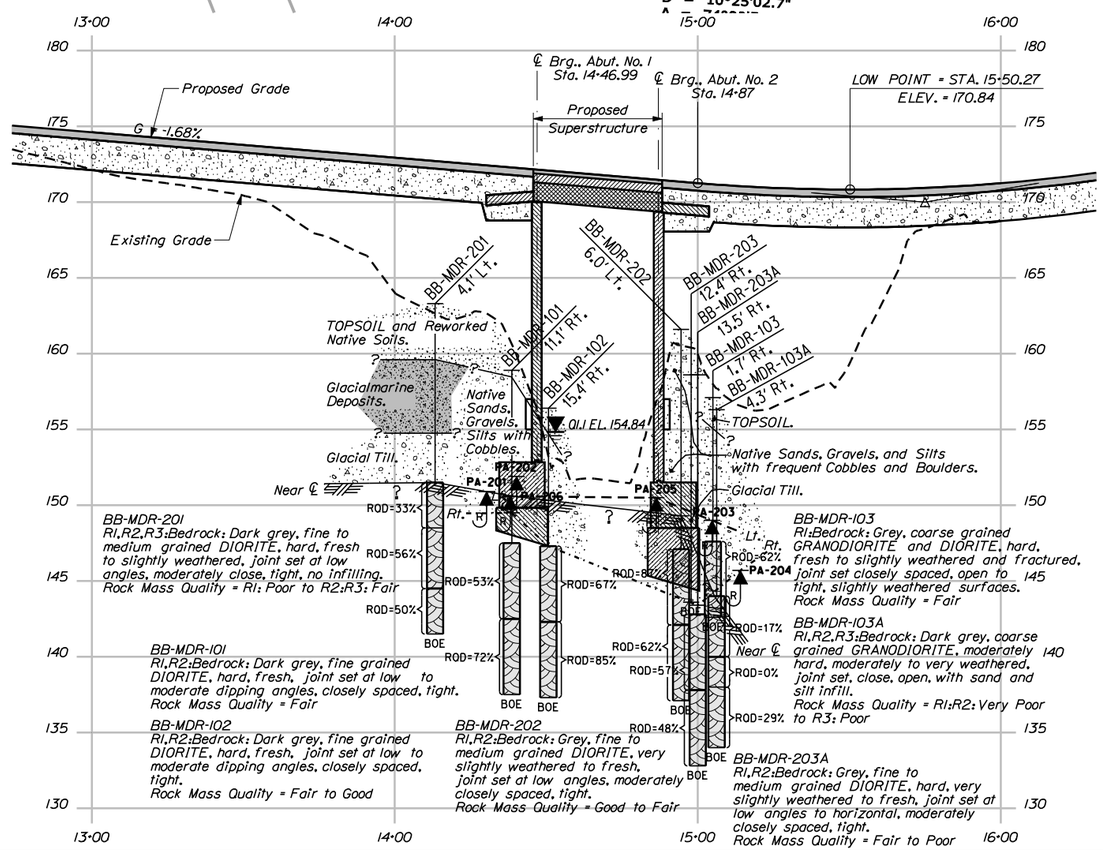
Sheets

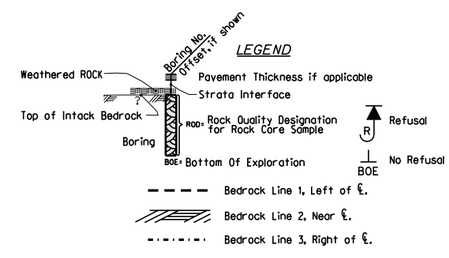


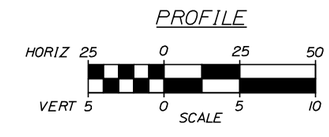
Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.









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

STP-2050(600)

WIN 20506.00

BRIDGE PLANS

BRIDGE NO. 3736

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	L.KRUSINSKI	JAN 2014
DESIGN3-DETAILED3	T.WHITE	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MEDDYBEMPS BRIDGE
DENNY'S RIVER
MEDDYBEMPS WASHINGTON COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 4

Filename: ...\geotech\msta\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:12/1/2014

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/1/2014

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-201 | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 163.3	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 7/14/2014; 12:00-15:30	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 14+13.3; 4.1 ft Lt.	Casing ID/OD: HW	Water Level*: None Observed	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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	24/14	0.00 - 2.00	1/2/2/3	4	5	SSA	163.10	0.2" SOD. Brown, damp, loose, Silty, fine to medium SAND, trace gravel. (Topsoil and Reworked Native Soils). — 0.20	
5	2D	24/18	5.00 - 7.00	6/8/4/4	10	13			Olive-brown, damp, stiff, SILT, some fine to medium SAND, trace gravel. (Weathered Glaciomarine Deposits). — 3.50	
								154.80	— 8.50	
10	3D	4.8/4.8	10.00 - 10.40	50(4.8")	---				Grey brown, wet, very dense, Gravelly, fine to coarse SAND, little silt, (Glacial Till). Cable from 10.4-10.7 ft bgs. Roller Coned ahead to 11.8 ft bgs. *110 blows for 0.8 ft.	
	R1	36/32	11.80 - 14.80	RQD = 33%	---		010 NQ-2	151.50	— 11.80	
									Top of Bedrock at Elev. 151.5 ft. R1:Bedrock: Dark grey, fine to medium grained, DIORITE, hard, slightly weathered, joint set at low angles to 45 degrees, moderately close, tight, no infilling. Rock Mass Quality = Poor. R1:Core Times (min:sec) 11.8-12.8 ft (2:30) 12.8-13.8 ft (2:25) 13.8-14.8 ft (3:50) 89% Recovery Core Blocked. R2:Bedrock: Similar to R1, except fresh. Rock Mass Quality = Fair. R2:Core Times (min:sec) 14.8-15.8 ft (2:20) 15.8-16.8 ft (3:45) 16.8-17.8 ft (3:40) 89% Recovery Core Blocked. R3: Bedrock: Similar to R2. Rock Mass Quality = Poor. R3:Core Times (min:sec) 18.8-19.8 ft (2:00) 19.8-20.8 ft (2:20) 20.8-21.8 ft (2:25) 97% Recovery	
15	R2	48/46	14.80 - 18.80	RQD = 56%						
	R3	36/35	18.80 - 21.80	RQD = 50%				141.50	— 21.80	
20									Bottom of Exploration at 21.80 feet below ground surface.	

Remarks: Auto Hammer #283

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-MDR-201

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-101 | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 158.9	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 1/13/2014; 10:00-15:30	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 14+38.6; 11.1 ft Rt.	Casing ID/OD: HW	Water Level*: 4.2 ft bgs.	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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	6/6	0.00 - 0.50	55(6")	---		SSA	158.40	Brown, wet, fine to coarse SAND, some silt, little gravel, trace roots. (Topsoil). — 0.50	
									Cobble from 0.5-1.0 ft bgs.	
5	2D	24/16	4.50 - 6.50	7/5/14/9	19	25			Brown, wet, medium dense, fine to coarse SAND, some silt, little gravel.	GM262337 A-2-4, SM WC=18.3%
									Cobble from 7.9-8.4 ft bgs.	
10	R1	60/57	11.40 - 16.40	RQD = 53%			010 NQ-2	149.50	9.40 — 010 blows for 0.4 ft. Top of Bedrock at Elev. 149.5 ft. Roller Coned ahead to 11.4 ft bgs. R1: Bedrock: Dark grey, fine grained DIORITE, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 11.4-12.4 ft (3:41) 12.4-13.4 ft (1:54) 13.4-14.4 ft (1:55) 14.4-15.4 ft (1:44) 15.4-16.4 ft (2:21) 95% Recovery	
15	R2	60/60	16.40 - 21.40	RQD = 72%					R2:Bedrock: Similar to R1. Rock Mass Quality = Fair. R2:Core Times (min:sec) 16.4-17.4 ft (1:25) 17.4-18.4 ft (1:36) 18.4-19.4 ft (1:25) 19.4-20.4 ft (1:22) 20.4-21.4 ft (1:30) 100% Recovery	
20								137.50	21.40 — Bottom of Exploration at 21.40 feet below ground surface.	

