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Report on Subsurface and Foundation Investigation

Replacement Bridge for Eliot Road Overpass Carrying Route 1 Bypass Over Route 103 (Eliot Road) Kittery, Maine

for

Maine Department of Transportation
State House Station 16
Augusta, Maine 04333-0016

December 11, 2008

MaineDOT PIN 15099.00
Soils Report No. 2008-20C
TEDOCS No. 933197

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December 11, 2008
08335

Laura Krusinski, P.E.
Senior Geotechnical Engineer
Maine Department of Transportation
State House Station 16
Augusta, ME 04333-0016

Report on Subsurface and Foundation Investigation, Replacement Bridge for Eliot Road Overpass Carrying Route 1 Bypass over Route 103 (Eliot Road), Kittery, Maine

Dear Laura:

We are pleased to submit herewith 6 copies of our report which summarizes the results of our subsurface and foundation investigation for the subject project. These services were undertaken in accordance with our Project Contract under GCA dated August 27, 2008.

In summary, we recommend that the replacement bridge be supported on reinforced concrete abutments founded on new spread footings bearing on sound, intact bedrock.

We appreciate the opportunity to provide geotechnical engineering services on this project. If you have questions or need more information, please contact us.

Sincerely,

SEBAGO TECHNICS, INC.

Kenneth L. Recker, P.E.
Geotechnical Engineering Manager

STATE OF MAINE
KENNETH L. RECKER
No. 5435
LICENSED
PROFESSIONAL ENGINEER

KLR:klr/dlf

Table of Contents

GEOTECHNICAL DESIGN SUMMARY

This report presents the results of our subsurface and foundation investigation for the replacement bridge for U.S. Route 1 Bypass over Eliot Road in Kittery, Maine. The replacement bridge will be a single span structure with span length of approximately 65 feet and a width of approximately 65 feet. The bridge will be supported on cantilever-type abutments bearing on spread footings founded on sound, intact bedrock.

Cantilever Abutments and Wingwalls – Abutments and wingwalls should be designed for all applicable strength and service limit states in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition, 2007. They should be designed to resist all lateral earth loads, superstructure loads, vehicular loads, and any loads transferred through the superstructure.

Spread footings supporting abutments and wing walls should be designed at the strength limit state considering nominal bearing resistance, sliding, overturning and structural failure. A bearing resistance factor, φ_b, of 0.45 should be applied to the nominal bearing resistance and a sliding resistance factor, φ_τ, of 0.80 should be applied to the nominal sliding resistance of spread footings bearing on bedrock. The eccentricity of loading at the strength limit state, based on factored loads, should not exceed 3/8 of the footing dimensions, in either direction.

Abutments and wingwalls that are free to rotate should be designed in accordance with the lateral earth pressure given in Section 3.6.1 of the MDOT Bridge Design Guide (BDG). The parameters for soil type No. 4 in Table 3.3 (φ = 32 deg., γ = 125 pounds per cubic foot, tan δ = 0.45) should be used. Lateral earth pressure resulting from construction or operating surcharge loads should be accounted for in accordance with Section 3.6.8 of the BDG for abutments and wingwalls if no approach slab is used.

Bearing Capacity – Footings should bear on sound, intact bedrock. Footings may be proportioned for a factored bearing stress at the service limit state of 20 kips per square foot (ksf) which incorporates a resistance factor of 1.0. Footings may be proportioned for a factored bearing stress at the strength limit state of 20 ksf which incorporates a resistance factor of 0.45. All footings should be a minimum of 2 feet wide.

Settlement – The grades immediately behind the abutments will be raised up to 8 feet. The soil overlying bedrock consists of medium dense to dense silty SAND; to stiff SILT (fill and glacial till). Therefore, post construction settlement due to compression of the soil is anticipated to be less than 0.75 inch and will occur immediately following loading. It is not expected to have unacceptable impact on the completed structure. Compression of the bedrock supporting the abutments and wingwalls will be nominal and will occur immediately following application of dead loads and is expected to be negligible.

Frost Protection – Sound, intact bedrock is not subject to movement when exposed to freezing temperatures and therefore there are not frost embedment requirements for foundations bearing on sound, intact bedrock.

Seismic Design Considerations – The Route 1 Bypass Bridge over Eliot Road is not on the National Highway System (NHS) and is not considered to be functionally important. Based on LRFD Figure 3.10.2-2, the horizontal bedrock acceleration coefficient, A, for the site is approximately 0.13g and per LRFD 3.10.4, Table 3.10.4-1, the bridge is assigned to Seismic Performance Zone 2. However, in accordance with LRFD 4.7.4.2, seismic analysis is not required for single-span bridges regardless of seismic zone, but superstructure connections and bridge seat dimensions should be in accordance with LRFD 3.10.9.3, 4.7.4.1 and 4.7.4.3.

Construction Considerations – It is anticipated that excavation for foundation construction can be accomplished with open excavation provided safe side slopes can be maintained. However, for convenience or construction staging, it may be appropriate to install steel sheet piles for excavation support. Preparation of bearing surfaces may require excavation and stepping of rock surface. It is anticipated that rock excavation will require drilling and blasting or mechanical means for removal. All footings should bear on sound, intact bedrock cleaned of loose rock fragments and soil using high pressure air or water.

1. INTRODUCTION

This report presents the results of our subsurface and foundation investigation for the replacement bridge for Route 1 Bypass over Eliot Road in Kittery, Maine.

The existing bridge is a three span structure with steel girders and concrete deck with bituminous overlay. The superstructure is supported on cast-in-place concrete abutments and piers. The support method for the abutments and piers is not known. The structure was constructed in approximately 1946.

The replacement bridge will be a 65 feet long single span structure on the same alignment as the existing bridge. The bridge width is approximately 65 feet wide and will include 2 travel lanes in each direction, a center median, and shoulders. We anticipate that the vertical profile of the bridge and approaches will match the existing. The abutments will be full-height, cantilever type, founded on spread footings bearing on sound, intact bedrock. The wingwalls will be cantilever type with butterfly ends. The existing bridge and substructures will be entirely removed.

2. GEOLOGIC SETTING

The Route 1 Bypass Bridge crosses over Eliot Road as shown on Figure 1, Location Map. Bedrock is exposed on the east and west sides of the north bridge abutment and on the east side of the south bridge abutment.

Based on the Maine Geological Survey Surficial Geology (MGS) Map of Portsmouth, Open-File No. 99-96, 1999, the area is mapped as marine nearshore deposits consisting of thin discontinuous till and water-laid sediments overlying bedrock. Glacial sediments were eroded and redeposited during regressive phase of late-glacial marine submergence. Bedrock exposures are locally numerous.

The MGS Open-File No. 08-78, 2008, "Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine and New Hampshire" classifies the bedrock at the site as variably thin to thick bedded, buff-weathering feldspathic and calcareous metawacke of the Kittery Formation.

