STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Letter of Transmittal

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Soils Report No.: 2015-13

Bridge #: 2981

Route: 105

WIN: 020491.00

Town: Camden

Attached is one (1) copy of Soils Report No. 2015-13, dated: June 22, 2015.

This report is available in TEDOCS as Document # 1470061.

att: 1 of 2015-13

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL REPORT

For the Superstructure Replacement and
Abutment Rehabilitation of:

BAKERY BRIDGE
STATE ROUTE 105
OVER MEGUNTICOOK RIVER
CAMDEN, MAINE

Prepared by:
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WIN 20491.00

Soils Report No. 2015-13
Bridge No. 2981

Fed No. STP-2049(100)
June 22, 2015

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GEOTECHNICAL SUMMARY

The purpose of this Geotechnical Report is to present subsurface information and make geotechnical recommendations for the rehabilitation of the existing concrete abutments, removal of the existing center pier and placement of precast concrete cable mats for scour protection at the Bakery Bridge on State Route 105 over Megunticook River in Camden, Maine. The following recommendations are discussed in detail in Section 6.0 of this report:

Abutment Rehabilitation and Reuse – The rehabilitation of the existing abutments will consist of:

- the removal of one (1) foot of concrete from the face of each abutment,
- placement of a mat of reinforcing steel (#5 bars at 12 inch spacing) doweled into the existing abutments,
- replacement of the concrete on the face of each abutment, and
- replacement of the bridge seats.

The existing abutments should be checked to insure that they meet current AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD) standards for sliding, bearing resistance, eccentricity and stability. The rehabilitated abutments shall be proportioned for all applicable load combinations specified in LRFD and shall be evaluated for all relevant strength, extreme and service limit states. Additional lateral earth pressure due to construction surcharge or live load surcharge is for abutments if an approach slab is not specified. Bridge seat modifications at both abutments includes placement of concrete to rehabilitate the existing bridge seats. The new bridge seat concrete will be doweled in to the existing abutments both vertically and horizontally.

Sliding - It is assumed that existing Abutment No. 1 is founded on native granular soils and existing Abutment No. 2 is founded on bedrock. Resistance factors for sliding analyses for both abutments are provided in Section 6.1.1, Table 2. Maximum friction coefficients for sliding analyses for both abutments are provided in Section 6.1.1, Table 3.

Bearing Resistance and Eccentricity – The existing abutments should be evaluated to insure that they will meet current LRFD standards against bearing capacity failure after the abutment rehabilitation. Application of permanent and transient loads is specified in LRFD. The stress distribution at Abutment No. 1 may be assumed to be a linearly distributed pressure over the effective base. The stress distribution at Abutment No. 2 may be assumed to be a triangular or trapezoidal distribution over the effective base. The bearing resistances for the existing abutments for all limit states are presented in Section 6.1.2, Table 5. The eccentricity limits for each abutment are presented in Section 6.1.2, Table 6.

Precast Concrete Cable Mats – Precast concrete cable mats shall be designed and placed in accordance with Special Provision 502 – Precast Block Mat. Precast concrete cable mats shall be underlain by a geotextile meeting the requirements of a Class 1 non-woven fabric. The top surface of the precast concrete cable mats shall match the existing streambed. The Contractor’s work shall not undermine or otherwise threaten the stability of the existing bridge foundations or the adjacent building foundations.

Construction Considerations – Rehabilitation of the existing abutments will require soil excavation and partial or full removal of the roadway approach fill. Construction activities may require earth support systems. Construction activities will include removal of the existing pier to no deeper than approximately elevation 24 feet (NAVD88). Pier removal activities shall be conducted with care so not to disturb, undermine or compromise the existing abutment foundations. This should be noted on the Plans.

There is potential for the adjacent building foundations to be impacted by the rehabilitation activities. A preconstruction survey of the adjacent building foundations should be conducted by the Contractor in order to establish their condition prior to construction.

1.0 INTRODUCTION

The purpose of this Geotechnical Report is to present subsurface information and make geotechnical recommendations for the rehabilitation of the existing concrete abutments, removal of the existing center pier and placement of precast concrete cable mats for scour protection at the Bakery Bridge on State Route 105 over Megunticook River in Camden, Maine. A subsurface investigation has been completed at the site. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for rehabilitation of the existing abutments. This report presents the soils and bedrock information obtained at the site during the subsurface investigation, rehabilitation recommendations, geotechnical design recommendations and construction recommendations.

The existing Bakery Bridge was built in 1933 and is an approximately 47 foot long, two-span, cast-in-place concrete superstructure founded on concrete abutments and a center, concrete pier. The 2014 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports assign the existing substructures a condition rating of 6 (satisfactory) with a Bridge Sufficiency Rating of 49.6. The Inspection Notes state that the existing abutments have moderate scaling and spalling. There are building foundations immediately adjacent to the bridge foundations both upstream and downstream. The bridge wingwalls are tied into the adjacent building foundations and the canal walls. Megunticook River is dam controlled with two (2) dams upstream and two (2) dams downstream. At the bridge location, the Megunticook River flows in a canal originally built for the Penobscot Woolen Mill.

The proposed bridge rehabilitation project will consist of bridge superstructure replacement with rehabilitation of the existing abutments and removal of the existing pier. Precast cable mats will be placed in the riverbed for scour protection of the rehabilitated abutments. The rehabilitation is estimated to have a service life of 50 years. The bridge will be closed during construction.

2.0 GEOLOGIC SETTING

Bakery Bridge on State Route 105 in Camden crosses the Megunticook River 200 feet northwest of US Route 1 as shown on Sheet 1 - Location Map.

According to the Camden Quadrangle, Maine Surficial Geologic map published by the Maine Geological Survey Open File No. 10-6 (2010) the surficial soils in the vicinity of the site consist of Presumpscot Formation deposits. These deposits generally consist of glaciomarine silt, clay, and sand deposited on the late-glacial sea floor.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, the bedrock in the vicinity of the site is identified as Ordovician-Cambrian pelite of the Megunticook Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings at the site. Test boring BB-CMR-101 was drilled at the location of the existing north abutment. Test boring BB-CMR-102 was at the location of the existing south abutment. These boring locations are shown in Sheet 2 - Boring Location Plan & Interpretive Subsurface Profile with Boring Logs. The borings were drilled on September 23, 2014 by the MaineDOT Materials Testing and Exploration drill crew using a trailer mounted drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on Sheet 2 - Boring Location Plan & Interpretive Subsurface Profile with Boring Logs.

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 MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in October 2014 and was found to deliver approximately 51.3 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.908 to the raw field N-values. This hammer efficiency factor, 0.908, and both the raw field N-value and the corrected N-value (N_{60}) are shown on the boring logs. The bedrock was cored in both borings using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated for the NQ cores.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs and identified field testing requirements. The MaineDOT Subsurface Inspector certified by the Northeast Transportation Technical Certification Program (NETTCP) logged the subsurface conditions encountered at the borings. The borings were located in the field by taping to site features after completion of the drilling program.

4.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the borings consisted of fill underlain by sand overlying bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on Sheet 2 - Boring Location Plan & Interpretive Subsurface Profile with Boring Logs. A brief summary description of the strata encountered is as follows:

4.1 Fill

A layer of fill was encountered beneath the pavement in both of the borings. The thickness of the fill layer in the borings ranged from approximately 7.5 to 8.0 feet. The fill is described as brown, damp, fine to coarse sand with little to some gravel, little to some silt.

Corrected SPT N-values in the fill ranged from 8 to 18 blows per foot (bpf) indicating that the fill is loose to medium dense in consistency.