Remarks: Auto Hammer #283

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-MDR-101

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-102 | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 156.3	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 1/14/2014; 07:30-11:30	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 14+50.7; 15.4 ft Rt.	Casing ID/OD: HW & NW	Water Level*: 4.0 ft bgs.	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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	24/13	0.00 - 2.00	4/6/7/7	13	17	SSA	154.30	Brown, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace of organics and roots. (Topsoil). — 0.00	
5	2D	12/12	4.50 - 5.50	8/54	---				Light brown, wet, fine to coarse SAND, some gravel, little silt, trace clay. Set in NW Casing; roller coned ahead to 9.1 ft bgs. Cobble from 5.5-6.2 ft bgs.	GM262318 A-1-b, SC-SM WC=15.2%
10	R1	60/60	9.10 - 14.10	RQD = 67%			NQ-2	147.20	9.10 — Top of Bedrock at Elev. 147.2 ft. Set in NW Casing at 9.1 ft bgs. R1:Bedrock: Dark grey, fine grained, DIORITE, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 9.1-10.1 ft (1:44) 10.1-11.1 ft (2:17) 11.1-12.1 ft (1:10) 12.1-13.1 ft (1:05) 13.1-14.1 ft (1:40) 100% Recovery	
15	R2	60/60	14.40 - 19.40	RQD = 85%					R2:Bedrock: Similar to R1, except Rock Mass Quality = Good. R2:Core Times (min:sec) 14.1-15.1 ft (1:04) 15.1-16.1 ft (1:02) 16.1-17.1 ft (1:10) 17.1-18.1 ft (1:08) 18.1-19.1 ft (1:02) 100% Recovery	
20								137.20	19.10 — Bottom of Exploration at 19.10 feet below ground surface.	

Remarks: Auto Hammer #283

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-MDR-102

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-202 | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 161.6	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 7/16/2014; 07:00-11:30	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 14+94.5; 6.0 ft Lt.	Casing ID/OD: HW	Water Level*: 8.0 ft bgs.	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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	24/14	0.00 - 2.00	1/2/4/13	6	8	SSA		Brown, moist, loose, fine to coarse SAND, little gravel, little silt, occasional cobble.	GM262311 A-1-b, SW-SM WC=16.1%
5	2D	14.4/7	5.00 - 6.20	3/5/50(2.4")	---			157.60	— 4.00 Brown, wet, very dense, Gravelly fine to coarse SAND, little silt, occasional cobbles.	
									Cobble from 10.0-10.6 ft bgs. Very cobbley from 10.0-12.3 ft bgs.	
10	R1	60/60	14.50 - 19.50	RQD = 81%			RC NQ-2	149.30	— 12.30 Top of Bedrock at Elev. 149.3 ft. Roller Coned ahead from 12.3-14.5 ft bgs.. NO Breaks. R1: Bedrock: Grey, fine to medium grained, DIORITE, very slightly weathered, joint set a low angles, moderately closely spaced, tight. Rock Mass Quality = Good. R1:Core Times (min:sec) 14.5-15.5 ft (1:45) 15.5-16.5 ft (2:10) 16.5-17.5 ft (2:30) 17.5-18.5 ft (2:30) 18.5-19.5 ft (3:00) 100% Recovery	
15	R2	60/60	19.50 - 24.50	RQD = 62%					R2:Bedrock: Similar to R1 except fresh. Rock Mass Quality = Fair. R2:Core Times (min:sec) 19.5-20.5 ft (2:00) 20.5-21.5 ft (2:10) 21.5-22.5 ft (1:50) 22.5-23.5 ft (2:00) 23.5-24.5 ft (2:20) 100% Recovery	
20								137.10	24.50 — Bottom of Exploration at 24.50 feet below ground surface.	

Remarks: Auto Hammer #283

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-MDR-202

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-103 | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 157.0	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 1/14/2014; 12:00-16:00	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 15+05; 1.1 ft Rt.	Casing ID/OD: HW & NW	Water Level*: 3.7 ft bgs.	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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	24/5	0.00 - 2.00	2/2/2/2	4	5	SSA		Dark brown, saturated, loose, fine to coarse SAND, little gravel, little silt, trace organics. (Topsoil). — 0.00	
								155.00		
									Cobble from 3.4-4.0 ft bgs.	
5	2D	24/10	5.00 - 7.00	12/12/14/27	26	35	15		Dark brown, dense, wet, fine to coarse SAND, little silt, little gravel, with rock fragment. Roller Coned ahead to 9.5 ft bgs.	
							26			
							44			
							87			
10	R1	60/60	9.50 - 14.50	RQD = 62%			050 NQ-2	147.50	9.50 — 050 blows for 0.5 ft. Top of Bedrock at Elev. 147.5 ft. Set in NW Casing at 9.5 ft bgs. R1:Bedrock: Grey, coarse grained, GRANODIORITE, transitioning to DIORITE then back to GRANODIORITE, hard, fresh, slightly weathered and fractured, joint set low to moderate dipping angles, closely spaced, open to tight, slightly weathered surfaces. Rock Mass Quality = Fair. R1:Core Times (min:sec) 9.5-10.5 ft (2:10) 10.5-11.5 ft (1:10) 11.5-12.5 ft (1:44) 12.5-13.5 ft (1:28) 13.5-14.5 ft (2:00) 100% Recovery	
15								142.50	14.50 — Bottom of Exploration at 14.50 feet below ground surface. Twisted off outer core barrel, left in hole. Moved to BB-MDR-103A.	
20										

Remarks: Auto Hammer #283

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-MDR-103

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's Location: Meddybemps, Maine

Boring No.: BB-MDR-103A | WIN: 20506.00

Driller: Northern Test Boring	Elevation (ft.) 156.2	Auger ID/OD: 5" Solid Stem	
Operator: Mike/Adam	Datum: NAVD88	Sampler: Standard Split Spoon	
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 1/15/2014; 07:00-13:00	Drilling Methods: Cased Wash Boring	Core Barrel: NQ-2*	
Boring Location: 15+06.2; 4.3 ft Rt.	Casing ID/OD: HW	Water Level*: 3.6 ft bgs.	
Hammer Efficiency Factor: 0.801	Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR = weight of rods 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							SSA		Similar to BB-MDR-103. — 0.00	
									Cobble from 3.6-4.2 ft bgs.	
5									Cobble from 8.0-8.6 ft bgs.	
10	1D	24/15	10.00 - 12.00	15/33/32/34	65	87	38			
							110	144.70	11.50 — Grey, wet, very dense, Gravelly fine to coarse SAND, little silt, (Glacial Till). *100 blows for 0.3 ft. Roller Coned ahead to 12.3 ft bgs.	GM262319 A-1-a, SM WC=8.1%
	R1	48/44	12.30 - 16.30	RQD = 17%			050 NQ-2	143.90	12.30 — Top of Bedrock at Elev. 143.9 ft. R1:Bedrock: Dark grey, coarse grained, GRANODIORITE, hard, fresh, slightly weathered to very weathered with fractured zones, joint set at low to moderately dipping angles, closes, opens with sand and silt infill. Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 12.3-13.3 ft (2:00) 13.3-14.3 ft (2:10) 14.3-15.3 ft (1:30) 15.3-16.3 ft (3:30) 92% Recovery Core Blocked. R2:Bedrock: Similar to R1. R2:Core Times (min:sec) 16.3-17.3 ft (2:30) 17.3-18.3 ft (3:35) 100% Recovery Core Blocked. R3:Bedrock: Similar to R1 except less fractured and less weathered, transitioning to DIORITE. Rock Mass Quality = Poor. R3:Core Times (min:sec) 18.3-19.3 ft (1:45) 19.3-20.3 ft (3:00) 20.3-21.3 ft (4:50) 21.3-22.3 ft (5:00) 96% Recovery Core Blocked	
15	R2	24/24	16.30 - 18.30	RQD = 0%						
	R3	48/46	18.30 - 22.30	RQD = 29%						
20								133.90	22.30 — Bottom of Exploration at 22.30 feet below ground surface.	