3. SUBSURFACE EXPLORATIONS

During the period September 16 to 18, 2008, Maine Test Borings, Inc. (MTB) of Brewer, Maine drilled 5 borings, B1 to B5 at the site at locations shown on Figure 2, Boring Location Plan. MTB drilled the borings to depths below bridge deck varying from 18.8 feet to 29.7 feet. Sebago Technics, Inc. monitored the borings and prepared the logs included in Appendix A. Table I summarizes the results of borings. MTB backfilled the borings with the drilled material and sand and placed a bituminous cold patch at the drilled hole in the bridge deck.

Borings were drilled using 3.0-inch inside diameter flush-joint steel casing using wash boring techniques. Soil samples were generally recovered at 5-feet intervals. Standard penetration resistance, N, was measured at each sample interval. Borings were advanced to refusal and a NQ2 core of bedrock was recovered in each boring and the Rock Quality Designation (RQD) of each core run was calculated. Due to the generally inaccessible conditions below and adjacent to the bridge, borings were drilled through the existing bridge deck and casing was lowered to the ground surface.

Locations of borings were determined by Sebago Technics, Inc. by taping from existing features on the bridge. The elevation of the deck at boring locations was estimated by linear interpolation between elevation contours at the plotted locations.

The boring logs and related information depict the subsurface conditions and water levels encountered at the locations and during the times indicated on the logs. Subsurface conditions at other locations may differ from those encountered in the explorations. The passage of time may result in a change in groundwater conditions at the exploration locations.

4. SUBSURFACE CONDITIONS

The borings disclosed three principal soil units overlying weathered bedrock and bedrock at the site: fill, sand and silt. Encountered thickness and generalized descriptions of the strata are presented below in order of increasing depth below ground surface. Due to the complexity of the deposition process, strata thickness will vary and may be absent at specific locations.

Fill – Fill consists of loose to very dense, brown silty coarse to fine SAND (SM), some gravel; to silty fine SAND (SM), trace coarse to medium sand and gravel. Encountered thickness varies from 2.1 feet to 8.0 feet.

Sand - Sand consists of dense, brown coarse to fine SAND (SM), some gravel and weathered rock fragments deposited as glacial till and possibly reworked. Boring B2 encountered 3.3 feet of sand.

Silt – Silt consists of stiff, olive-brown SILT (ML), some clay and coarse to fine sand and gravel deposited as glacial till. The silt may contain cobbles and boulders that could not be sampled in the split-spoon sampler. Boring B2 encountered 5.6 feet of silt.

Weathered bedrock consisting of sand and gravel sized pieces of rock fragments were encountered in all borings overlying sound bedrock. Encountered thickness varies from 0.1 foot to 0.8 foot.

Bedrock was recovered in all borings. Lengths of coring vary from 4.7 feet to 10.5 feet. Elevations of the top of sound bedrock are included in Table I. Bedrock consists of hard, slightly to moderately weathered, dark gray aphanitic METAPELITE; to dark gray coarse to fine grained GABBRO; to dark gray aphanitic to fine-grained BASALT. The primary joint set is very close to close, horizontal to moderate, partly open to moderately wide, planar to undulating and rough. In general, high angle secondary joint sets are present throughout.

The RQD of the bedrock cores varies from 0 to 90 with the average being 48. Rock cores with 0 RQD had core lengths of 1.0 foot and are not typical of the remaining cores. If these cores are excluded, RQD varies from 17 to 90 with the average RQD equal to 57 indicating poor to excellent quality with an average fair rock quality.

5. GEOTECHNICAL ENGINEERING DESIGN RECOMMENDATIONS

The following sections present the geotechnical design recommendations for support of the abutments and wingwalls on spread footings bearing on bedrock.

5.1 Support

The existing fill and sand and silt are not considered suitable for support of the abutments and wingwalls. In the vicinity of the abutments and wingwalls, the top of sound bedrock varies from approximately 2.9 to 11.4 feet below ground surface. The abutments and wingwalls should be supported on sound, intact bedrock cleaned of all loose rock and soil. The maximum slope of the bedrock surface should not be steeper than 4 horizontal to 1 vertical. Steeper slopes should be benched or tapered to the above criteria.

5.2 Abutment and Wingwall Design

Abutments and wingwalls should be designed for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and for all appropriate strength and service limit states. Abutment and wingwall design should consider bearing resistance, overturning, lateral sliding and structural failure.

Consistent with LRFD Table 10.5.5.2.2-1, a sliding resistance factor, φ_τ, equal to 0.80 should be applied to the sliding resistance of abutments and wingwalls supported by spread footings bearing on bedrock. A coefficient of friction equal to 0.70 may be used for footings bearing on sound bedrock. The eccentricity of loading at the strength limit state, based on factored loads, should not exceed 3/8 of the footing dimensions, in either direction.

A resistance factor for the service limit state should be taken as 1.0 for settlement, horizontal movement and overall stability.

Abutments and wingwalls that are free to rotate should be designed in accordance with the lateral earth pressure given in Section 3.6.1 of the MDOT Bridge Design Guide (BDG). The parameters for soil type No. 4 in Table 3.3 (φ = 32 deg., γ = 125 pounds per cubic foot, tan δ = 0.45) should be used. Earth loads should be calculated using an active earth pressure coefficient, K_a, calculated using Rankine Theory for cantilever type abutments and wingwalls. For horizontal backfill, $K_a = \tan^2 \{45^\circ - \varphi/2\}$.

To minimize abrupt differential movement between the abutment and road surface, we recommend that an approach slab, supported on the abutment, be included.

Drainage behind abutments and wingwalls should be designed in accordance with provisions of BDG, Section 5.4.1.4. Backfill within 10 feet of abutments and wingwalls should consist of free-draining soil meeting the requirements of Granular Borrow for Underwater Backfill, MDOT Specification 7.09.19.

5.3 Bearing Stress

Spread footings should be proportioned for stability against bearing failure. LRFD Article 11.5.5 provides specification for application of permanent and transient loads. Applied stresses may be assumed to be linearly distributed over the effective width of the footing as shown in FRFD Figure 11.6.3.2-2. The bearing resistance for footings founded on sound, intact bedrock shall be evaluated at the strength limit state using factored loads and factored bearing resistance of 20 ksf. This is based on a bearing resistance factor, φ_b, for spread footings on rock of 0.45. Bearing surfaces should be cleaned of all loose, fractured bedrock and overlying soil. A factored bearing resistance of 20 ksf may be used for analysis of the service limit state and for proportioning footing sizes, using a resistance factor of 1.0. Calculations of factored bearing resistance are included in Appendix B. All footings should be at least 2 feet wide.

5.4 Settlement

The grades immediately behind the abutments will be raised up to 8 feet. The soil overlying bedrock consists of medium dense to dense silty SAND; to stiff SILT (fill and glacial till). Therefore, post construction settlement due to compression of the soil is anticipated to be less than 0.75 inch and will occur immediately following loading. It is not expected to have unacceptable impact on the completed structure. Compression of the bedrock supporting the abutments and wingwalls will be nominal and will occur immediately following application of dead loads and is expected to be negligible.