4.2 Native Sand

A layer of native sand was encountered below the fill in both of the borings. The thickness of the native sand layer in the borings ranged from approximately 3.6 to 10.7 feet. The native sand is described as:

- Brown, wet, fine to coarse sand, some gravel, some silt, occasional cobbles and
- Grey, wet, silty fine to coarse sand, little gravel.

Corrected SPT N-values in the native sand ranged from 9 to 54 blows per foot (bpf) indicating that the sand is loose to very dense in consistency.

4.3 Bedrock

The bedrock was cored both of the borings. Table 1 summarizes approximate depths to bedrock, corresponding top of bedrock elevations and RQD at the boring locations:

Boring Number Substructure	Approximate Depth to Intact Bedrock	Approximate Bedrock Elevation	Estimated RQD
BB-CMR-101 Abutment No. 1	19.1 feet	16.1 feet (weathered bedrock) 15.7 feet (intact bedrock)	50%
BB-CMR-102 Abutment No. 2	12.0 feet	23.7 feet (weathered bedrock) 22.8 feet (intact bedrock)	76%

Table 1 – Summary of Approximate Bedrock Depths, Bedrock Elevations and RQD

Weathered bedrock was encountered at the bedrock surface in both borings. The thickness of the weathered bedrock ranged from 0.4 feet in boring BB-CMR-101 to 0.9 feet in boring BB-CMR-102. The bedrock is identified as dark grey, fresh, meta-pelite, with pyrite, garnet and kyanite crystals, joints dipping approximately 70 to 80 degrees and sub-horizontal joints at 30 to 45 degrees. The bottom third of the core shows more oxidation and fragmentation than the rest. The RQD of the bedrock ranged from 50 to 76% indicating a Rock Mass Quality of poor to good.

4.4 Groundwater

Groundwater was not observed in the borings. Groundwater levels will fluctuate, with changes in the water levels in the river, seasonal changes, precipitation, runoff and adjacent construction activities.

5.0 PROJECT ALTERNATIVES

This project was originally programed as a bridge replacement project. During development of the Preliminary Design Report (PDR), three (3) replacement options and one (1) rehabilitation option were considered by the MaineDOT Bridge Program. All of the replacement options proposed the use of a single span superstructure with full height, mass concrete abutments founded on spread footings on bedrock.

Due to the high cost of full replacement and the relatively good condition of the existing abutments the project scope was changed to superstructure replacement and abutment rehabilitation. The existing center pier will be removed as a part of the proposed project. The rehabilitation of the abutments will consist of vertical surface repair and bridge seat replacement. Precast cable mats are proposed in the river in front of the abutments for scour protection. The rehabilitation is estimated to have a service life of 50 years.

6.0 GEOTECHNICAL RECOMMENDATIONS

The following subsections will discuss geotechnical recommendations for the rehabilitation of the existing concrete abutments, removal of the existing center pier, and placement of precast concrete cable mats for scour protection at the Bakery Bridge. The rehabilitation of the abutments will consist of vertical surface repair and bridge seat replacement. Precast cable mats are proposed in the river in front of the abutments for scour protection. The design recommendations in this Section are in accordance with AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD), Bridge Design Guide (BDG) Section 10.6 – Substructure Rehabilitation and BDG Section 10.7 – Substructure Reuse.

6.1 Abutment Rehabilitation and Reuse

The rehabilitation of the existing abutments will consist of:

- the removal of one (1) foot of concrete from the face of each abutment,
- placement of a mat of reinforcing steel (#5 bars at 12 inch spacing) doweled into the existing abutments,
- replacement of the concrete on the face of each abutment, and
- replacement of the bridge seats.

The rehabilitation is estimated to have a service life of 50 years.

The existing abutments should be evaluated to insure that they meet current LRFD standards for sliding, bearing resistance, eccentricity and stability. The rehabilitated abutments shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be evaluated for all relevant strength, extreme and service limit states. LRFD Figures C11.5.6-1 and C11.5.6-2 illustrate the typical load factors to produce the extreme factored effect for sliding, bearing resistance and eccentricity.

6.1.1 Sliding

Based on the existing bridge plans and the borings conducted at the site it is assumed that existing Abutment No. 1 is founded on native granular soils and existing Abutment No. 2 is founded on bedrock. Table 2 presents the resistance factors for sliding analyses, φ_τ, for both abutments.

Substructure	Assumed Bearing Material	Condition	Limit State	Sliding Resistance Factor φ_τ	LRFD Reference
Abutment No. 1	Granular soils	Cast-in-place concrete on sand	Strength	0.80	Table 10.5.5.2.2-1
			Service	1.0	Article 10.5.5.1
			Extreme	1.0	Article 10.5.5.3
Abutment No. 2	Bedrock	Cast-in-place concrete on bedrock	Strength	0.90	FHWA Guidance
			Service	1.0	Article 10.5.5.1
			Extreme	1.0	Article 10.5.5.3

Table 2 – Resistance Factors for Sliding

Sliding analyses for resistance of both abutment footings to lateral loads shall be calculated using the maximum friction coefficients provided in Table 3:

Substructure	Assumed Bearing Material	Limit State	Friction Angle δ	Coefficient of Friction tan δ (dim.)	LRFD Reference
Abutment No. 1	Granular soils	All	28°	0.53	Table 3.11.5.3-1
Abutment No. 2	Bedrock	All	31°	0.60	Table 3.11.5.3-1

Table 3 – Maximum Friction Coefficients

Passive earth pressure due to streambed soils in front of the abutment footings shall be neglected in the sliding analyses.

Additional lateral earth pressure due to construction surcharge or 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 on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 4:

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

Table 4 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Bridge seat modifications at both abutments includes placement of concrete to rehabilitate the existing bridge seats. The new bridge seat concrete will be doweled in to the existing abutments both vertically and horizontally.

6.1.2 Bearing Resistance and Eccentricity

Based on the existing bridge plans and the borings conducted at the site it is assumed that existing Abutment No. 1 is founded on native granular soils and existing Abutment No. 2 is founded on bedrock. The existing abutments should be checked to insure that they will continue meet current LRFD standards against bearing capacity failure after the abutment rehabilitation. Application of permanent and transient loads is specified in LRFD Article 11.5.6. The stress distribution at Abutment No. 1 may be assumed to be a linearly distributed pressure over the effective base as shown in LRFD Figure 11.6.3.2-1. The stress distribution at Abutment No. 2 may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. Table 5 summarizes the bearing resistances for the existing abutments:

Substructure	Assumed Bearing Material	Limit State	Resistance Factor φ_b	Factored Bearing Resistance (ksf)	LRFD Reference
Abutment No. 1	Granular soils	Service	1.0	3	Article 10.5.5.1
		Strength	0.45	11	Table 10.5.5.2.2-1
		Extreme	0.8	20	Article C11.5.8
Abutment No. 2	Bedrock	Service	1.0	20	Article 10.5.5.1
		Strength	0.45	57	Table 10.5.5.2.2-1
		Extreme	0.8	101	Article C11.5.8

Table 5 – Bearing Resistances

See Appendix B – Calculations for supporting documentation.

The eccentricity limits for each abutment are presented in Table 6:

Substructure	Assumed Bearing Material	Location of Resultant	LRFD Reference
Abutment No. 1	Granular soils	Within the middle two-thirds (2/3) of the base width	Article 11.6.3.3
Abutment No. 2	Bedrock	Within the middle nine-tenths (9/10) of the base width	Article 11.6.3.3

Table 6 – Eccentricity Limits

6.2 Precast Concrete Cable Mats

Precast concrete cable mats will be placed in the river at the locations shown on the Plans. Precast concrete cable mats shall be designed and placed in accordance with Special Provision 502 – Precast Block Mat which is included in Appendix C.