Remarks: Auto Hammer #283

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-MDR-103A

State of Maine - Department of Transportation
Power Auger Probe Summary Sheet

Town(s): Meddybemps **Work Number: 20506.00**

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Elev. (Feet)	Comments / Date 7/14,16/2014
14+30.3	2.8 Lt.		12.0		162.9	PA-201
14+40.2	26.4 Rt.		6.2		158.1	PA-202
15+04.7	17.6 Lt.		18.4		167.4	PA-203
15+14.1	19.7 Rt.		8.2		153.9	PA-204
14+86.3	25.5 Lt.		19.2		169.7	PA-205
14+38.1	23.9 Lt.		19.4		170.0	PA-206

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2050(600)

WIN 20506.00

BRIDGE PLANS

BRIDGE NO. 3736

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	L.KRUSINSKI	DEC 2014
DESIGN3-DETAILED3	T.WHITE	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MEDDYBEMPS BRIDGE
DENNY'S RIVER
MEDDYBEMPS WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
3
OF 4

Filename: ...\msta\008_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:12/1/2014

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's
Location: Meddybemps, Maine

Boring No.: BB-MDR-203
WIN: 20506.00

Driller:	Northern Test Boring	Elevation (ft.)	158.6	Auger ID/OD:	5" Solid Stem
Operator:	Mike/Adam	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrick D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/15/2014; 09:30-12:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+97.8, 12.4 ft Rt.	Casing ID/OD:	HW	Water Level*:	None Observed

Hammer Efficiency Factor: 0.801 — 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 140lbs. 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
$S_{u(lab)}$ = 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	24/8	0.00 - 2.00	2/3/3/6	6	8	SSA		Brown, moist, loose, Gravelly fine to coarse SAND, little silt, occasional cobbles.	
									Boulder from 2.0-4.5 ft bgs.	
									Boulder from 3.4-4.7 ft bgs.	
5	MD	2.4/0	5.00 - 5.20						Failed sample attempt. Cobble from 5.2-5.8 ft bgs.	
									Cobble from 6.6-7.0 ft bgs.	
	2D	8.4/4	7.00 - 7.70	11/50(2.4")	---				Brown, wet, very dense, Gravelly fine to coarse SAND, trace silt, occasional cobbles. Boulder from 7.7-9.5 ft bgs.	
								149.10	9.50	
10	3D	24/13	10.00 - 12.00	16/34/16/45	50	67			Grey, wet, very dense, gravelly fine to coarse SAND, occasional small and large cobbles. (Glacial Till).	
15								143.60	15.00	
									Bottom of Exploration at 15.00 feet below ground surface. Broke casing no Boulder. Moved to BB-MDR-203A.	
20										
25										

Remarks:
Auto Hammer #283
Broke Casing. Left 5 ft of casing in bore hole.

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-MDR-203

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

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's
Location: Meddybemps, Maine

Boring No.: BB-MDR-203A
WIN: 20506.00

Driller:	Northern Test Boring	Elevation (ft.)	158.6	Auger ID/OD:	5" Solid and 6"[illegible] Hollow
Operator:	Mike/Adam	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrick D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/15/2014; 13:00-16:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+00+, 13.5 ft Rt.	Casing ID/OD:	HW	Water Level*:	9.0 ft bgs.

Hammer Efficiency Factor: 0.801 — 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 140lbs. 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
$S_{u(lab)}$ = 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							SSA		Soils similar to BB-MDR-203. Auger to 15.0 ft bgs with Solid Stem, then auger to 15.0 ft bgs with Hollow Stem and set in HW Casing.	
									Boulder from 2.4-4.0 ft bgs.	
									Boulder from 4.3-5.5 ft bgs.	
5									Boulder from 8.0-9.6 ft bgs.	
10							20			
							15			
							46			
							30			
							51			
15							RC	143.40	15.20	
	R1	57.6/ 57.6	16.00 - 20.80	RQD = 57%			NQ-2		Top of Bedrock at Elev. 143.4 ft. Roller Cone Ahead to 16.0 ft bgs. R1 Bedrock: Granite to Diorite grained. DIORITE, hard, very slightly weathered to fresh, joint set a few angles to horizontal, moderately closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 16.0-17.0 ft (2:00) 17.0-18.0 ft (1:45) 18.0-19.0 ft (2:00) 19.0-20.0 ft (2:45) 20.0-20.8 ft (3:00) 100% Recovery Core Blocked R2: Bedrock: Similar to R2, except Rock Mass Quality= Poor. R2:Core Times (min:sec) 20.8-21.8 ft (2:45) 21.8-22.8 ft (2:15) 22.8-23.8 ft (1:45) 23.8-24.8 ft (1:35) 24.8-25.8 ft (2:00) 100% Recovery	
20	R2	60/60	20.80 - 25.80	RQD = 48%						
25								132.80	25.80	
									Bottom of Exploration at 25.80 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:
Auto Hammer #283

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-MDR-203A

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2050(600)

WIN 20506.00

BRIDGE NO. 3736 — BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	L.KRUSINSKI	DEC 2014
DESIGN3-DETAILED3	T.WHITE	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

MEDDYBEMPS BRIDGE
DENNY'S RIVER
MEDDYBEMPS WASHINGTON COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs and Power Auger Probes

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 thumbnai
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

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

Sample Container Labeling Requirements:

PIN	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: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-101
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	158.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/13/2014; 10:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+38.6, 11.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	4.2 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	6/6	0.00 - 0.50	55(6")	---		SSA	158.40	Brown, wet, fine to coarse SAND, some silt, little gravel, trace roots. (Topsoil).	
									0.50	
									Cobble from 0.5-1.0 ft bgs.	
	2D	24/16	4.50 - 6.50	7/5/14/9	19	25			Brown, wet, medium dense, fine to coarse SAND, some silt, little gravel.	G#262317 A-2-4, SM WC=18.3%
5							20			
							26			
							88			
							85		Cobble from 7.9-8.4 ft bgs.	
							a70 RC	149.50	a70 blows for 0.4 ft.	
									9.40	
10									Top of Bedrock at Elev. 149.5 ft. Roller Coned ahead to 11.4 ft bgs.	
	R1	60/57	11.40 - 16.40	RQD = 53%			NQ-2		R1: Bedrock: Dark grey, fine grained DIORITE, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 11.4-12.4 ft (3:41) 12.4-13.4 ft (1:54) 13.4-14.4 ft (1:55) 14.4-15.4 ft (1:44) 15.4-16.4 ft (2:21) 95% Recovery	
15										
	R2	60/60	16.40 - 21.40	RQD = 72%					R2:Bedrock: Similar to R1. Rock Mass Quality = Fair. R2:Core Times (min:sec) 16.4-17.4 ft (1:25) 17.4-18.4 ft (1:36) 18.4-19.4 ft (1:25) 19.4-20.4 ft (1:22) 20.4-21.4 ft (1:30) 100% Recovery	
20										
								137.50	21.40	
									Bottom of Exploration at 21.40 feet below ground surface.	
25										

Remarks:

Auto Hammer #283

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-102
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	156.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/14/2014; 07:30-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+50.7, 15.4 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.00 - 2.00	4/6/7/7	13	17	SSA		Brown, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace of organics and roots, (Topsoil).	
								154.30	0.00	
	2D	12/12	4.50 - 5.50	8/54	---				Light brown, wet, fine to coarse SAND, some gravel, little silt, trace clay. Set in HW Casing, roller coned ahead to 9.1 ft bgs. Cobble from 5.5-6.2 ft bgs.	G#262318 A-1-b, SC-SM WC=15.2%
5							12			
							14			
							64			
							40			
								147.20	9.10	
	R1	60/60	9.10 - 14.10	RQD = 67%			NQ-2		Top of Bedrock at Elev. 147.2 ft. Set in NW Casing at 9.1 ft bgs. R1:Bedrock: Dark grey, fine grained, DIORITE, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 9.1-10.1 ft (1:44) 10.1-11.1 ft (2:17) 11.1-12.1 ft (1:10) 12.1-13.1 ft (1:05) 13.1-14.1 ft (1:40) 100% Recovery	
10										
	R2	60/60	14.40 - 19.40	RQD = 85%					R2:Bedrock: Similar to R1, except Rock Mass Quality = Good. R2:Core Times (min:sec) 14.1-15.1 ft (1:04) 15.1-16.1 ft (1:02) 16.1-17.1 ft (1:10) 17.1-18.1 ft (1:08) 18.1-19.1 ft (1:02) 100% Recovery	
15										
								137.20	19.10	
									Bottom of Exploration at 19.10 feet below ground surface.	
20										
25										