5.5 Frost Protection

Sound, intact bedrock is not subject to movement when exposed to freezing temperatures and therefore there are not frost embedment requirements for foundations bearing on sound, intact bedrock.

5.6 Seismic Design Considerations

The Route 1 Bypass Bridge over Eliot Road is not on the National Highway System (NHS) and is not considered to be functionally important. Based on LRFD Figure 3.10.2-2, the horizontal bedrock acceleration coefficient, A, for the site is approximately 0.13g and per LRFD 3.10.4, Table 3.10.4-1, the bridge is assigned to Seismic Performance Zone 2. However, in accordance with LRFD 4.7.4.2, seismic analysis is not required for single-span bridges regardless of seismic zone, but superstructure connections and bridge seat dimensions should be in accordance with LRFD 3.10.9.3, 4.7.4.1 and 4.7.4.3.

6.0 CONSTRUCTION CONSIDERATIONS

It is anticipated that excavation for foundation construction can be accomplished with open excavation provided safe side slopes can be maintained. However, for convenience or construction staging, it may be appropriate to install steel sheet piles for excavation support. Temporary excavations and lateral support systems should be designed and constructed in accordance with all OSHA and other applicable regulatory agency requirements.

Preparation of bearing surfaces may require excavation and stepping of rock surface. It is anticipated that rock excavation will require drilling and blasting or mechanical means for removal. The contractor should conduct all blasting activity in accordance with Section 105.2.6 of the MDOT Standard Specifications. In addition, the contractor should conduct all blasting activity in such a manner that the peak particle velocity of ground vibration measured at the location of the nearest structures to the blast does not exceed the "safe limits" recommended by the U.S. Bureau of Mines as presented in Figure B-1 in Appendix B of BUMINES RI 8507 and the peak airblast overpressure measured at the location of the nearest above ground occupied structures to the blast (considering wind direction) does not exceed 0.014 pounds per square inch. Blasting mats should be utilized for all blast rounds detonated to prevent the throw of flyrock from the blasting area.

All footings should bear on sound, intact bedrock cleaned of loose rock fragments and soil using high pressure air or water. The maximum slope of the bedrock surface should not be steeper than 4 horizontal to 1 vertical. Steeper slopes should be benched or tapered to the above criteria. The contractor may also use dowels or other means to resist sliding where the bedrock slope is steeper than 4:1. The final bedrock surface should be approved by the Resident prior to placing concrete for footings.

Groundwater may be encountered at bearing level of footings. If encountered, open pumping from sumps can likely control groundwater. In general, the contractor should control groundwater and water from runoff and other sources by methods which prevent disturbance to bearing surfaces or adjacent soils and allow construction in-the-dry.

7.0 CLOSURE

This report has been prepared for specific application to the proposed replacement of the Route 1 Bypass Bridge over Eliot Road in Kittery, Maine in accordance with generally accepted geotechnical engineering practices. In the event that any changes in the nature, design or location of the replacement bridge are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

The recommendations presented herein are based in part on the data obtained from the referenced test borings. The nature and extent of variations between the explorations may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

We request that we be provided the opportunity for a general review of final design and specifications in order to determine that our earthwork and foundation recommendations have been interpreted and implemented in the design and specifications as they were intended.

TABLE

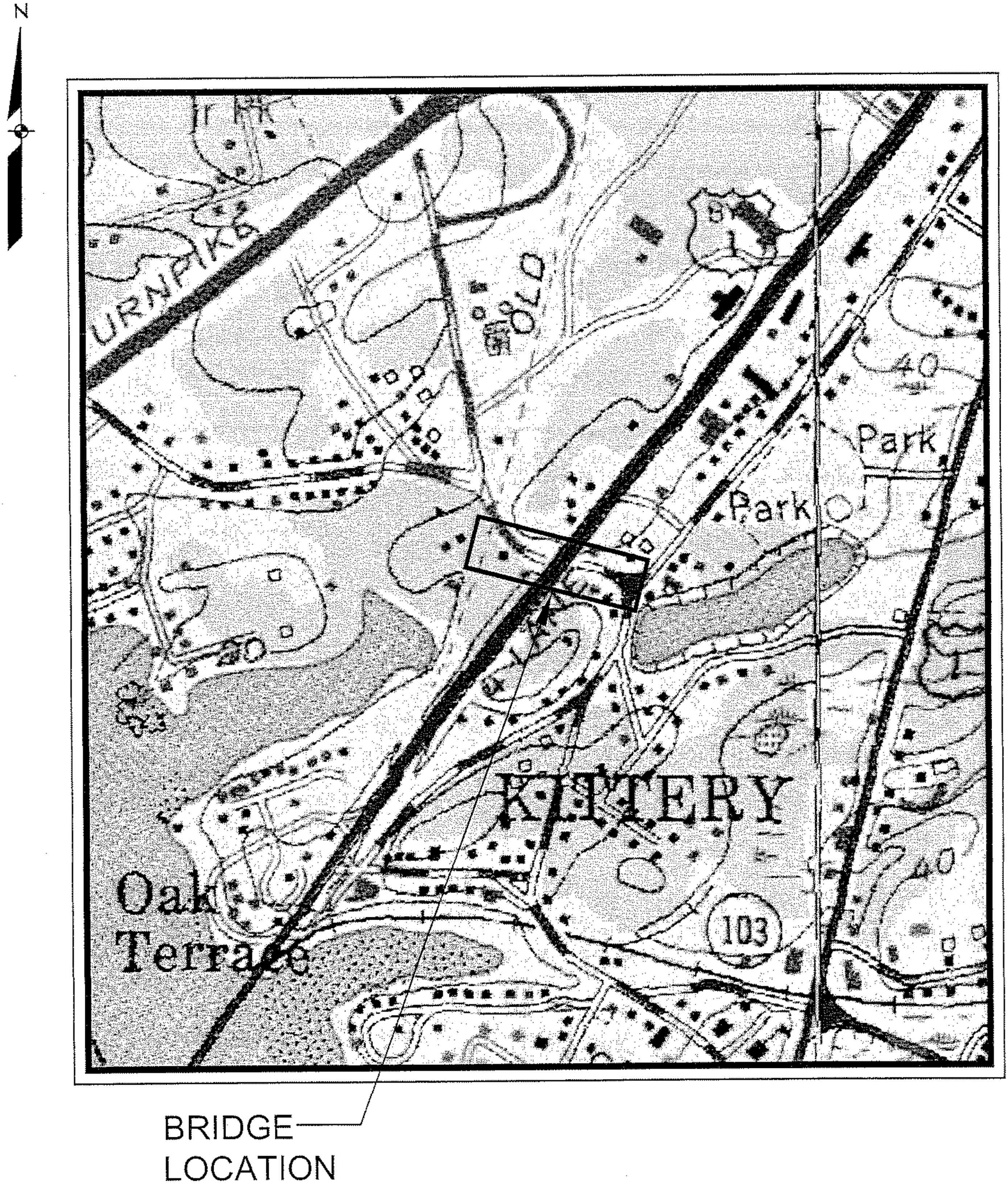
TABLE I
SUMMARY OF BORINGS
REPLACEMENT BRIDGE
ROUTE 1 BYPASS OVER ELIOT ROAD
KITTERY, MAINE