The minimum concrete strength for the precast blocks shall be 4000 psi at 28 days. Precast concrete cable mats shall be underlain by a geotextile meeting the requirements of a Class 1 non-woven fabric specified in Standard Specification 722.03.

The top surface of the precast concrete cable mats shall match the existing streambed. The Contractor's work shall not undermine or otherwise threaten the stability of the existing bridge foundations or the adjacent building foundations. Dredge generated from the placement of the precast concrete cable mats shall be beneficially reused on site or disposed of properly.

6.3 Construction Considerations

Rehabilitation of the existing abutments will require soil excavation and partial or full removal of the roadway approach fill. Construction activities may require earth support systems.

Construction activities will include removal of the existing pier to no deeper than approximately elevation 24 feet (NAVD88). Pier removal activities shall be conducted with care so not to disturb, undermine or compromise the existing abutment foundations. This should be noted on the Plans.

There is potential for the adjacent building foundations to be impacted by the rehabilitation activities. This condition should be noted on the Plans. A preconstruction survey of the adjacent building foundations should be conducted by the Contractor in order to establish their condition prior to construction.

The roadway approach fill soils may be saturated and water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

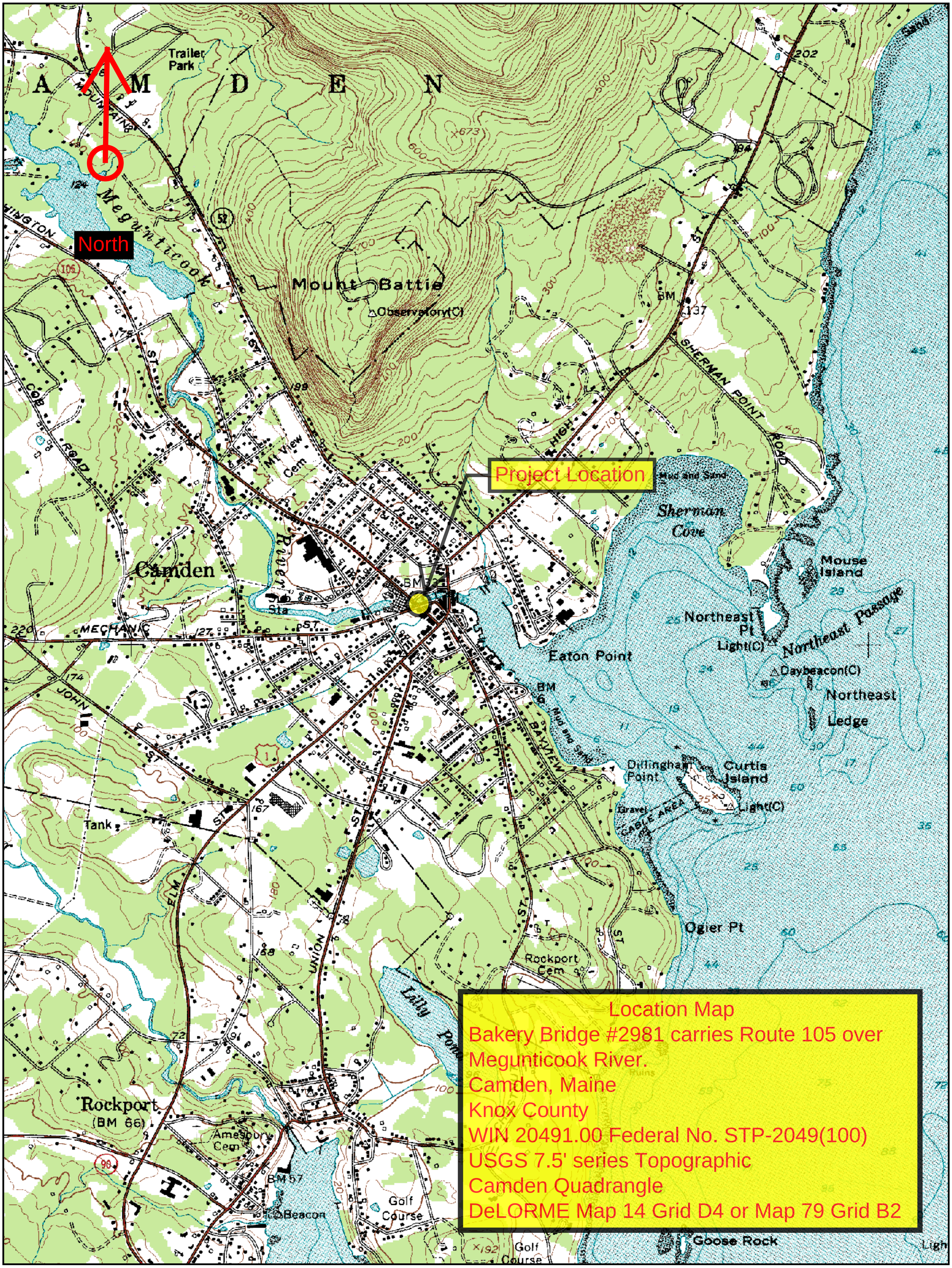
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the newly constructed approach. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

7.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed rehabilitation of the existing concrete abutments, removal of the existing center pier and placement of precast concrete cable mats for scour protection at the Bakery Bridge in Camden, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. 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.

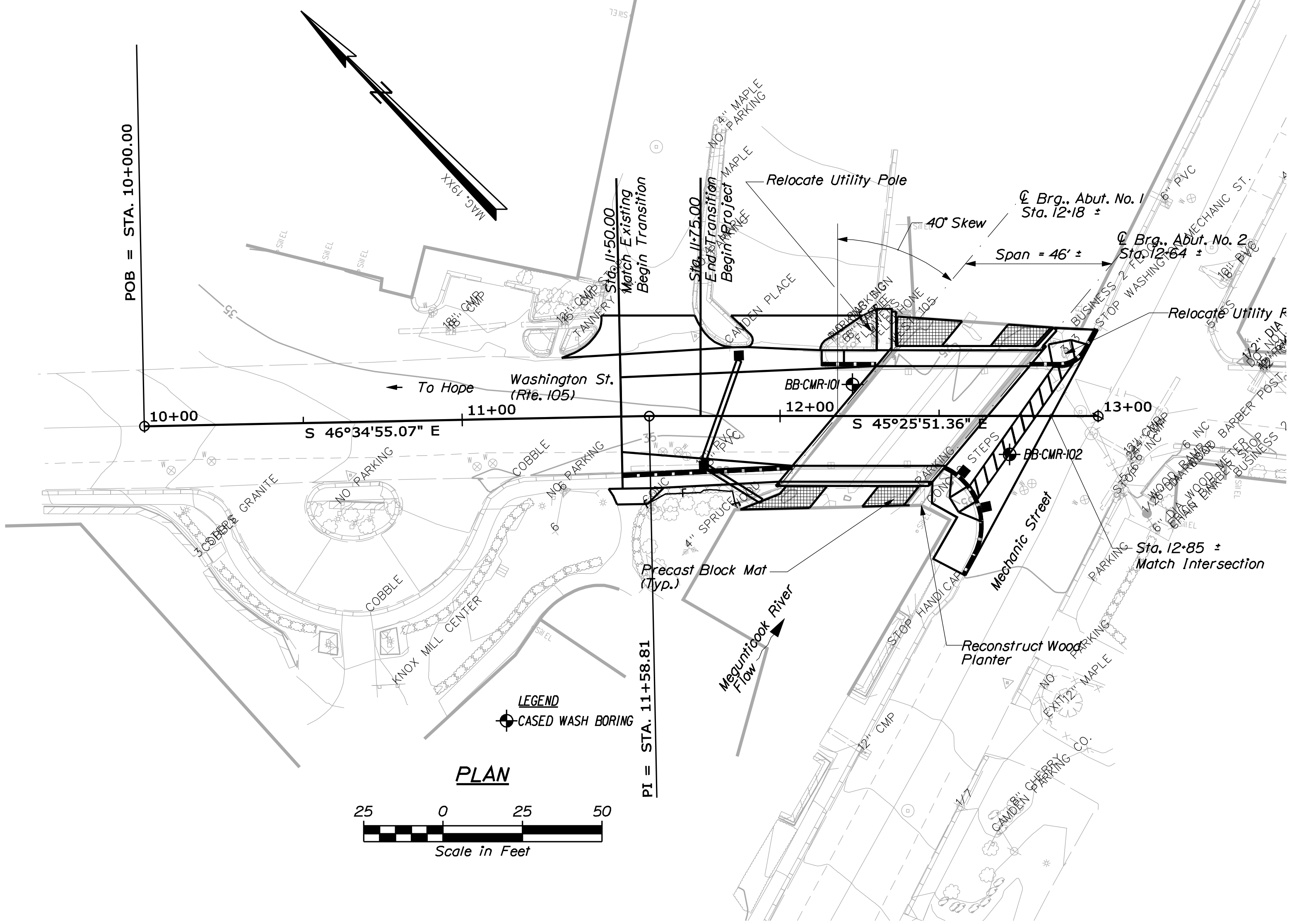
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