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-201
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	163.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/14/2014; 12:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+13.3, 4.1 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	1/2/2/3	4	5	SSA	163.10	0.2' SOD.	
									0.20 Brown, damp, loose, Silty, fine to medium SAND, trace gravel. (Topsoil and Reworked Native Soils).	
								159.80	3.50	
5	2D	24/18	5.00 - 7.00	6/6/4/4	10	13			Olive-brown, damp, stiff, SILT, some fine to medium SAND, trace gravel. (Weathered Glaciomarine Deposits).	G#242670 A-4, ML WC=18.8%
								154.80	8.50	
10	3D	4.8/4.8	10.00 - 10.40	50(4.8")	---		57		Grey brown, wet, very dense, Gravelly, fine to coarse SAND, little silt, (Glacial Till). Cobble from 10.4-10.7 ft bgs. Roller Coned ahead to 11.8 ft bgs.	
	R1	36/32	11.80 - 14.80	RQD = 33%			[a]110 NQ-2	151.50	[a]110 blows for 0.8 ft. 11.80	
	R2	48/46	14.80 - 18.80	RQD = 56%					Top of Bedrock at Elev. 151.5 ft. R1:Bedrock: Dark grey, fine to medium grained, DIORITE, hard, slightly weathered, joint set at low angles to 45 degrees, moderately close, tight, no infilling. Rock Mass Quality = Poor. R1:Core Times (min:sec) 11.8-12.8 ft (2:30) 12.8-13.8 ft (2:25) 13.8-14.8 ft (3:50) 89% Recovery Core Blocked R2:Bedrock: Similar to R1, except fresh. Rock Mass Quality = Fair. R2:Core Times (min:sec) 14.8-15.8 ft (2:20) 15.8-16.8 ft (1:45) 16.8-17.8 ft (1:40) 17.8-18.8 ft (3:40) 89% Recovery Core Blocked	
15										
	R3	36/35	18.80 - 21.80	RQD = 50%					R3: Bedrock: Similar to R2. Rock Mass Quality = Poor. R3:Core Times (min:sec) 18.8-19.8 ft (2:00) 19.8-20.8 ft (2:20) 20.8-21.8 ft (2:25) 97% Recovery	
20								141.50	21.80	
25									**Bottom of Exploration at 21.80 feet below ground surface.**	

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-103
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	157.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/14/2014; 12:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+05, 1.7 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	3.7 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.00 - 2.00	2/2/2/2	4	5	SSA			Dark brown, saturated, loose, fine to coarse SAND, little gravel, little silt, trace organics, (Topsoil).	
								155.00		0.00	
										Cobble from 3.4-4.0 ft bgs.	
5	2D	24/10	5.00 - 7.00	12/12/14/27	26	35	15			Dark brown, dense, wet, fine to coarse SAND, little silt, little gravel, with rock fragment. Roller Coned ahead to 9.5 ft bgs.	
							26				
							44				
							87				
	R1	60/60	9.50 - 14.50	RQD = 62%			[a]50 NQ-2	147.50		[a]50 blows for 0.5 ft. 9.50	
10										Top of Bedrock at Elev. 147.5 ft. Set in NW Casing at 9.5 ft bgs. R1:Bedrock: Grey, coarse grained, GRANODIORITE transitioning to DIORITE then back to GRANODIROITE, hard, fresh to slighty weathered and fractured, joint set at low to moderately dipping angles, closely spaced, open to tight, slightly weathered surfaces. Rock Mass Quality = Fair. R1:Core Times (min:sec) 9.5-10.5 ft (2:10) 10.5-11.5 ft (1:10) 11.5-12.5 ft (1:44) 12.5-13.5 ft (1:28) 13.5-14.5 ft (2:00) 100% Recovery	
								142.50		14.50	
15										**Bottom of Exploration at 14.50 feet below ground surface.** Twisted off outer core barrel, left in hole. Moved to BB-MDR-103A.	
20											
25											

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-103A
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	156.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/15/2014; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+06.2, 4.3 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.6 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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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							SSA		Similar to BB-MDR-103.	
									Cobble from 3.6-4.2 ft bgs.	
5										
									Cobble from 8.0-8.6 ft bgs.	
10	1D	24/15	10.00 - 12.00	15/33/32/34	65	87	38			
							110	144.70	11.50	G#262319 A-1-b, SM WC=8.1%
	R1	48/44	12.30 - 16.30	RQD = 17%			a30 NQ-2	143.90	Grey, wet, very dense, Gravelly fine to coarse SAND, little silt, (Glacial Till). a30 blows for 0.3 ft. Roller Coned ahead to 12.3 ft bgs. 12.30 Top of Bedrock at Elev. 143.9 ft. R1:Bedrock: Dark grey, coarse grained, GRANODIORITE, moderately hard, moderately weathered to very weathered, with fractured zones, joint set at low to moderately dipping angles, close, open, with sand and silt infill. Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 12.3-13.3 ft (2:00) 13.3-14.3 ft (2:10) 14.3-15.3 ft (1:30) 15.3-16.3 ft (3:30) 92% Recovery Core Blocked	
15	R2	24/24	16.30 - 18.30	RQD = 0%					R2:Bedrock: Similar to R1. R2:Core Times (min:sec) 16.3-17.3 ft (2:30) 17.3-18.3 ft (3:35) 100% Recovery Core Blocked	
	R3	48/46	18.30 - 22.30	RQD = 29%					R3:Bedrock: Similar to R1 except less fractured and less weathered, transitioning to DIORITE. Rock Mass Quality = Poor. R3:Core Times (min:sec) 18.3-19.3 ft (1:45) 19.3-20.3 ft (3:00) 20.3-21.3 ft (4:50) 21.3-22.3 ft (5:00) 96% Recovery	
20								133.90		
25										

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-103A
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	156.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1/15/2014; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+06.2, 4.3 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	3.6 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu 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 = 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.
25										Core Blocked 22.30	
										Bottom of Exploration at 22.30 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-202
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	161.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/16/2014; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+94.5, 6.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	1/2/4/13	6	8	SSA		Brown, moist, loose, fine to coarse SAND, little gravel, little silt, occasional cobble.	G#242671 A-1-b, SW-SM WC=16.1%
								157.60	4.00	
5	2D	14.4/7	5.00 - 6.20	3/5/50(2.4")	---				Brown, wet, very dense, Gravelly fine to coarse SAND, little silt, occasional cobbles.	
10									Cobble from 10.0-10.6 ft bgs. Very cobbley from 10.0-12.3 ft bgs.	
							RC	149.30	12.30	
									Top of Bedrock at Elev. 149.3 ft. Roller Coned ahead from 12.3-14.5 ft bgs., NO Breaks.	
15	R1	60/60	14.50 - 19.50	RQD = 87%			NQ-2		R1: Bedrock: Grey, fine to medium grained, DIORITE, very slightly weathered, joint set a low angles, moderately closely spaced, tight. Rock Mass Quality = Good. R1:Core Times (min:sec) 14.5-15.5 ft (1:45) 15.5-16.5 ft (2:10) 16.5-17.5 ft (2:30) 17.5-18.5 ft (2:30) 18.5-19.5 ft (3:00) 100% Recovery	
20	R2	60/60	19.50 - 24.50	RQD = 62%					R2:Bedrock: Similar to R1 except fresh. Rock Mass Quality = Fair. R2:Core Times (min:sec) 19.5-20.5 ft (2:00) 20.5-21.5 ft (2:10) 21.5-22.5 ft (1:50) 22.5-23.5 ft (2:00) 23.5-24.5 ft (2:20) 100% Recovery	
25								137.10	24.50	

Remarks:

Auto Hammer #283

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: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-202
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	161.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/16/2014; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+94.5, 6.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	8.0 ft bgs.
Hammer Efficiency Factor:	0.801	**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.
25										**Bottom of Exploration at 24.50 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #283