Boring Number	Depth Below Bridge Deck (Ft)	Deck Surface El. (Ft)	Depth to Water (Ft)	Strata Thickness (Ft)								El. Top of Bedrock (Ft)
				Bituminous	Concrete	Void	Fill	Sand	Silt	Weathered Bedrock	Bedrock	
B1	25.8	44.4	21.2	0.8	0.7	12.8	3.7	--	--	0.3	7.5	26.1
B2	29.7	44.2	19.7	0.8	0.7	10.5	2.2	3.3	5.6	0.3	6.3	20.8
B3	20.3	44.3	16.8	0.8	0.7	11.2	2.1	--	--	0.8	4.7	28.7
B4	26.5	44.3	19.7	0.8	0.7	6.5	8.0	--	--	0.1	10.5	28.3
B5	18.8	44.3	19.7	0.8	0.7	5.3	4.0	--	--	0.4	7.6	33.1

NOTES:

1. BORINGS DRILLED FROM EXISTING BRIDGE DECK BY CORING THROUGH DECK WITH THIN-WALL CORE AND LOWERING CASING TO GROUND SURFACE.
2. ELEVATION OF DECK AT BORING LOCATIONS ESTIMATED BY LINEAR INTERPOLATION BETWEEN ELEVATION CONTOURS AT THE PLOTTED LOCATIONS.
3. -- INDICATES THAT STRATUM NOT ENCOUNTERED WITHIN DEPTH OF BORING.
4. * INDICATES DEPTH OF PENETRATION INTO STRATUM.