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.



PLAN

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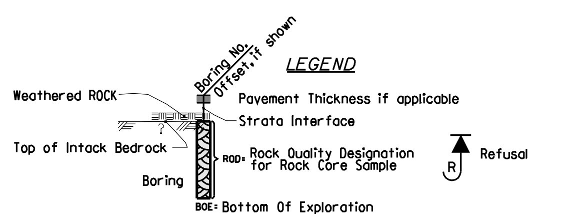

PROFILE

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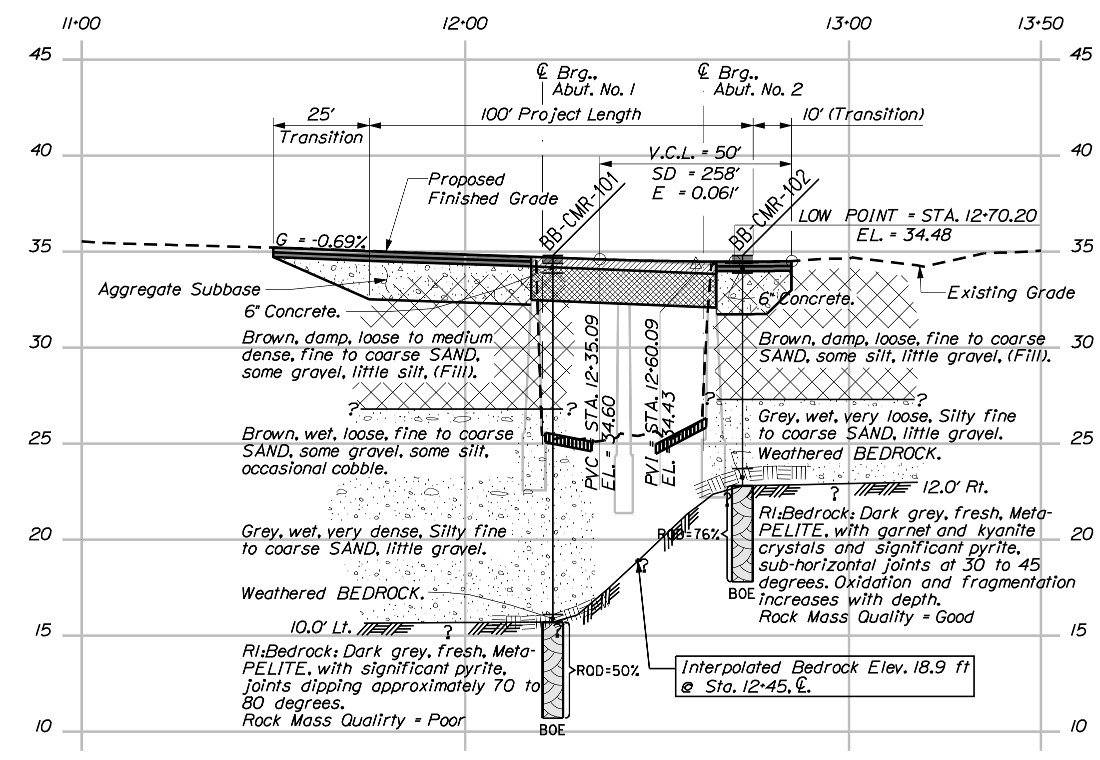

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably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

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

Project: Bakery Bridge #2981 carries Route 105 over Megunticook River		Location: Camden, Maine		Boring No.:	BB-CMR-101
				WIN:	20491.00
Driller:	MaineDOT	Elevation (ft.)	34.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/23/2014; 08:00-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+22.8, 10.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	34.38	5" Pavement (0.42)	
								33.88	6" Concrete (0.92)	
	1D	24/8	2.00 - 4.00	2/2/4/3	6	9			Brown, damp, loose, fine to coarse SAND, some gravel, little silt (Fill).	
5	2D	24/17	5.00 - 7.00	5/5/7/6	12	18			Similar to above, except medium dense.	
								26.80	(8.00)	
10	3D	24/13	10.00 - 12.00	2/2/4/6	6	9	5		Brown, wet, loose, fine to coarse SAND, some gravel, some silt, occasional cobble.	
							9			
							24			
							42			
	4D	24/10	14.00 - 16.00	23/26/10/7	36	54	26		Grey, wet, very dense, Silty, fine to coarse SAND, little gravel.	
15							29			
							31			
							100			
							250	16.10	(18.70)	
	MD/R1	60/60	19.10 - 24.10	RQD = 50%			a50 NQ-2	15.70	Weathered BEDROCK. a50 blows for 0.1 ft. (19.10) Failed sample attempt. Top of Intact Bedrock at Elev. 15.7 ft. R1:Bedrock: Dark grey, fresh, Meta-PELITE, with significant pyrite, joints dipping approximately 70 to 80 degrees. Rock Mass Quality = Poor R1:Core Times (min:sec) 19.1-20.1 ft (3:07); 20.1-21.1 ft (4:28); 21.1-22.1 ft (5:39); 22.1-23.1 ft (4:45); 23.1-24.1 ft (5:25) 100% Recovery	
20										
								10.70	(24.10) Bottom of Exploration at 24.10 feet below ground surface.	
25										

Remarks:

Left 5.0 ft of casing in borehole.
300-400# down pressure on Core Barrel.

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-CMR-101

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

Project: Bakery Bridge #2981 carries Route 105 over Megunticook River		Location: Camden, Maine		Boring No.:	BB-CMR-102
				WIN:	20491.00
Driller:	MaineDOT	Elevation (ft.)	34.8	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	9/23/2014; 11:30-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	12+72.2, 12.0 ft Rt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.908		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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WC = water content, percent
LL = Liquid Limit
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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	34.38	5" Pavement (0.42)	
								33.88	6" Concrete (0.92)	
5	1D	24/18	5.00 - 7.00	2/2/3/4	5	8			Brown, damp, loose, fine to coarse SAND, some silt, little gravel, (Fill).	
								27.30	(7.50) Roller Coned ahead from 7.5-10.5 ft bgs.	
10	2D	7.2/4	10.50 - 11.10	5/40(1.2")	---		a100		a100 blows for 0.5 ft. Grey, wet, very dense, Silty fine to coarse SAND, little gravel.	
								23.70	(11.10) Roller Coned ahead from 11.1-12.0 ft bgs. Weathered BEDROCK.	
	R1	60/46	12.00 - 17.00	RQD = 76%			NQ-2	22.80	(12.00) Top of Intact Bedrock at Elev. 22.8 ft. R1:Bedrock: Dark grey, fresh, Meta-PELITE, with garnet and kyanite crystals, and significant pyrite, sub-horizontal joints at 30 to 45 degrees. Oxidation and fragmentation increase with depth. Rock Mass Quality = Good. R1:Core Times (min:sec) 12.0-13.0 ft (3:07) 13.0-14.0 ft (4:30) 14.0-15.0 ft (4:40) 15.0-16.0 ft (5:20) 16.0-17.0 ft (5:20) 76% Recovery	
15										
								17.80	(17.00) Bottom of Exploration at 17.00 feet below ground surface.	
20										
25										

Remarks:

Left 14" of Cone in borehole, core barrel lift broke drilling through bent shoe, core barrel broke.