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-203
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	158.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/15/2014; 09:30-12:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+97.8, 12.4 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MU = Unsuccessful Thin Wall Tube Sample attempt
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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	2/3/3/6	6	8	SSA		Brown, moist, loose, Gravelly fine to coarse SAND, little silt, occasional cobbles.	
									Boulder from 2.0-4.5 ft bgs.	
									Boulder from 3.4-4.7 ft bgs.	
5	MD	2.4/0	5.00 - 5.20						Failed sample attempt. Cobble from 5.2-5.8 ft bgs.	
									Cobble from 6.6-7.0 ft bgs.	
	2D	8.4/4	7.00 - 7.70	11/50(2.4")	---				Brown, wet, very dense, Gravelly fine to coarse SAND, trace silt, occasional cobbles. Boulder from 7.7-9.5 ft bgs.	
10	3D	24/13	10.00 - 12.00	16/34/16/45	50	67		149.10	9.50	
									Grey, wet, very dense, gravelly fine to coarse SAND, occasional small and large cobbles, (Glacial Till).	
15								143.60	15.00	
									Bottom of Exploration at 15.00 feet below ground surface. Broke casing no Boulder, moved to BB-MDR-203A.	
20										
25										

Remarks:

Auto Hammer #283
Broke Casing, left 5 ft of casing in bore hole.

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: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-203A
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	158.6	**Auger ID/OD:**	5" Solid and 6¾" Hollow
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/15/2014; 13:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+00, 13.5 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.801	**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							SSA			Soils similar to BB-MDR-203. Auger to 15.0 ft bgs with Solid Stem, then auger to 15.0 ft bgs with Hollow Stem and set in HW Casing.	
										Boulder from 2.4-4.0 ft bgs.	
										Boulder from 4.3-5.5 ft bgs.	
5											
										Boulder from 8.0-9.6 ft bgs.	
10							20				
							15				
							46				
							30				
							51				
15							RC	143.40		15.20	
	R1	57.6/57.6	16.00 - 20.80	RQD = 57%			NQ-2			Top of Bedrock at Elev. 143.4 ft. Roller Coned ahead to 16.0 ft bgs. R1: Bedrock: Grey, fine to medium grained, DIORITE, hard, very slightly weathered to fresh, joint set a low angles to horizontal, moderately closely spaced, tight. Rock Mass Quality = Fair. R1:Core Times (min:sec) 16.0-17.0 ft (2:00) 17.0-18.0 ft (1:45) 18.0-19.0 ft (2:00) 19.0-20.0 ft (2:45) 20.0-20.8 ft (3:00) 100% Recovery Core Blocked	
20	R2	60/60	20.80 - 25.80	RQD = 48%						R2: Bedrock: Similar to R2, except Rock Mass Quality=Poor. R2:Core Times (min:sec) 20.8-21.8 ft (2:45) 21.8-22.8 ft (2:15) 22.8-23.8 ft (1:45) 23.8-24.8 ft (1:35) 24.8-25.8 ft (2:00) 100% Recovery	
25											

Remarks:

Auto Hammer #283

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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.

Boring No.: BB-MDR-203A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Meddybemps Bridge #3736 carries State Route 191 over Denny's River
Location: Meddybemps, Maine

Boring No.: BB-MDR-203A
PIN: 20506.00

Driller:	Northern Test Boring	**Elevation (ft.)**	158.6	**Auger ID/OD:**	5" Solid and 6¾" Hollow
Operator:	Mike/Adam	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/15/2014; 13:00-16:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+00, 13.5 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.801	**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.
25								132.80		25.80	
										Bottom of Exploration at 25.80 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #283

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.

State of Maine - Department of Transportation

Power Auger Probe Summary Sheet

Town(s): Meddybemps Work Number: 20506.00

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Elev. (Feet)	Comments / Date 7/14,16/2014
14+30.3	2.8 Lt.		12.0		162.9	PA-201
14+40.2	26.4 Rt.		6.2		158.1	PA-202
15+04.7	17.6 Lt.		18.4		167.4	PA-203
15+14.1	19.7 Rt.		8.2		153.9	PA-204
14+86.3	25.5 Lt.		19.2		169.7	PA-205
14+38.1	23.9 Lt.		19.4		170.0	PA-206

MaineDOT Drill Crew
Logged By: B. Wilder
Drill Rig: CME 45C

5" Solid Stem Auger

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Meddybemps Work Number: 20506.00

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-MDR-101, 2D	14+38.6	11.1 Rt.	4.5-6.5	262317	1	18.3			SM	A-2-4	II
BB-MDR-102, 2D	14+50.7	15.4 Rt.	4.5-5.5	262318	1	15.2			SC-SM	A-1-b	III
BB-MDR-103A, 1D	15+06.2	4.3 Rt.	11.5-12.0	262319	1	8.1			SM	A-1-b	II
BB-MDR-201, 2D	14+13.3	4.1 Lt.	5.0-7.0	242670	2	18.8			ML	A-4	IV
BB-MDR-202, 1D	14+94.5	6.0 Lt.	0.0-2.0	242671	2	16.1			SW-SM	A-1-b	0

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.

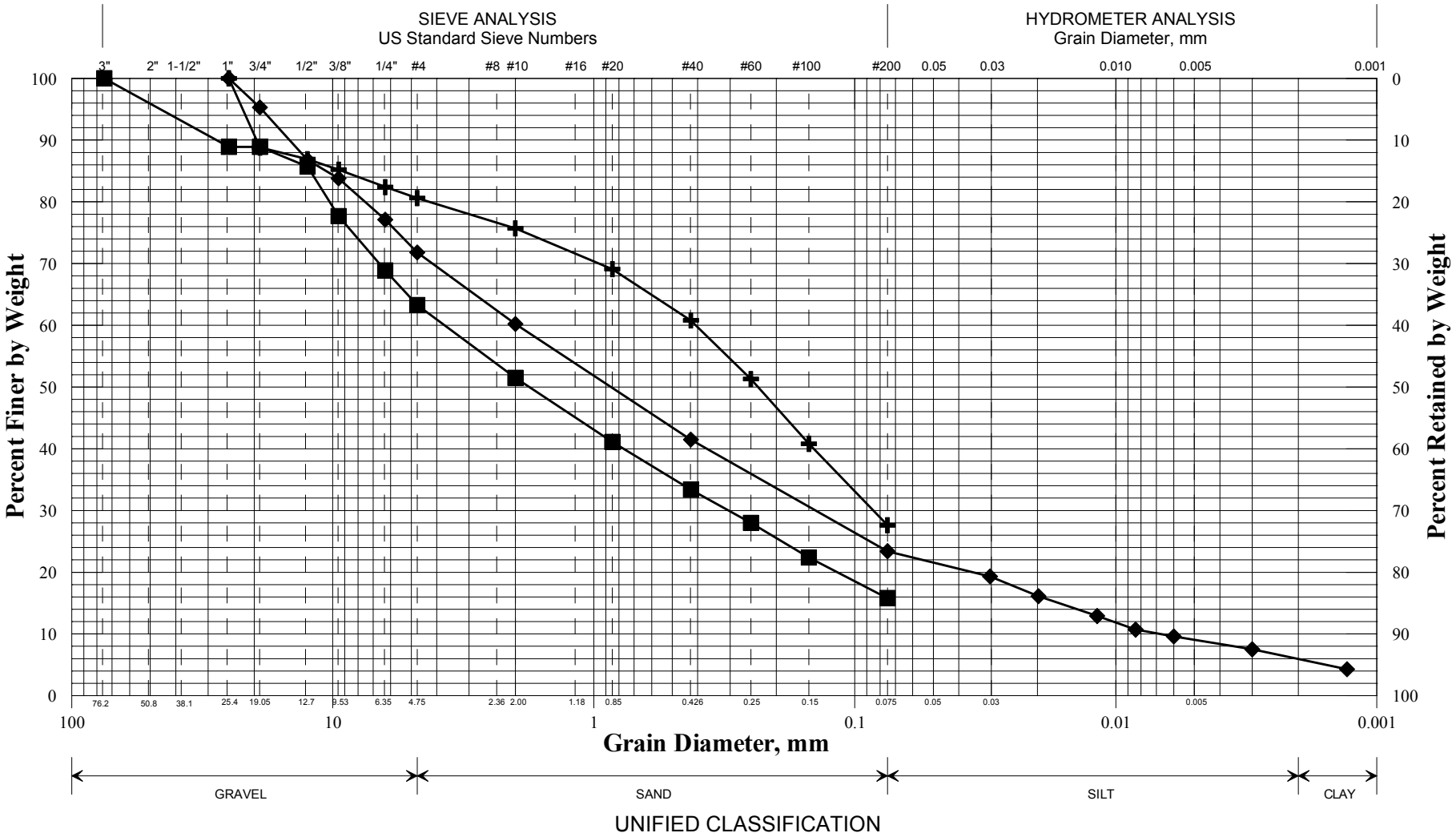
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