FIGURES



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LOCATION MAP

OF: ROUTE ONE BYPASS/ROUTE 103 BRIDGE
KITTERY, MAINE

SCALE:	1"=600'
DATE:	12-2-08
SHEET:	FIGURE 1

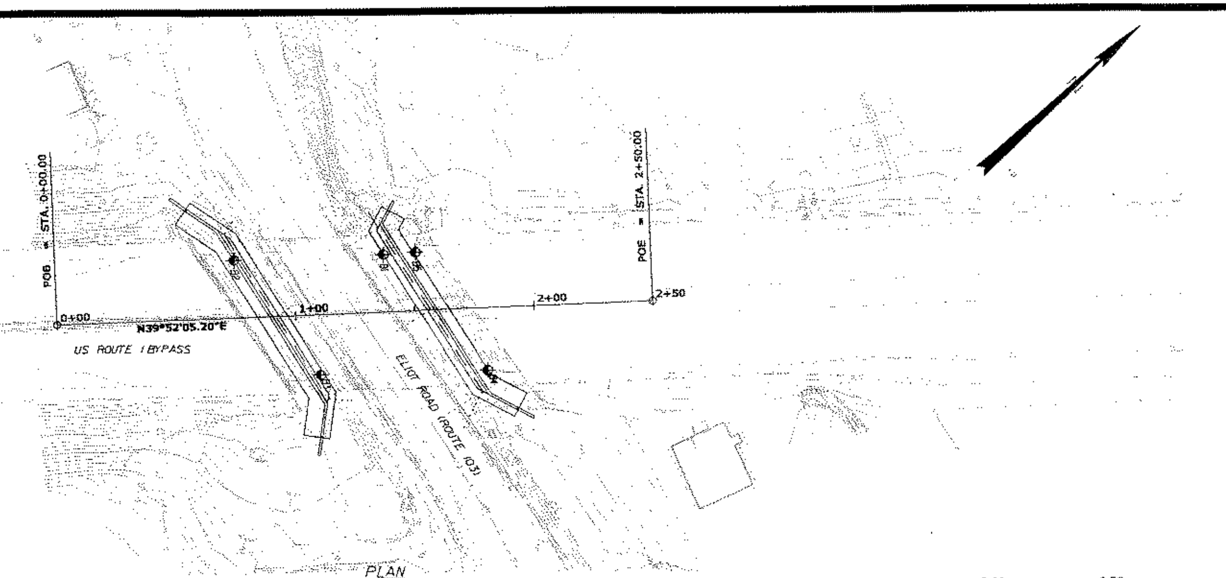
08335LOC.dwg, TAB: LOC

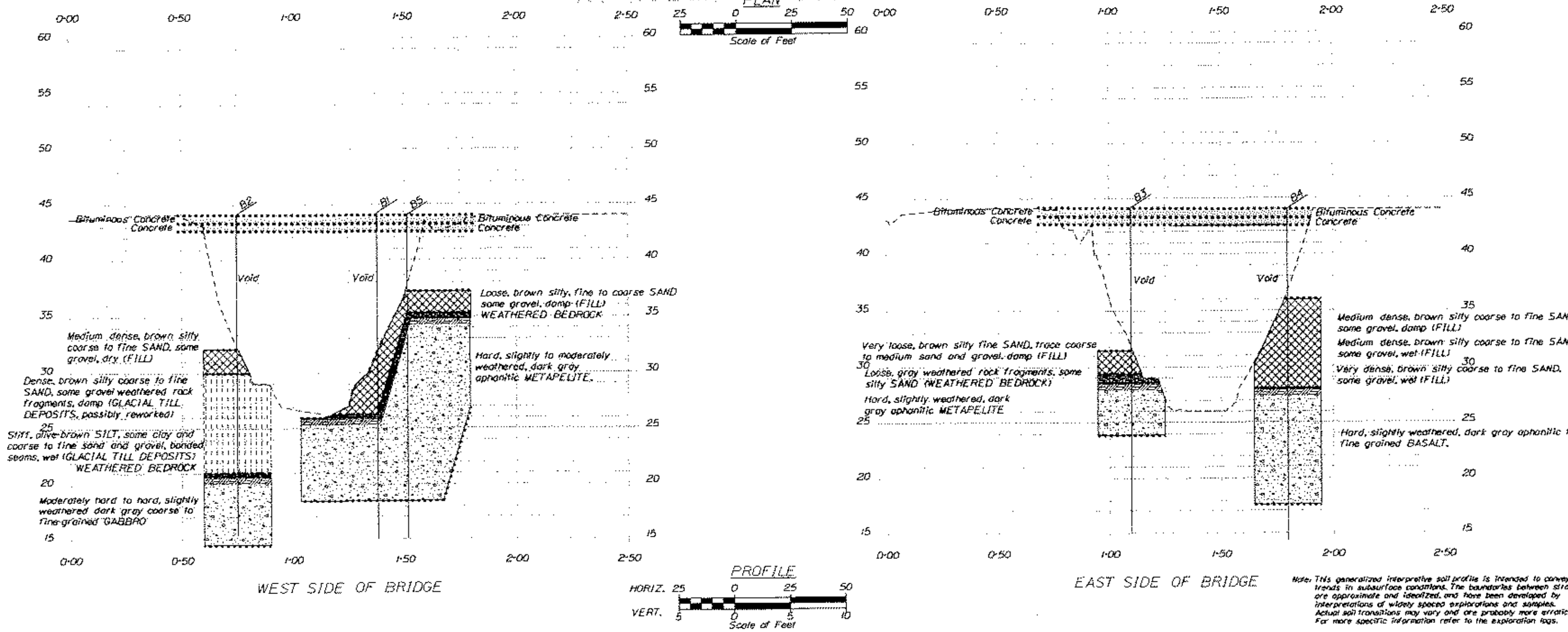
LEGEND

B1 — NUMBER AND LOCATION OF BORINGS DRILLED BY MAINE TEST BORINGS, INC. DURING SEPTEMBER 16 TO 18, 2008.

NOTES

1. BASE PLAN PREPARED FROM ELECTRONIC FILE PROVIDED BY MDOT.
2. BORINGS MONITORED BY SEBAGO TECHNICS, INC.
3. LOCATIONS OF BORINGS DETERMINED BY SEBAGO TECHNICS, INC. BY TAPING FROM EXISTING SITE FEATURES. ELEVATION OF BRIDGE DECK AT BORING LOCATIONS ESTIMATED BY LINEAR INTERPOLATION BETWEEN EXISTING ELEVATION CONTOURS AT PLOTTED LOCATIONS.
4. BORINGS WERE DRILLED THROUGH THE EXISTING BRIDGE DECK AND CASING WAS LOWERED TO THE GROUND SURFACE.





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

PIN 16099.00 — HIGHWAY PLANS

	BY	DATE	
PROJ. MANAGER			SIGNATURE
DESIGN-DETAILED	B. LYON	12/5/08	
CHECKED-REVIEWED	K. BECKER	12/5/08	
DESIGN2-DETAILED2			P.E. NUMBER
DESIGN3-DETAILED3			
REVISIONS 1			DATE
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

KITTERY
ROUTE 1 BYPASS

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

FIGURE
2

Filename: ...\00\BRIDGE\MSTA\001_HDPlan1.dgn   Division: HIGHWAY   Username: blyon   Date:12/11/2008

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project: Elliot Road Overpass over Route 103

Location: Kittery, Maine

Boring No.: B1

PIN: 15099.00

Driller:	Maine Test Borings, Inc.	Elevation (ft.)	44.4	Auger ID/OD:	
Operator:	D. McKeen	Datum:		Sampler:	24" Standard Split Spoon
Logged By:	K. B. Stephenson	Rig Type:	Mobile B53 Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/16/08	Drilling Method:	Cased/Drive/Wash	Core Barrel:	NQ2
Boring Location:	See Plan	Casing ID/OD:	NW	Water Level*:	21.15' below top pavement

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							43.7		BITUMINOUS CONCRETE 0.8	
							42.9		CONCRETE 1.5	
									VOID	
5										
10							30.1		14.3	
	1D	24/8	14.3 - 16.3	2/6/4/4	10	3			Loose, brown silty fine to coarse SAND, some gravel, damp (FILL)	
15						29				
						40				
						104	26.4		18.0	
	R1	22.8/23	18.3 - 20.2	RQD = 26%		39/.2	26.1		WEATHERED BEDROCK Advanced roller cone to 18.3 ft. 18.3	
						~~NQ~~ CORE			R1: Hard, slightly to moderately weathered, dark gray aphanitic METAPELITE. Two low angle joints from 18.3 to 18.9 ft. Remainder of core is highly fractured, rusty, with clayey silt infill from 19.8 to 20.2 ft.	
20	R2	64.8/64	20.4 - 25.8	RQD = 17%					Drove NW casing to 20.1 ft. Advanced roller cone to 20.4 ft. R2: Hard, slightly to moderately weathered, dark gray aphanitic METAPELITE. Primary joint set is extremely close to close, horizontal to moderate, partly open to moderately wide, planar to undulating and rough. High angle secondary joint set present. Core is moderately fractured. Rusty discolorations on joint surfaces. Silt infilling from 20.4 to 21.4 ft.	
25	R3	39.6/40	25.8 - 29.1	RQD = 65			18.6		R3: Same lithology as R1 and R2. Primary joint set same as R2 except extremely close to moderate. Rusty discolorations on joint surfaces. 25.8	
									Bottom of Exploration at 25.8 feet below ground surface.	
30										

Remarks:

Rope/cathead, safety hammer. 6 in. thinwall core of pavement

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.: B1

Maine Department of Transportation Soil/Rock Exploration Log US UNITS	Project: Elliot Road Overpass over Route 103 Location: Kittery, Maine	Boring No.: B2 PIN: 15099.00
Driller: Maine Test Borings, Inc.	Elevation (ft.): 44.2	Auger ID/OD:
Operator: D. McKeen	Datum:	Sampler: 24" Standard Split Spoon
Logged By: K. B. Stephenson	Rig Type: B53 Mobile Truck	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 09/16/08-09/17/08	Drilling Method: Cased/Drive/Wash	Core Barrel: NQ2
Boring Location: See Plan	Casing ID/OD: NW	Water Level*: 19.7' below top of pavement

Definitions:	Definitions:	Definitions:
D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample R = Rock Core Sample V = Insitu Vane Shear Test SSA = Solid Stem Auger	S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) $S_{u(lab)}$ = Lab Vane Shear Strength (psf) WOH = weight of 140lb. hammer WOR = weight of rods	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-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							43.5	BITUMINOUS CONCRETE —0.8	
							42.7	CONCRETE —1.5	
								VOID	
5.5									
11							32.2	—12.0	
	1D	24/4	12.0 - 14.0	2/4/14/18	18	3		Medium dense, brown silty coarse to fine SAND, some gravel, dry (FILL)	
						18			
						39	30.0	—14.2	
	2D	24/14	15.0 - 17.0	16/20/15/11	35	14		Dense, brown silty coarse to fine SAND, some gravel, weathered rock fragments, damp (GLACIAL TILL DEPOSITS, possibly reworked)	
16.5						22			
						15			
						7			
						6			
	3D	24/18	20.0 - 22.0	4/4/8/9	12	3		Stiff, olive-brown SILT, some clay and coarse to fine sand and gravel, bonded seams, wet (GLACIAL TILL DEPOSITS)	
22						13			
						25	21.1	—23.1	
	R1	24/18	23.4 - 25.4	RQD = 50%		11/.1	20.8	Advanced roller cone to 23.4 ft. (WEATHERED BEDROCK) —23.4	
	R2	51.6/54	25.4 - 29.7	RQD = 90%				R1: Moderately hard to hard, slightly weathered, dark gray coarse to fine-grained GABBRO. Primary joint set is very close to close, horizontal to low, open, planar to undulating and rough. R2: Same lithology as R1. Primary joint set same except extremely close to moderate.	
27.5									
							14.5	—29.7 **Bottom of Exploration at 29.7 feet below ground surface.**	
33									