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-CMR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2049(100)
WIN 20491.00
BRIDGE PLANS
BRIDGE NO. 2981

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	M.MAGUIRE / T.WHITE	MAR 2015
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

CAMDEN BAKERY BRIDGE
MEGUNTICOOK RIVER
KNOX COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE WITH BORING LOGS

SHEET NUMBER
2
OF 2

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bakery Bridge #2981 carries Route 105 over Megunticook River
Location: Camden, Maine

Boring No.: BB-CMR-101
WIN: 20491.00

Driller:	MaineDOT	**Elevation (ft.)**	34.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2014; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+22.8, 10.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	34.38 33.88		5" Pavement —0.4 6" Concrete —0.9	
	1D	24/8	2.0 - 4.0	2/2/4/3	6	9				Brown, damp, loose, fine to coarse SAND, some gravel, little silt (Fill).	
5	2D	24/17	5.0 - 7.0	5/5/7/6	12	18				Similar to above, except medium dense.	
								26.80		—8.0	
10	3D	24/13	10.0 - 12.0	2/2/4/6	6	9	5			Brown, wet, loose, fine to coarse SAND, some gravel, some silt, occasional cobble.	
							9				
							24				
							42				
	4D	24/10	14.0 - 16.0	23/26/10/7	36	54	26			Grey, wet, very dense, Silty, fine to coarse SAND, little gravel.	
15							29				
							31				
							100				
							250	16.10		—18.7	
	MD/R1	60/60	19.1 - 24.1	RQD = 50%			a50 NQ-2	15.70		Weathered BEDROCK. a50 blows for 0.1 ft. —19.1	
20										Failed sample attempt. Top of Intact Bedrock at Elev. 15.7 ft. R1:Bedrock: Dark grey, fresh, Meta-PELITE, with significant pyrite, joints dipping approximately 70 to 80 degrees. Rock Mass Quality = Poor R1:Core Times (min:sec) 19.1-20.1 ft (3:07); 20.1-21.1 ft (4:28); 21.1-22.1 ft (5:39); 22.1-23.1 ft (4;45); 23.1-24.1 ft (5:25) 100% Recovery	
								10.70		—24.1 **Bottom of Exploration at 24.10 feet below ground surface.**	
25											

Remarks:

Left 5.0 ft of casing in borehole.
300-400# down pressure on Core Barrel.

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: Bakery Bridge #2981 carries Route 105 over Megunticook River
Location: Camden, Maine

Boring No.: BB-CMR-102
WIN: 20491.00

Driller:	MaineDOT	**Elevation (ft.)**	34.8	**Auger ID/OD:**	5" Solid Stem
Operator:	Giles/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/23/2014; 11:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	12+72.2, 12.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.908	**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	34.38		5" Pavement (0.4)	
								33.88		6" Concrete (0.9)	
5	1D	24/18	5.0 - 7.0	2/2/3/4	5	8				Brown, damp, loose, fine to coarse SAND, some silt, little gravel, (Fill).	
								27.30		(7.5) Roller Coned ahead from 7.5-10.5 ft bgs.	
10	2D	7.2/4	10.5 - 11.1	5/40(1.2")	---		a100			a100 blows for 0.5 ft. Grey, wet, very dense, Silty fine to coarse SAND, little gravel.	
								23.70		(11.1) Roller Coned ahead from 11.1-12.0 ft bgs. Weathered BEDROCK.	
	R1	60/46	12.0 - 17.0	RQD = 76%			NQ-2	22.80		(12.0) Top of Intact Bedrock at Elev. 22.8 ft. R1:Bedrock: Dark grey, fresh, Meta-PELITE, with garnet and kyanite crystals, and significant pyrite, sub-horizontal joints at 30 to 45 degrees. Oxidation and fragmentation increase with depth. Rock Mass Quality = Good. R1:Core Times (min:sec) 12.0-13.0 ft (3:07) 13.0-14.0 ft (4:30) 14.0-15.0 ft (4:40) 15.0-16.0 ft (5;20) 16.0-17.0 ft (5:20) 76% Recovery	
15								17.80		(17.0) **Bottom of Exploration at 17.00 feet below ground surface.**	
20											
25											

Remarks:

Left 14" of Cone in borehole, core barrel lift broke drilling through bent shoe, core barrel broke.

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

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

Boring No.: BB-CMR-102

Appendix B

Calculations

Bearing Resistance Existing Abutment No. 1
Spread footing on Native Granular Soils:

Part 1 - Service Limit State

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 7th Edition 2014
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Fine to coarse sand (SM)

Based on an N-value of 9 from Boring BB-CMR-101 - Soils are loose at bearing elevation

Consistency In Place: loose

Bearing Resistance: Ordinary Range (ksf) 2 to 4

Recommended Value of Use: $q_{nom} := 3 \cdot ksf$

Resistance factor at the **service limit state** (LRFD Article 10.5.5.1) $\phi_{service} := 1.0$

$q_{factored\_bc} := q_{nom} \cdot \phi_{service}$ $\qquad$ $q_{factored\_bc} = 3 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - Spread footing on native granular soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. The footing is founded at ~ Elev 22.5

   Ground surface is at ~ Elev 25.0

   $D_{ftg} := 2.5 \cdot ft$
2. Assumed parameters for soils: (Ref: Bowles 5th Ed Table 3-4)

   Saturated unit weight: $\gamma_s := 125 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 32 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Assume Depth the water table: $D_w := 8 \cdot ft$ $\qquad$ Unit Weight of water: $\gamma_w := 62.4 \cdot pcf$

Effective stress at footing bearing level:

$$q_{eff} := D_w \cdot \gamma_s + \left(D_{ftg} - D_w\right) \cdot \left(\gamma_s - \gamma_w\right) \qquad q_{eff} = 0.656 \cdot ksf$$

Assume footing width (B) is: $B := 7 \cdot ft$

Terzaghi Shape factors from Table 4-1 For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For ϕ=32 deg $N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q_{nominal} := c_{ns} \cdot N_c \cdot s_c + q_{eff} \cdot N_q + 0.5\left(\gamma_s\right) B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nominal} = 24.8 \cdot ksf$$

Factored Bearing Resistance for Strength Limit State

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{factored} := q_{nominal} \cdot \phi_b \qquad q_{factored} = 11 \cdot ksf$$

<u>Part 3 - Extreme Limit State</u>

Factored Bearing Resistance for Extreme Limit State

Resistance Factor: $\phi_b := 0.8$ AASHTO LRFD Article 11.5.8

$$q_{factored} := q_{nominal} \cdot \phi_b \qquad q_{factored} = 20 \cdot ksf$$

Bearing Resistance Existing Abutment No. 2 Spread footing on Bedrock:

Part 1 - Service Limit State

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 7th Edition 2014
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Weathered or broken bedrock of any kind, except shale