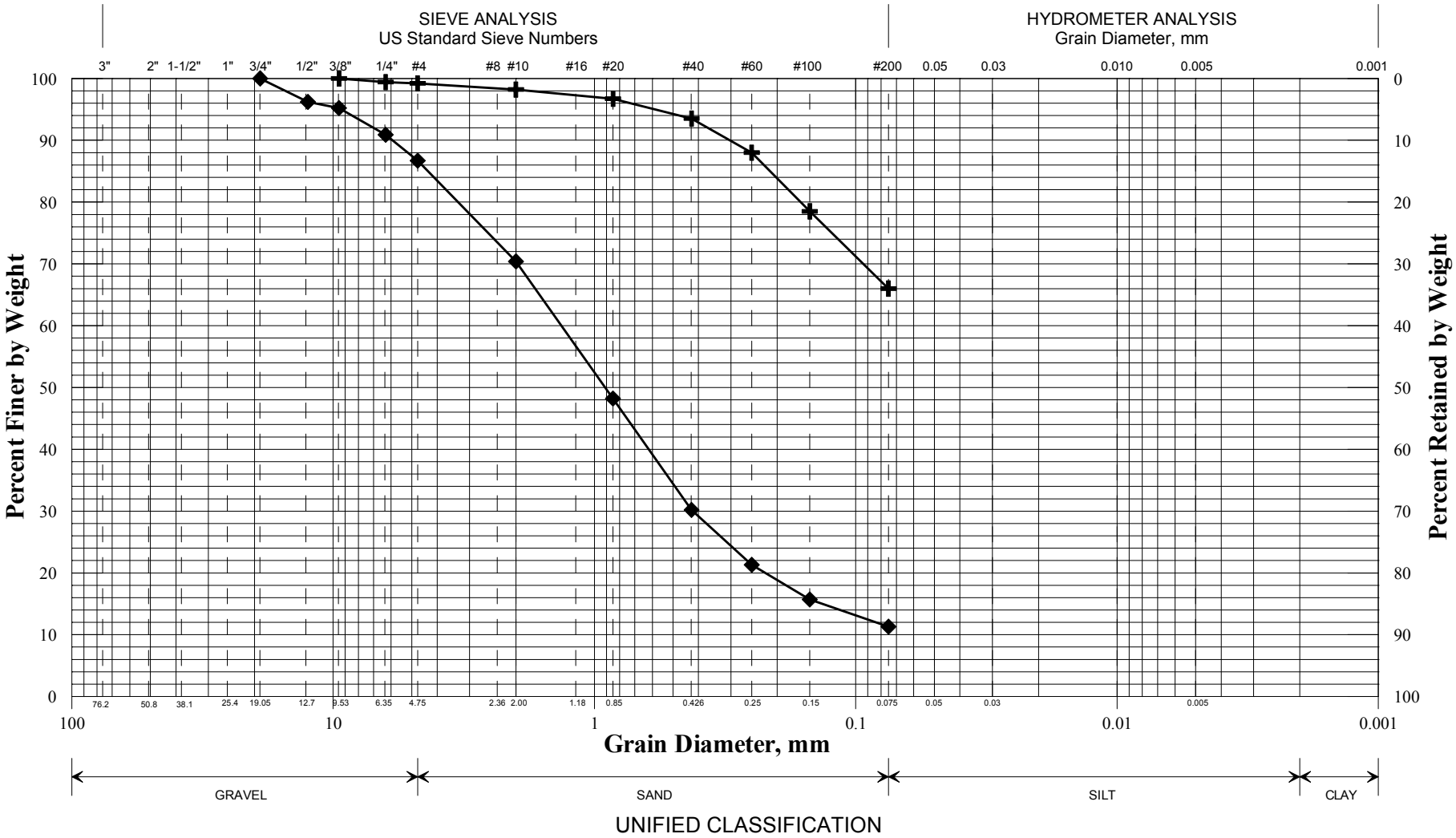


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-MDR-101/2D	14+38.6	11.1 RT	4.5-6.5	SAND, some silt, little gravel.	18.3			
◆	BB-MDR-102/2D	14+50.7	15.4 RT	4.5-5.5	SAND, some gravel, little silt, trace clay.	15.2			
■	BB-MDR-103A/1D	15+06.2	4.3 RT	11.5-12.0	Gravelly SAND, little silt.	8.1			
●									
▲									
✖									

WIN
020506.00
Town
Meddybemps
Reported by/Date
WHITE, TERRY A 1/29/2014

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-MDR-201/2D	14+13.3	4.1 LT	5.0-7.0	SILT, some sand, trace gravel.	18.8			
◆	BB-MDR-202/1D	14+94.5	6.0 LT	0.0-2.0	SAND, little gravel, little silt.	16.1			
■									
●									
▲									
✖									

WIN
020506.00
Town
Meddybemps
Reported by/Date
WHITE, TERRY A 9/11/2014

Appendix C

Calculations

Analysis

Calculation of nominal and factored bearing resistance on rock for Strength Limit State Analysis - Abutment 1 and Abutment 1 Wingwalls

Method

Use data from borings at proposed abutment locations and calculate the nominal bearing resistance as follows:

1. Bedrock Properties from Borings
2. Calculation of Rock Mass Rating
3. Determine rock property constants s and m
4. Calculate nominal bearing resistance of bedrock at each substructure, qn, using RMR/GSI method in Wyllie "Foundations on Rock".

References

1. AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

2. Wyllie, Duncan C, "Foundations on Rock", Second Edition, 2009.

3. "The Hoek-brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

1. Bedrock Properties from Borings

BB-MDR-101, Diorite, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight.
RQD = 53% (RI) and 72% (R2).

BB-MDR-102, Diorite, hard, fresh, joint set at low to moderately dipping angles, closely spaced, tight.
RQD = 67% (RI) and 85% (R2).

BB-MDR-201, Diorite, hard, slightly weathered to fresh, joint set at low angles to 45 degrees, moderately close, tight, no infilling.
RQD = 33% (RI), 56% (R2), and 50%(R3).

Compressive Strength

No UCT Tests conducted on rock samples.

Estimated range of compressive strengths - Ref: Standard Specifications for Highway Bridges 17th Ed. 2002, Table 4.4.8.1.2B

Quartzdiorite Co = 200 - 2,100 ksf or 1,400 - 14,000 psi
Use 7,000 psi

$q_{uc} := 7000 \cdot psi$

B. Determination of Rock Mass Rating (RMR) from LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

From AASHTO - RMR is determined as the sum of five relative ratings listed in LRFD Table 10.4.6.4-1

1. Strength of intact rock

From Standard Specifications for Highway Bridges, 17th Edition, 2002
Table 4.4.8.1.2B uniaxail compressive strength - examine values for quartzdiorite:

Quartzdiorite Co = 200 - 2,100 ksf or 1,400 - 14,000 psi
Use 7,000 psi

Use qu = 7,000 psi = 1008 ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 520-1080 ksf: Relative Rating = 4

2. Drill Core Quality (average RQD of all R1's)

Bedrock RQD = 51% (fair) From LRFD Table 10.4.6.4-1, RQD 50% to 75%;
Select Rating between 8 (20%-50%) and 13 (50%-75%) - Relative Rating = 10

3. Spacing of joints

Spacing of joints follow the foliation and at 45-70 degress to foliation.
From LRFD Table 10.4.6.4.-1 Spacing of joints 2 in. - 1 ft; Relative Rating = 10

4. Condition of joints

Assume slightly rough surfaces, separation <0.05 in., hard joint wall rock; Relative Rating = 20

5. Groundwater conditions

General Conditions = Water under moderate pressure (Relative Rating 4) to Moist only (Relative Rating 7)

Relative Rating = 5

6. From LRFD Table 10.4.6.4.-2 Geomechanics Rating Adjustment for Joint Orientations

Stike and dip orientations of joints are Fair (-7) use Relative Rating = -7

ADJUSTED RMR

$$RMR := 4 + 10 + 10 + 20 + 5 - 7$$

$$RMR = 42$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - E = Coarse grained polymineralic igneous & metamorphic crystalline rocks

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.5.4-3, RMR = 42 is indicative of Fair Quality Rock Mass.