Remarks:

Rope/cathead, safety hammer. 6 in. thinwall core of pavement

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.: B2

Maine Department of Transportation

Soil/Rock Exploration Log

US UNITS

Project: Elliot Road Overpass over Route 103	Boring No.: B3	
Location: Kittery, Maine	PIN: 15099.00	

Driller:	Maine Test Borings, Inc.	Elevation (ft.)	44.3	Auger ID/OD:	
Operator:	D. McKeen	Datum:		Sampler:	24" Standard Split Spoon
Logged By:	K. B. Stephenson	Rig Type:	B53 Mobile Truck	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/17/08	Drilling Method:	Cased/Drive/Wash	Core Barrel:	NQ2
Boring Location:	See Plan	Casing ID/OD:	NW	Water Level*:	16.8' below top of pavement

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							43.6		BITUMINOUS CONCRETE 0.8	
							42.8		CONCRETE 1.5	
5									VOID	
10										
	1D	24/8	12.7 - 14.7	1/1/1/6	2	WOH	31.6		12.7	
						3			Very loose, brown silty fine SAND, trace coarse to medium sand and gravel, damp (FILL)	
	2D	7/7	14.8 - 15.4	44/50-.1		11	29.5		14.8	
15	1R	56.4/54	15.6 - 20.3	RQD = 55%		50/.6	28.7		Loose, gray weathered rock fragments, some silty SAND (WEATHERED BEDROCK) 15.6	
									Hard, slightly weathered, dark gray aphanitic METAPELITE. Primary joint set is extremely close to moderate, horizontal to moderate, open, planar to undulating and rough. High angle secondary joints at 16.5 and 19.7 ft. Calcite/ quartz stringers present throughou. Rusty discolorations on joints.	
20							24.0		20.3	
									Bottom of Exploration at 20.3 feet below ground surface. Note: piece of rock from core barrel blocks borehole- unable to core beyond 20.3 ft.	
25										
30										

Remarks:

Rope/cathead, safety hammer. 6 in. thinwall core of pavement

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 UNITS

Project:	Elliot Road Overpass over Route 103			**Boring No.:**	B4
Location:	Kittery, Maine			**PIN:**	15099.00
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	44.3	**Auger ID/OD:**	
Operator:	D. McKeen	**Datum:**		**Sampler:**	24" Standard Split Spoon
Logged By:	K. B. Stephenson	**Rig Type:**	B53 Mobile Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/17/08-09/18/08	**Drilling Method:**	Cased/Drive/Wash	**Core Barrel:**	NQ2
Boring Location:	See Plan	**Casing ID/OD:**	NW	**Water Level*:**	19.7' below top of pavement

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							43.6		BITUMINOUS CONCRETE 0.8	
							42.8		CONCRETE 1.5	
5									VOID	
							36.3		8.0	
	1D	24/10	8.0 - 10.0	3/5/7/10	12	5			Medium dense, brown silty coarse to fine SAND, some gravel, damp (FILL)	
						14				
10	2D	24/8	10.5 - 12.5	13/14/8/11	22	10			Medium dense, brown silty coarse to fine SAND, some gravel, wet (FILL)	
						16.0				
						21				
						13				
						16				
15	3D	10/7	15.1 - 15.9	12/50-.3		30/.9	28.3		Very dense, brown silty coarse to fine SAND, some gravel, wet. (FILL) Split spoon refusal at 15.9 ft. Advanced roller cone to 16.0 ft. 16.0	
	R1	22.8/13	16.0 - 17.9	RQD = 57%		NQ ~~CORE~~			R1: Hard, slightly weathered, dark gray aphanitic to fine-grained BASALT. Primary joint set is extremely close to close, horizontal to moderate, open, planar to undulating and rough. Rusty discolorations on joint surfaces.	
	R2	12/12	17.9 - 18.9	RQD = 0%					R2: Same lithology and joint set as R1.	
	R3	28.8/27	18.9 - 21.3	RQD = 76%					R3: Same lithology and joint set as R1, except primary joint set is very close to moderate, horizontal to low angle. At 20.9 ft., contact with hard, slightly weathered, gray-brown fine-grained ANDESITE.	
20										
	R4	61.2/62	21.3 - 26.4	RQD = 85%					R4: Approximately 1/2 in. end of contact with ANDESITE. Hard, slightly weathered, dark gray aphanitic to fine grained BASALT. Fine-grained ANDESITE intrusions at 22.4, 23.9 and 26.3 ft. Host rock inclusions and secondary mineralization from 25.7 to 26.0 ft.	
25										
							17.8		26.5	
									Bottom of Exploration at 26.5 feet below ground surface.	
30										

Remarks:

Rope/cathead, safety hammer. 6 in. thinwall core of pavement

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.: B4

Maine Department of Transportation

Soil/Rock Exploration Log
US UNITS

Project:	Elliot Road Overpass over Route 103	**Boring No.:**	B5		
Location:	Kittery, Maine	**PIN:**	15099.00		
Driller:	Maine Test Borings, Inc.	**Elevation (ft.)**	44.3	**Auger ID/OD:**	
Operator:	D. McKeen	**Datum:**		**Sampler:**	24" Standard Split Spoon
Logged By:	K. B. Stephenson	**Rig Type:**	B53 Mobile Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/18/08	**Drilling Method:**	Cased/Spin/Wash	**Core Barrel:**	NQ2
Boring Location:	See Plan	**Casing ID/OD:**	NW	**Water Level*:**	19.7' below top of pavement

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							43.6		BITUMINOUS CONCRETE — 0.8	
							42.8		CONCRETE — 1.5	
5									VOID	
							37.5		6.8 — FILL. See Boring B1 for similar overburden.	
10							33.5		10.8	
	R1	21.6/24	11.2 - 13.0	RQD = 0%		NQ ~~CORE~~	33.1		Spun NW casing to 11.1 ft. Roller bit through weathered bedrock from 10.8 to 11.2 ft. — 11.2	
	R2	12/15	13.0 - 14.0	RQD = 58%					R1: Hard, slightly to moderately weathered, dark gray aphanitic METAPELITE. Primary joint set is extremely close to close, horizontal to moderate, open, planar to undulating and rough. High angle secondary joints present throughout. Core is moderately to highly fractured. Rusty discolorations on joints; many vugs, pits.	
15	R3	57.6/57	14.0 - 18.8	RQD = 44%					R2: Same lithology as R1. One horizontal joint and one high angle joint present. Joints are open, close, planar to undulating and rough. Some vugs, rusty discolorations.	
									R3: Same lithology as R1 and R2. Primary joints same as R1, except extremely close to moderate. High angle secondary joint present to 17.5 ft. Rusty discolorations on joints. Solid core from 17.5 to 18.8 ft.	
							25.5		18.8 — **Bottom of Exploration at 18.8 feet below ground surface.**	
20										
25										
30										