Based on RQD of bedrock from Boring BB-CMR-102 = 76 percent

Consistency In Place: medium hard rock

Bearing Resistance: Ordinary Range (ksf) 16 to 24

Recommended Value of Use: $q_{nom} := 20 \cdot ksf$

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1) $\phi := 1.0$

$q_{factored\_bc} := q_{nom} \cdot \phi$ $q_{factored\_bc} = 20 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

Part 2 - Strength Limit State

Determine Bearing Resistance using RMR Method

Reference: AASHTO LRFD Bridge Design Specifications 6th Edition 2012
Article 10.4.6.4 Rock Mass Strength

Bedrock at the site is Pelite (siltstone) which was found to be "good" in quality.
RQD of 76% in Boring BB-CMR-102

Determine RMR from Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

From AASHTO - RMR is determined as the sum of the five relative ratings listed in 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 uniaxial compressive strength for Pelite (siltstone) = 200 to 2,500 ksf = 1,400 to 17,000 psi

Use: $q_u := 1200 \cdot ksf$ $q_u = 8333 \cdot psi$

From Table 10.4.6.4.-1:
For Uniaxial Compressive Strength = 1080 to 2160 ksf: Relative Rating = 7

2. Drill Core Quality

Bedrock RQD = 76% (good) From Table 10.4.6.4.-1: RQD 75% to 90%: Relative Rating = 17

3. Spacing of joints

Assume Spacing of 1 to 3 feet From Table 10.4.6.4.-1: Relative Rating = 20

4. Condition of joints

Assume slightly rough surfaces <0.05 in, soft joint wall rock From Table 10.4.6.4.-1: Relative Rating = 12

5. Groundwater conditions

General Conditions = Water under moderate pressure From Table 10.4.6.4.-1: Relative Rating = 4

Raw RMR = 60

Adjustment to RMR for joint Orientations from Table 10.4.6.4-2

Assume Strike and Dip Orientations of Joints = Fair For Foundations: Rating = -7

Adjusted RMR = 53 $RMR := 53$

Determine Rock Mass Class from Adjusted RMR Rating

For Adjusted RMR = 53 From LRFD Table 10.4.6.4.-3: Class No. = III - Fair Rock

Determine Rock Type from LRFD Table 10.4.6.4.-4

Rock Type B - Sitlstone

Determine Rock Property constants *m* and *s:*

Reference: The Hoek and Brown Failure Criterion - a 1988 Update,
15th Canadian Rock Mechanics Symposium

m/m_i= exp ((RMR-100)/14) Eq 18 - for disturbed rock masses

where m_i = m for intact rock $m_i := 10$ From LRFD Table 10.4.6.4-4

$$m_{Efair} := m_i \cdot \exp\left(\frac{RMR - 100}{14}\right) \qquad m_{Efair} = 0.348$$

s = exp ((RMR-100)/6) Eq 19 - for disturbed rock masses

$$s_{Efair} := \exp\left(\frac{RMR - 100}{6}\right) \qquad s_{Efair} = 0.0004$$

<u>Determine nominal and factored bearing resistance of Bedrock:</u>

Foundation Shape correction factor:

$C_{f1} := 1.0$ From Foundations on Rock, Wyllie, Table 5.4 pg 138

Uniaxial Compressive Strength - Sandstone

$$q_{uc} := \begin{pmatrix} 1400 \\ 8333 \\ 15000 \\ 17000 \end{pmatrix} \cdot psi$$

Upper and lower bounds from from Standard Specifications for Highway Bridges
17th Edition - 2002 Table 4.4.8.1.2B

Determine Nominal Bearing Resistance:

From Foundations on Rock, Wyllie, Table 5.4 pg 138

$$q_{nom} := C_{f1} \cdot \sqrt{s_{Efair}} \cdot q_{uc} \cdot \left[1 + \sqrt{m_{Efair} \cdot \left(s_{Efair}^{\frac{-1}{2}} \right) + 1} \right] \qquad q_{nom} = \begin{pmatrix} 21 \\ 127 \\ 228 \\ 258 \end{pmatrix} \cdot ksf$$

<u>Determine Factored Bearing Resistance at the Strength Limit State:</u>

From Table 10.5.5.2.2-1 Resistance factor for footing on rock $\phi_b := 0.45$

The factored resistance $q_R = \phi_b \times q_n$ equation 10.6.3.1.1-1 AASHTO LRFD

$$q_R := \phi_b \cdot q_{nom} \qquad q_R = \begin{pmatrix} 10 \\ 57 \\ 103 \\ 116 \end{pmatrix} \cdot ksf$$

Recommend 57 ksf for Strength Limit State

<u>Determine Factored Bearing Resistance at the Extreme Limit State:</u>

Use a bearing resistance factor of 0.8 for Extreme Limit State for gravity and semigravity walls per LRFD Article C11.5.8.

Resistance factor - $\phi_{bc} := 0.8$

$$q_{rEE} := \phi_{bc} \cdot q_{nom} \qquad q_{rEE} = \begin{pmatrix} 17 \\ 101 \\ 182 \\ 207 \end{pmatrix} \cdot ksf$$

For Gravity and Semigravity Walls

Recommend 101 ksf for Extreme Limit State

Appendix C

Special Provisions

SPECIAL PROVISION
SECTION 502
STRUCTURAL CONCRETE
(Precast Block Mat)

Add the following to the end of Section 502- Structural Concrete:

Description. This work shall consist of excavating, grading, and placing an articulating precast concrete block system hereinafter, Precast Block Mat, designated on the Plans as Precast Block Mat, on designated channels in accordance with these specifications and in reasonably close conformity with the lines, grades and thickness as shown in the Plans or as directed by the Resident. The Contractor shall furnish all labor, materials, equipment, and incidentals required to perform all operations in connection with the installation of the Precast Block Mat. This Precast Block Mat system shall be made up of mattresses of precast concrete blocks and connecting cables with geotextile attached or the precast concrete blocks can be placed on top of geotextile material. The Precast Block Mats are made up of precast concrete blocks interlocked by cables cast within each block, forming an articulating concrete block armor layer. Refer to Plans for approximate limits required. Multiple irregular Precast Block Mat sizes may be designed for side by side placement and clamped together to provide one homogeneous erosion protection system.

Design. The Precast Block Mat system shall be comprised of concrete blocks that are wet-cast. The size of the concrete blocks shall be approximately 15.5 inches square at the base and 11.5 inches square at the top face (a truncated pyramid shape). The height of the block shall be as noted on the Contract Plans. No holes will be allowed in the concrete blocks. The Contractor may submit a site specific design for an alternate size mat. Any alternate design considered shall meet the requirements of the specifications listed herein.

If the required block height is not noted on the Contract Plans than the blocks shall be designed for the following conditions:

Flow	Velocity (ft/s)	Shear Stress (psf)
Q500	15	6.0

Concrete for Precast Block Mat. The minimum required concrete strength is 4000 pounds per square inch (psi) at 28 days. Air entrainment of 4 percent to 7 percent shall also be added. All applicable ASTM standards will be met in the production of the concrete. The finished concrete product shall consist of a minimum density of 140 pounds per cubic foot (lbs/cf) in an average of 3 units. No individual block shall consist of a minimum concrete density lower than 135 lbs/cf.

Individual concrete blocks shall be solid and intact with the stainless steel cables fully imbedded inside. No cracks are allowed in any of the concrete blocks. Repairing of individual concrete blocks is not allowed. The surface of the concrete blocks shall be true and even, free from stone pockets and depressions or projections and of uniform texture. All Precast Block Mats shall be handled, stored and shipped in such a manner as to eliminate the chance of chipping, cracks, fracture and excessive bending stresses. Any units found damaged upon delivery, or damaged after delivery, shall be subject to rejection by the Resident.