3. Rock Property Constants s and m (Ref. #1 and Ref. #4)

Direct calculation of m ans s is required, Reference 4 (Hoek and Brown, 1988), Table 1.

For a disturbed rock mass: m/mi = exp ((RMR-100)/14)

s = exp ((RMR-100)/6)

mi = m for intact rock

For rock type E, for intact rock, RMR=100, mi = 25 (Ref. # 4, Table 1)

$$m_i := 25$$

$$m := m_i \cdot \exp\left(\frac{RMR - 100}{14}\right)$$

$$m = 0.397$$

$$s := \exp\left(\frac{RMR - 100}{6}\right)$$

$$s = 6.336071 \times 10^{-5}$$

Check with upper and lower bounds:

Fair Qualtiy Rock Mass, RMR = 44 m= 0.458 s= 0.00009

Good Quality Rock Mass, RMR = 23 m= 0.102 s= 3 x 10-6

4. Nominal and Factored Bearing Resistance of Bedrock

Correction Factor for Foundation Shape, from Wyllie Table 5.4 Pg. 138 (Ref. #3)

$C_{f1} := 1.0$ Conservative selection of Cfl = 1.0 for L/B>6

Uniaxial Compressive Strength

$q_{uc} := 7000 \cdot psi$

Nominal Bearing Resistance (Wyllie)

Reference #3: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot q_{uc} \cdot \left[1 + \sqrt{m \cdot \left(s^{\frac{-1}{2}}\right) + 1}\right]$$

$q_n = 65 \cdot ksf$

Factored Bearing Resistance for Strenght Limit State

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$\phi_{bc} := 0.45$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 29 \cdot ksf$ Strength Limit State

Factored Bearing Resistance for Extreme Limit State

Use a resistance factor of 0.80

$\phi_{bc} := 0.80$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 52 \cdot ksf$ Extreme Limit State

Nominal Bearing Resistance (Carter and Kulhawy (1988)

Reference : NCHRP, Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, pg 40, Eq. 82b, and refered to in LRFD C.10.6.3.2.2. Same equation.

$$q_n := q_{uc} \cdot \left[\sqrt{s} + \sqrt{m \cdot \left(\sqrt{s}\right) + s}\right]$$

$$q_n = 65 \cdot ksf$$

Factored Bearing Resistance

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

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

$$q_r := q_n \cdot \phi_{bc}$$

$$q_r = 29 \cdot ksf$$ Strength Limit State

Factored Bearing Resistance for Extreme Limit State

Use a resistance factor of 0.80

$$\phi_{bc} := 0.80$$

$$q_r := q_n \cdot \phi_{bc}$$

$$q_r = 52 \cdot ksf$$ Extreme Limit State

Analysis

Calculation of nominal and factored bearing resistance on rock for Service Limit State Analysis

Approach 1

Per AASHTO LRFD 10.6.2.4.4 - Settlement of Footings on Rock, "For footings bearing on fair to very good rock according to Geomechanics Classification system, as defined in Article 10.4.6.4, and designed in accordance with the provisions of this Section, elastic settlement may generally be assumed to be less than 0.5 inch."

Alternative Method

LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures* , Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Bearing Material:	Weathered or broken bedrock of any kind, except shale.
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16-24 ksf
AASHTO Recommended Value	20 ksf

MaineDOT recommended value based on igneous classification $q_{nominal} := 24 \cdot ksf$

Resistance Factor for Service Limit State $\phi_r := 1.0$

Per LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values for the factored bearing resistance for Service Limit State Analyses, settlement is typically limited to 1 inch

$$q_{factored} := \phi_r \cdot q_{nominal}$$

$$q_{factored} = 24 \cdot ksf$$

Analysis

Calculation of nominal and factored bearing resistance on rock for Strength Limit State Analysis - Abutment 2 and Abutment 2 Wingwalls

Method

Use data from borings at proposed abutment locations and calculate the nominal bearing resistance as follows:

1. Bedrock Properties from Borings and Lab Tests
2. Calculation of Rock Mass Rating
3. Determine rock property constants s and m
4. Calculate nominal bearing resistance of bedrock at each substructure, qn, using RMR/GSI method in Wyllie "Foundations on Rock".

References

1. AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 and 7th Edition, 2014

2. Wyllie, Duncan C, "Foundations on Rock", Second Edition, 2009.

3. "The Hoek-brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

1. Bedrock Properties from Borings

BB-MDR-103, Coarse grained Granodiorite and Diorite, hard, fresh to slightly weathered and fractured, joint set at low to moderately dipping angles, closely spaced, open to tight, slightly weathered surfaces.
RQD = 62% (RI).

BB-MDR-103A, Granodiorite, coarse grained, moderately hard to very weathered, with fractured zones, joint set at low to moderately dipping angles, close, open, with sand and silt infilling.
RQD = 17 and 0% (RI and R2). R3 is smilar, except less fractured and less weathered, and transitions back to Diorite. RQD=29%.

BB-MDR-202, Fine to medium grained, Diorite, hard, very slightly weathered, joint set at low angles, mod. closely spaced, tight.
RQD = 87% and 62% (RI and R2).

BB-MDR-203A, Fine to medium grained, Diorite, hard, very slightly weathered to fresh, joint set at low angles to horizontal, mod. close, tight.
RQD = 57% and 48% (RI and R2).

Average RQD's of R1 core runs is approximately 55%.

Average of all core runs is 45%

Compressive Strength

No UCT Tests conducted on rock samples

Estimated range of compressive strengths - Ref: Standard Specifications for Highway Bridges 17th Ed. 2002, Table 4.4.8.1.2B

Quartzdiorite Co = 200 - 2,100 ksf or 1,400 - 14,000 psi
Use 7,000 psi

$q_{uc} := 7000 \cdot psi$

B. Determination of Rock Mass Rating (RMR) from LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

From AASHTO - RMR is determined as the sum of five relative ratings listed in LRFD Table 10.4.6.4-1

1. Strength of intact rock

From Standard Specifications for Highway Bridges, 17th Edition, 2002
Table 4.4.8.1.2B uniaxail compressive strength - examine values for quartzdiorite:

Quartzdiorite Co = 200 - 2,100 ksf or 1,400 - 14,000 psi
Use 7,000 psi

Use qu = 7,000 psi = 1008 ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 520-1080 ksf: Relative Rating = 4

2. Drill Core Quality using average of all R1's

Bedrock RQD = 55% (fair) based on average of R1's. From LRFD Table 10.4.6.4-1,
Select Rating between 13 (for RQD 50% - 75%) and 8 (for RQD 25 - 50%); Relative Rating = 10

3. Spacing of joints

From LRFD Table 10.4.6.4.-1 Spacing of joints 2 in. - 1 ft; Relative Rating = 10

4. Condition of joints

Assume slightly rough surfaces, separation <0.05 in., soft joint wall rock has a Relative Rating = 12. (Noteworthy is BB-MDR-103 and 103A); use a Rating of 15 (between rating for hard joint walls and soft joint walls)

5. Groundwater conditions

General Conditions = Water under moderate pressure (Relative Rating 4) to Moist only (Relative Rating 7)

Relative Rating = 5

6. From LRFD Table 10.4.6.4.-2 Geomechanics Rating Adjustment for Joint Orientations

Stike and dip orientations of joints are Fair (-7) use Relative Rating = -7

ADJUSTED RMR

$$RMR := 4 + 10 + 10 + 15 + 5 - 7$$

$$RMR = 37$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - E = Coarse grained polymineralic igneous & metamorphic crystalline rocks

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.5.4-3, RMR = 37 is indicative of Poor Quality Rock Mass.