Remarks:

Rope/cathead, safety hammer. 6 in. thinwall core of pavement

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.: B5

Appendix B

Calculations

08335
Replacement of Eliot Road Overpass Carrying Route 1 Bypass over Route 103, Kittery, Maine
P.I.N. 15099.00

11/18/08
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K. Recker
Checked by: J. Haskell

Rock Description

Boring No.	Core Depth (Ft)	RQD	Description
B1	18.3 – 20.2	26	Hard, slightly to moderately weathered, dark gray METAPELITE. Primary joint set is extremely close to close, horizontal to moderate, partly open wide, planar to undulating and rough.
	20.4 – 25.8	17	Same as above.
	25.8 – 29.1	65	Same as above.
B2	23.4 – 25.4	50	Moderately hard to hard, slightly weathered, dark gray coarse to fine grained GABBRO. Primary joint set is very close to close, horizontal to low, open, planar to undulating and rough.
	25.4 – 29.7	90	Same as above.
B3	15.6 – 20.3	55	Hard, slightly weathered, dark gray aphanitic METAPELITE. Primary joint set is extremely close to moderate, horizontal to moderate, open, planar to undulating and rough.
B4	16.0 – 17.9	57	Hard, slightly weathered, dark gray aphanitic to fine-grained BASALT. Primary joint set is extremely close to close, horizontal to moderate, open, planar to undulating and rough.
	17.9 – 18.9	0	Same as above
	18.9 – 21.3	76	Same as above.
	21.3 – 26.4	85	Same as above but with fine-grained ANDESITE intrusions.
B5	11.2 – 13.0	0	Hard, slightly to moderately weathered, dark gray aphanitic METAPELITE. Primary joint set is extremely close to close, horizontal to moderate, open, planar to undulating and rough.
	13.0 – 14.0	58	Same as above.
	14.0 – 18.8	44	Same as above.

08335
Replacement of Eliot Road Overpass Carrying Route 1 Bypass over Route 103, Kittery, Maine
P.I.N. 15099.00

11/18/08
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K. Recker
Checked by: J. Haskell

PART I – NOMINAL BEARIBNG RESISTANCE FOR SERVICE LIMIT ANALYSES

Method 1 – Presumptive Bearing Resistance for Service Limit State

Reference: Foundation Engineering, Second Edition, Peck, Hanson & Thornburn, 1974, Table 22.2, "Allowable Contact Pressure q_a on Jointed Rock."

Bearing Material:	Metapelite, Gabbro and Basalt
Description:	Hard, slightly to moderately weathered, primary joint sets are extremely close to close, horizontal to moderate, open, planar to undulating and rough. RQD varies from 0 to 90%, average = 48%.
Allowable Bearing Stress:	Range: 10 to 200 tons per square foot (tsf)
Design Value:	Recommended value = 10 tsf = 20 kips per square foot (ksf)

Method 2 – Presumptive Bearing Resistance for Service Limit State

Reference: NAVFAC DM 7.2, September 1986, Foundations and Earth Structures, Table 1, Page 7.2-142, "Presumptive Values of Allowable Bearing Pressure for Spread Footings."

Bearing Material:	Metapelite, Gabbro and Basalt
Description:	Hard, slightly to moderately weathered, primary joint sets are extremely close to close, horizontal to moderate, open, planar to undulating and rough. RQD varies from 0 to 90%, average = 48%.
Presumptive Bearing Stress:	Range: 8 to 12 tons per square foot (tsf)
Design Value:	Recommended value = 10 tsf = 20 kips per square foot (ksf)

Method 3 – Presumptive Bearing Resistance for Service Limit State

Reference: AASHTO LRFD Bridge Design Specifications 4th Edition 2007, Table C10.6.2.6.1-1, "Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material:	Metapelite, Gabbro and Basalt
Description:	Hard, slightly to moderately weathered, primary joint sets are extremely close to close, horizontal to moderate, open, planar to undulating and rough. RQD varies from 0 to 90%, average = 48%.
Presumptive Bearing Stress:	Range: 16 to 24 kips per square foot (ksf)
Design Value:	Recommended value = 20 ksf

Nominal Bearing Resistance for Service Limit State

Use 20 ksf for the nominal bearing resistance based on Methods 1, 2 and 3 above.

$q_{nominal} = 20$ ksf

Resistance Factor for Service Limit State

$\varphi_r = 1.0$ (AASHTO Table 3.4.1-1)

Factored Bearing Resistance for Service Limit State Analyses; Settlement Limited to 1.0 Inch

$q_{factored} = \varphi_r \times q_{nominal}$

$q_{factored} = 20$ ksf

PART II – FACTORED BEARING RESISTANCE FOR STRENGTH LIMIT ANALYSES

Method 1 – Nominal and Factored Bearing Resistance of Bedrock

Reference: US Army Corps of Engineers, Engineering and Design, EM1110-1-2908, 30 November 1994, "Rock Foundations."

Figure 6-1.c, for open joints, mode of failure is compressive failure of individual rock columns

Equation 6-5, page 6-5

$q_{ult} = 2\ c \tan(45 + \varphi/2)$

Where:
c = 90 pounds per square inch = 12,960 pounds per square foot (cohesion selected from reference "estimation of Soil and Rock Properties for Foundation Design", by Kulhawy, 2005, Short Course Lecture Notes MDOT. Geological Strength Index = 40, ratio of cohesion strength/uniaxial strength of intact rock = 0.03, compressive strength = 3,000 psi)
φ = 30 deg. (Based on review of typical values shown in Goodman, Table 3.3, page 78)