Cables. Component cables of the articulating block system shall be constructed of high tenacity, low elongating, and continuous stainless steel aircraft cable of Type 302 or 304. The cable shall be of type 1 x 19 construction. Cable shall be integral (cast into) to the concrete block, and shall traverse through each block in both longitudinal and lateral directions of the Precast Block Mat system.

Geotextile. The geotextile used is to be specified by the manufacturer of the Precast Block Mat. The standard geotextile material used on non-specific projects is a Class I, non-woven fabric meeting the requirements of Standard Specification 722.03. The geotextile fabric can be attached to the bottom of concrete blocks or the geotextile can be placed separately on the prepared subbase prior to the installation of the Precast Block Mat.

Clamps. Stainless steel wire rope or 3/16 inch stainless steel U-type clamps shall be used to secure loops of adjoining Precast Block Mats. Sufficient stainless steel clamps shall be used to secure loops of adjoining Precast Block Mats. The standard placement of clamps shall be placed evenly at 4 foot centers interlocking adjoining Precast Block Mats together. A minimum of two clamps shall be used along the edge of a Precast Block Mat to attach to the adjacent Precast Block Mat. Clamps shall be installed as close to the concrete blocks as possible.

Anchoring. Precast Block Mats shall be anchored in accordance with the manufacturer's recommendations. Anchorage shall be provided along the perimeter of the Precast Block Mat system areas. Anchorage of the leading upstream edge and trailing downstream edge of Precast Block Mat area shall be accomplished by complete burial of at least two entire block rows.

Ground Preparations. The subbase of the Precast Block Mat area shall be clear of all deformities such as roots, grade stakes, debris and large stones. The entire area shall be smooth so that intimate contact with each individual block can be achieved. To obtain required streambed elevations, clean borrow meeting the requirements of Subsection 703.12, Aggregate for Crushed Stone Surface, may be used as a leveling base. Minor excavation and shaping shall be accomplished to the extent required to remove obstructions, to prepare an optimal contact surface for the Precast Block Mat systems and to place the top of Precast Block Mat systems in a way that conforms to the established streambed elevations.

If a very large boulder or obstruction is encountered that cannot practicably be removed than the Contractor can choose between one of the following options:

The top of the obstruction or boulder shall be removed so that the Precast Block Mat can go over the obstruction with a maximum slope of 2 horizontal to 1 vertical. The second option is to trim and/or cut the Precast Block Mat to fit as tightly as possible around the obstruction. The gap between the obstruction and the Precast Block Mat shall be grouted around the entire obstruction. The grout shall completely fill the void space and extend a minimum of one and half blocks on to the Precast Block Mat.

Additionally, the streambed through the bridge site shall be shaped to provide a low flow channel within the stream that will sustain fish passage in low flow conditions. The location of the low flow channel will be determined by the Resident. For a single span bridge, the low flow channel shall be three feet (2 blocks) wide and two feet lower than established streambed. There shall be a 2:1 slope from the bottom of the low flow channel to the established streambed elevation. Diagram of low flow channel configuration can be seen in Figure 1 below. For a multiple span bridge, the low flow channel only needs to be done for one span as determined by the Resident. Once the streambed/ground preparations are complete and the Contractor can demonstrate the Precast Block Mats will be installed at the desired streambed elevations (top and bottom of sag), the streambed/ground preparations shall be approved by the Resident so installation can proceed.

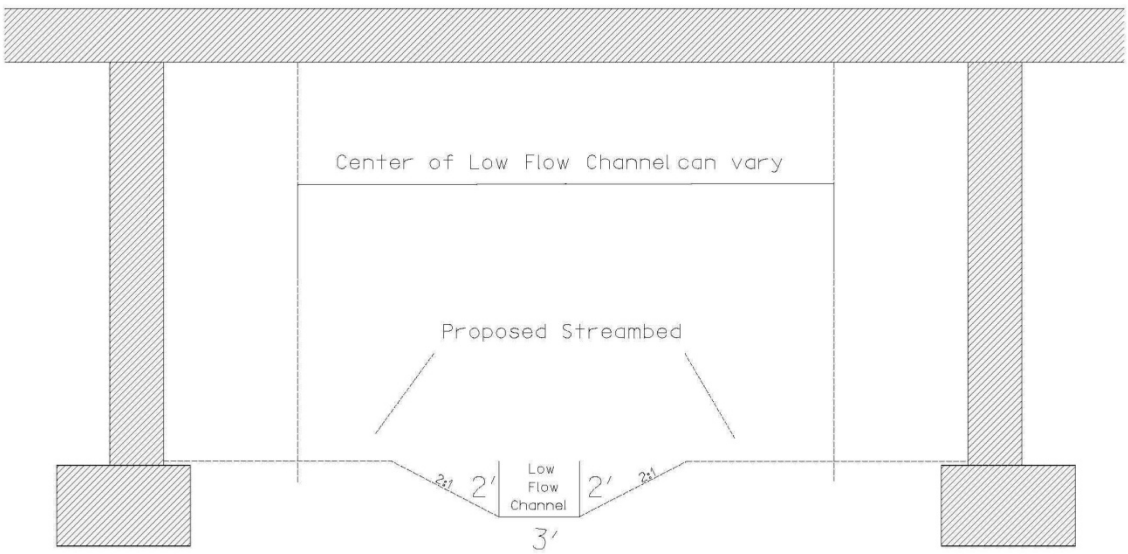

Figure 1.

Installation. Placement of the Precast Block Mats shall start at the downstream end of the channel and proceed upstream. It may be necessary to weight down the geotextile outside the limits of the Precast Block Mat to be placed, prior to installation of the Precast Block Mat.

When the geotextile is secured to the bottom of the Precast Block Mat, an overlap of at least 2 feet shall be incorporated on three sides of the Precast Block Mat. The overlap shall provide an area for the adjoining Precast Block Mats to be placed upon and prevent undermining of the erosion control system.

Rips or damage in the geotextile material shall be repaired in accordance with the manufacturer's recommendations. No individual block within the plane of placed articulating concrete block systems shall protrude more than two inch. The Contractor shall ensure that the concrete blocks are flush and develop intimate contact with the subbase.

If assembled and placed as large mattresses, Precast Block Mats shall be attached to a spreader bar or other approved device to aid in the lifting and placing of the Precast Block Mats in their proper position by the use of a crane or other approved equipment. The equipment used should have adequate capacity to place the Precast Block Mats without bumping, dragging, tearing or otherwise damaging the underlying geotextile. The Precast Block Mats shall be placed side-by-side and/or end-to-end, so that the Precast Block Mats adjoin each other. The gaps between each Precast Block Mat and seams between Precast Block Mats shall not be greater than 2 inches, both below and above water. Grouting will only be permitted where the Precast Block Mats are sealed along structures or at any locations where the stainless steel cables have been cut.

Individual concrete blocks can be cut or trimmed to allow for a tight fit along structures, large obstructions or as required to accommodate the Precast Block Mat layout. The Contractor shall make every effort practicable to minimize the number of individual blocks cut. Avoid cutting the stainless steel cables if at all possible. Any cut blocks shall be secured with sections of stainless steel cable and clamps to the adjacent blocks. These supplementary cables shall be used between every block that has been cut and the adjacent uncut blocks. The supplementary cable shall be made as tight as possible. In addition to the supplementary cables, the area in the vicinity of any cut blocks shall be grouted. The grout shall extend out at least 1 ½ blocks in all directions from the location of the cut block. The method of cutting the blocks shall be approved by the Resident. No overlapping of the Precast Block Mats is allowed.

Installation of the Precast Block Mats shall be done during low-flow stream conditions and during the in-stream work window.