3. Rock Property Constants s and m (Ref. #1 and #3)

Direct calculation of m ans s is required, Reference 3 (Hoek and Brown, 1988), Table 1.

For a disturbed rock mass: m/mi = exp ((RMR-100)/14)

s = exp ((RMR-100)/6)

mi = m for intact rock

For rock type E, for intact rock, RMR=100, mi = 25 (Ref. # 4, Table 1)

$$m_i := 25$$

$$m := m_i \cdot \exp\left(\frac{RMR - 100}{14}\right)$$

$$m = 0.278$$

$$s := \exp\left(\frac{RMR - 100}{6}\right)$$

$$s = 2.753645 \times 10^{-5}$$

Check with upper and lower bounds:

Fair Qualtiy Rock Mass, RMR = 44 m= 0.458 s= 0.00009

Poor Quality Rock Mass, RMR = 23 m= 0.102 s= 3 x 10-6

4. Nominal and Factored Bearing Resistance of Bedrock

Correction Factor for Foundation Shape, from Wyllie Table 5.4 Pg. 138 (Ref. #3)

$C_{f1} := 1.0$ Conservative selection of Cfl = 1.0 for L/B>6

Uniaxial Compressive Strength

$q_{uc} := 7000 \cdot psi$

Nominal Bearing Resistance (Wyllie)

Reference #3: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot q_{uc} \cdot \left[1 + \sqrt{m \cdot \left(s^{\frac{-1}{2}}\right) + 1}\right]$$

$q_n = 44 \cdot ksf$

Factored Bearing Resistance for Strength Limit State

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$\phi_{bc} := 0.45$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 20 \cdot ksf$ Strength Limit State

Factored Bearing Resistance for Extreme LImit State

Use a bearing resistance factor of 0.80

$\phi_{bc} := 0.80$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 35 \cdot ksf$ Extreme Limit State

Nominal Bearing Resistance (Carter and Kulhawy (1988)

Reference : NCHRP, Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, pg 40, Eq. 82b, and refered to in LRFD C.10.6.3.2.2. Same equation.

$q_n := q_{uc} \cdot \left[\sqrt{s} + \sqrt{m \cdot \left(\sqrt{s}\right) + s}\right]$

$q_n = 44 \cdot ksf$

Factored Bearing Resistance for Strength Limit State

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$\phi_{bc} := 0.45$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 20 \cdot ksf$ Strength Limit State

Factored Bearing Resistance for Extreme LImit State

Use a bearing resistance factor of 0.80

$\phi_{bc} := 0.80$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 35 \cdot ksf$ Extreme Limit State

Analysis

Calculation of nominal and factored bearing resistance on rock for Service Limit State Analysis

Approach 1

Per AASHTO LRFD 10.6.2.4.4 - Settlement of Footings on Rock, "For footings bearing on fair to very good rock according to Geomechanics Classification system, as defined in Article 10.4.6.4, and designed in accordance with the provisions of this Section, elastic settlement may generally be assumed to be less than 0.5 inch."

Alternative Method

LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures* , Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

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Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16-24 ksf
AASHTO Recommended Value	20 ksf

MaineDOT recommended value $q_{nominal} := 24 \cdot ksf$

Resistance Factor for Service Limit State $\phi_r := 1.0$

Per LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values for the factored bearing resistance for Service Limit State Analyses, settlement is typically limited to 1 inch

$$q_{factored} := \phi_r \cdot q_{nominal}$$

$$q_{factored} = 24 \cdot ksf$$

Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi_1 := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Active Earth Pressure for Abutments and Wingwalls

Rankine Theory

Either Rankine or Coulomb may be used for **long heeled** cantilever walls, where the failure surface is uninterupted by the top of the wall stem. In general, use Rankine though. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot deg - \frac{\phi_1}{2}\right)^2 \qquad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot deg$

$$K_{aslope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \qquad K_{aslope} = 0.307$$

- Pa is oriented at an angle of β to the vertical plane

Coulomb Theory

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface in restricted by the top of wall - the wedge of soil does not move.

Angle of back face of wall to the horizontal, θ :

$\theta := 90 \cdot deg$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" δ = 17 to 22 degrees; select 20 degrees.

$\delta := 20 \cdot deg$ for a gravity shaped wall where the interface friction is between soil and concrete

to $\delta := 24 \cdot deg$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, δ=1/3 to 2/3 Φ

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot deg$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{ac} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2} \qquad K_{ac} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface or 'pressure surface.'

At-rest Earth Pressure

Reference LRFD Article 3.11.5.2

There is no estimation of at-rest earth pressure which considers sloped backfill.

- For vertical walls with level backslope

$$K_o := 1 - \sin(\phi_1) \qquad K_o = 0.47$$

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

From Design Freezing Index Map: **Meddybemps, Maine**
DFI = 1450 degree-days.
Case 1 - coarse grained fills above the watertable W=10% .

Interpolate between frost depth of 79.2 inches at 1400 DFI and 82.1 inches at 1500 DFI

Depth of Frost Penetration =

$$d := \frac{82.1 - 79.2}{100} \cdot 50 \cdot in + 79.2 \cdot in \qquad d = 6.721 \cdot ft$$

Method 2 - ModBerg Software

Examine potential foundations placed on coarse-grained fills above the watertable; use ModBerg weather database information for Ellsworth which is on a DFI contour similar to Meddybemps

--- ModBerg Results ---

Project Location: Ellsworth, Maine

Air Design Freezing Index = 1256 F-days
N-Factor = 0.80
Surface Design Freezing Index = 1005 F-days
Mean Annual Temperature = 44.6 deg F
Design Length of Freezing Season = 126 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	66.3	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 = 5.53 ft = 66.3 in.

Recommendation: use average of 6 feet for design of foundations constructed on native soils or granular fill soils

Date: 11/2014
By: L. Krusinski
Check by: KM
Date Reviewed: 11/2014

Site Classification

BB-MDR-101					BB-MDR-102					BB-MDR-103 and -103A					BB-MDR-201					BB-MDR-202					BB-MDR-203				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
1	10	Topsoil	0.5	0.05	1	17	topsoil	2	0.12	1	5	Topsoil	2	0.40	1	5	topsoil	3.5	0.70	1	8	sands	4	0.50	1	8	sand	2	0.25
5.5	25	Sands	8.9	0.36	5	50	sand	7.1	0.14	6	35	Granular	9.5	0.27	6	13	silt	5	0.38	5.5	50	Sands	8.3	0.17	6	50	cobbles	7.5	0.15
										13	50	Till	0.8	0.02	10	50	till	3.3	0.07						11	50	till	5.5	0.11
SUM			**9.4**	**0.41**				**9.1**	**0.26**	**SUM**			**12.3**	**0.69**				**11.8**	**1.15**	**SUM**			**12.3**	**0.67**				**15**	**0.51**
			di/di/N	**23.15**				**di/di/N**	**35.05**				**di/di/N**	**17.89**				**di/di/N**	**10.26**				**di/di/N**	**18.47**				**di/di/N**	**29.41**

Note: Weight of rod (WOR) and weight of hammer (WHO) values are taken as N=1.

SUM	Nav.	22.37

Conclusion: Site Class D - Site Class conservatively evaluated using only the N values and thickness of soil above rock thereby reducing the effective thickness of the profile and neglecting bedrock in the upper 100 feet

Site Classification per LRFD Table C310.3.1-1 - Method B

Zip Code - 04657
Zip Code Latitude = 45.010700
Zip Code Longitude = -067.379000
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.085	PGA - Site Class B
0.2	0.165	Ss - Site Class B
1.0	0.042	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04657
Zip Code Latitude = 45.010700
Zip Code Longitude = -067.379000
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	0.136	As - Site Class D
0.2	0.264	SDs - Site Class D
1.0	0.100	SD1 - Site Class D