Anchor trenches and flanking trenches along upstream and downstream terminations shall be backfilled and compacted flush with the top of the blocks. The integrity of the trench backfill must be maintained so as to ensure a surface that is flush with the top surface of the concrete blocks for its entire service life. Backfilling and compaction of trenches shall be completed in a timely fashion.

Any excess stainless steel cables that protrude above the top of the Precast Block Mats shall either be tucked underneath the Precast Block Mats, or secured with nylon cable ties (i.e. zip ties) so that the stainless steel cable is below the top of the block.

Once all clamps and anchors have been installed, inspected and accepted, the gaps in the articulating Precast Block Mat system shall be partially backfilled from the geotextile material up to the flush surface of the concrete block. For Precast Block Mats within the stream bed, the Precast Block Mats shall be backfilled with replaced streambed material or a suitable alternative approved by the Resident.

<u>Precast Block Mat – Concrete Structure Interface.</u> The interface between the Precast Block Mats and the existing structure, such as an abutment, pier, wingwall, or retaining wall, shall be tightly sealed to prevent the loss of streambed material. The maximum gap between the Precast Block Mat and the abutment, pier, wingwall, or retaining wall shall be two inches. The methods listed below are acceptable methods to accomplish this. The Contractor may propose other methods, but must receive approval in writing from the Resident to proceed.

1. Grout Placement. The interface between the Precast Block Mats and the existing structure shall be sealed using 3000 psi minimum concrete or grout. The concrete or grout shall be minimally as thick as the Precast Block Mat and shall completely encapsulate at least two (2) rows of concrete blocks. The grout shall be sloped to drain away from the structure. The entire joint between the Precast Block Mat and structure shall be closed at the face of the structure.

2. Grout Filled Bags. Grout filled bags shall be a minimum of one (1) foot thick, three (3) feet wide, and six (6) feet long and placed directly over the interface of the structure and Precast Block Mat so that the completed position of the grout-filled bag is resting atop the Precast Block Mat and against the structure. The bag shall be made of material meeting the properties of a Class 1 erosion control geotextile and shall be equipped with a self-sealing fill valve. If the bag is longer than twenty (20) feet, a second self-sealing fill valve shall be installed. Grout bags shall be butted against each other to form a continuous row along the entire interface. The grout bags shall be filled using 3000 psi minimum concrete or grout as recommended by the manufacturer.

Test Standards and Specifications.

ASTM C31	Practice for Making and Curing Concrete Test Specimens in the Field
ASTM C33	Specifications for Concrete Aggregates
ASTM C39	Compressive Strength of Cylindrical Concrete Specimens
ASTM C42	Obtaining & Testing Drilled Cores and Sawed Beams of Concrete
ASTM C140	Sampling and Test Concrete Masonry Units
ASTM C150	Specification for Portland Cement
ASTM C207	Specification for Hydrated Lime Types
ASTM C618	Specifications for Fly Ash and Raw or Calcined Natural Pozzolans for use in Portland Cement Concrete.
ASTM D18.25.04	Specifications for Articulated Concrete Clock Systems (In Design)
ASTM D698	Laboratory Compaction Characteristics of Soil Using Standard Effort
ASTM D3786	Hydraulic Burst Strength of Knitted Goods and Non-woven Fabrics
ASTM D4355	Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water
ASTM D4491	Water permeability of Geotextiles by Permittivity
ASTM D4533	Trapezoidal Tearing Strength of Geotextiles
ASTM D4632	Breaking Load and Elongation of Geotextiles (grab Method)
ASTM D4751	Determining Apparent Opening Size of a Geotextile
ASTM D4833	Index Puncture Resistance of Geotextiles, Geomembranes and Related Products
ASTM D5101	Measuring the Soil-Geotextile System Clogging Potential by the Gradient Ratio
ASTM D5567	Hydraulic Conductivity Ratio Testing of Soil/Geotextile Systems
ASTM D6684-04	Standard Specification for Materials and Manufacture Articulating Concrete Block (ACB) Revetment Systems
AASHTO T88	Determining the Grain–size Distribution of Soil
AASHTO M288-96	Standard Specification for Geotextiles

FHWA-RD-89-199	November 1989 Standard Testing for Hydraulic Stability of Concrete Revetment System During Overtopping Flow
FHWA-RD-88-181	Minimizing Embankment Damage During Overtopping Flow (Replace by FHWA-RD-89-199 in November 1989)

Quality Control. Units shall be sampled and tested in accordance with ASTM D 6684-04, Standard Specification for Materials and Manufacture of Articulating Concrete Block (ACB) Revetment Systems.

All units shall be sound and free of defects that would interfere with either the proper placement of the unit or impair the performance of the system. Surface cracks incidental to the usual method of manufacture, or surface chipping resulting from the customary methods of handling in shipment and delivery, shall not be deemed grounds for rejection. Chipping resulting in a weight loss exceeding 10 percent of the average weight of a concrete unit shall be deemed grounds for rejection. Blocks rejected prior to delivery from the point of manufacture or at the jobsite shall be repaired with structural grout or replaced at the expense of the Contractor. The Department or their authorized representative shall be accorded proper access to facilities to inspect and sample the units at the place of manufacture from lots ready for delivery.

Field installation procedures shall comply with the procedures utilized during the hydraulic testing procedures of the recommended system. All system restraints and ancillary components shall be employed as they were during testing. For example, if the hydraulic testing installations utilize a drainage layer, then the field installation must utilize a drainage layer; and installation without the drainage layer would not be permitted.

The theoretical force-balance equation used for performance extrapolation tends for conservative performance values of thicker concrete units based on actual hydraulic testing of thinner units. When establishing performance values of thinner units based on actual hydraulic testing of thicker units, there is a tendency to overestimate the hydraulic performance values of the thinner units. Therefore, all performance extrapolation must be based on actual hydraulic testing of a thinner unit then relating the values to the thicker units in the same family of blocks.

Additional testing, if required, for alternate designs shall be the responsibility of the Contractor.

Hydraulic Testing, Calculations and Submittals. The Contractor shall submit to the Resident all manufacturer's hydraulic testing and calculations in support of the proposed articulated concrete block system and geotextile filter fabric. All calculations submitted must be consistent with the hydraulic details found in the section and stamped by a Professional Engineer (PE) licensed in the State of Maine.

The Contractor shall furnish the manufacturer's Certificates of Compliance for Precast Block Mat, revetment cable, and any revetment cable fittings and connectors as specified in this Special Provision. The Contractor shall also furnish the manufacturer's specifications, literature, shop drawings for the layout of Precast Block Mats, and any recommendations, if applicable, that are specifically related to the Project. The Contractor shall also submit the proposed method for anchoring the Precast Block Mat, both to the embankments and the streambed/abutments.

Alternative materials may be considered. Such materials must be approved in writing by the Resident. Submittal packages must include, as a minimum, the following:

1. Full-scale laboratory testing and associated engineered calculations quantifying the hydraulic capacity of the proposed Precast Block Mat system in similar conditions to the specific project. Submitted calculations must be PE stamped by a duly licensed Engineer licensed in the State of Maine.

2. A list of five comparable projects, in terms of size and applications, in the United States, where the results of the specific alternate revetment system used can be verified after a minimum of five (5) years of service life.

Method of Measurement. The Precast Block Mat will be measured for payment by the area of articulating Precast Block Mat system in square feet, accepted and in place.

Basis of Payment. The accepted quantity of Precast Block Mat shall be paid for at the contract unit price. Such payment being full compensation for all labor, materials, equipment, Quality Control, submittals, testing and incidentals necessary to complete the work as specified including, but not limited to, ground preparation, Precast Block Mats, geotextile, anchors, clamps, grouting, grout bags and backfill.

Payment will be made under:

Pay Item	Pay Unit
502.83 Precast Block Mat	Square Foot