$q_{ult} = 2(12{,}960) \tan(45 + 30/2)$

$q_{ult} = 44{,}894 \text{ psf} = 44.9 \text{ ksf} = q_{nominal}$

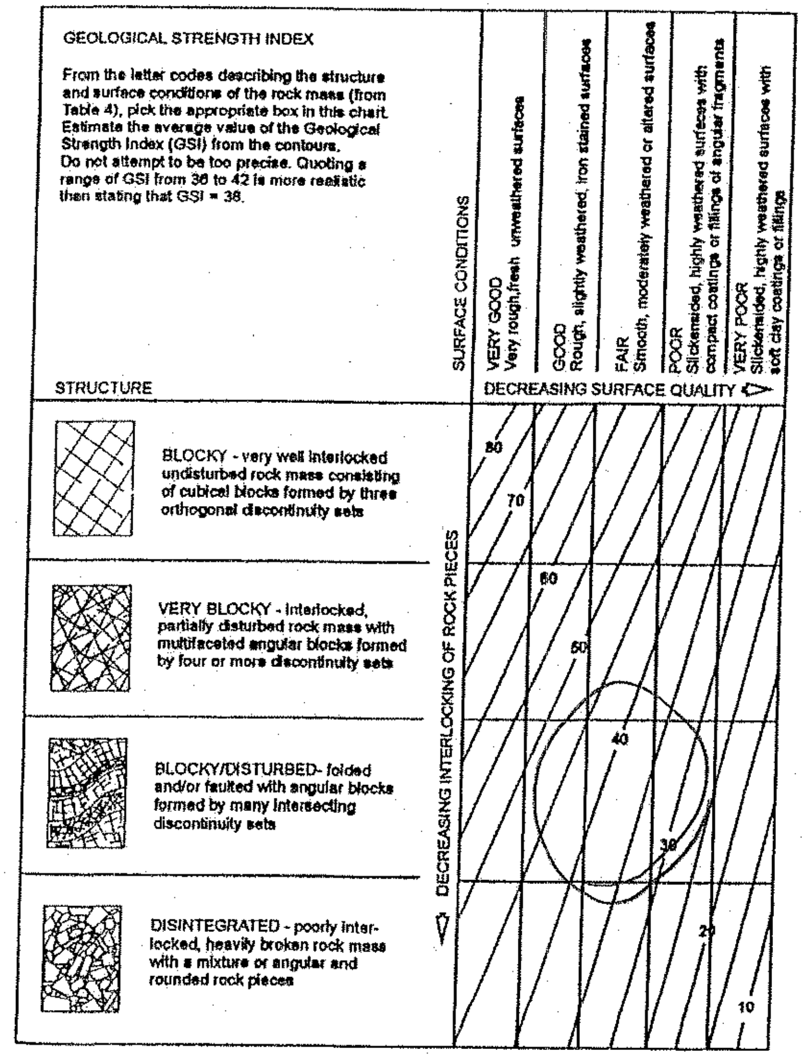
Factored Bearing Resistance

Per AASHTO LRFD Table 10.5.5.2.2-1, for footings on rock, $\varphi_b = 0.45$

$q_{factored} = q_{nominal}\ (\varphi_b) = 44.9\ (0.45)$

$q_{factored} = 20.2 \text{ ksf}$

GEOLOGICAL STRENGTH INDEX



(Hoek & Brown, IJRMMS, 1997)

ATHENS ROCK PARAMETERS

Rock mass characteristics and estimated shear strengths for different rock mass units

Rock mass type	UCS σ_{ci} MPa	Constant m_i	Estimated GSI	Cohesion c MPa	Friction angle $\phi°$
Limestone	54	10	50 ± 10	2.3 ± 0.4	35 ± 2
Sericite sandstone	37	19	50 ± 10	1.7 ± 0.2	37 ± 2
Greywacke	25	18	30 ± 8	0.7 ± 0.1	31 ± 2
Dark grey siltstone	18	9	30 ± 8	0.55 ± 0.2	25 ± 2
Black shales (classified as Disintegrated)	1–5	8	15 ± 8	0.05 ± 0.04	19 ± 3
Black shales (classified in the new Foliated laminated/ sheared, rock structure)	1–5	8	10 ± 6	0.04 ± 0.03	17 ± 2

(Hoek, Marinos & Benissi, Bull IAEG, 1998)

ESTIMATING c & ϕ FROM H & B

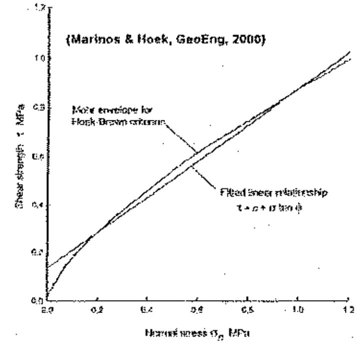
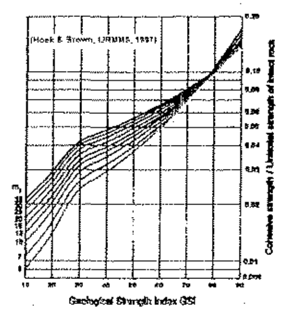
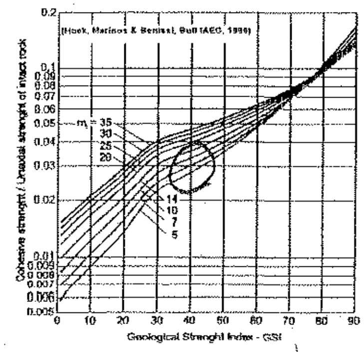


Figure 5. Mohr envelope for Hoek Brown criterion and fitted linear relationship for the normal stress range $0 < \sigma_n < \sigma_v$ where σ_v = depth x unit weight.

ESTIMATED "COHESION"

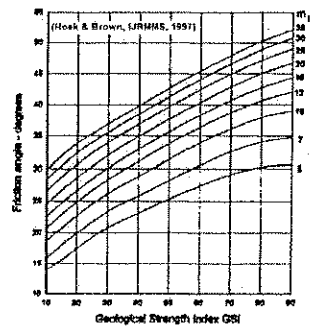


INCORRECT

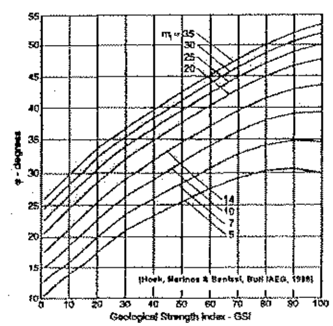


Correct

ESTIMATED "FRICTION ANGLE"



INCORRECT



Correct

Ref: "Introduction to Rock Mechanics" Goodman.

TABLE 3.3

Representative Values for Shear Strength Intercept (S_i) and Angle of Internal Friction (ϕ) for Selected Rocks

Description	Porosity (%)	S_i (MPa)	ϕ	Range of Confining Pressure (MPa)	Reference[b]
Berea sandstone	18.2	27.2	27.8	0–200	4
Bartlesville sandstone		8.0	37.2	0–203	3
Pottsville sandstone	14.0	14.9	45.2	0–68.9	8
Repetto siltstone	5.6	34.7	32.1	0–200	4
Muddy shale	4.7	38.4	14.4	0–200	4
Stockton shale		0.34	22.0	0.8–4.1	2
Edmonton bentonitic shale (water content 30%)	44.0	0.3	7.5	0.1–3.1	9
Sioux quartzite		70.6	48.0	0–203	3
Texas slate; loaded					
30 degrees to cleavage		26.2	21.0	34.5–276	6
90 degrees to cleavage		70.3	26.9	34.5–276	6
Georgia marble	0.3	21.2	25.3	5.6–68.9	8
Wolf Camp limestone		23.6	34.8	0–203	3
Indiana limestone	19.4	6.72	42.0	0–9.6	8
Hasmark dolomite	3.5	22.8	35.5	0.8–5.9	4
Chalk	40.0	0	31.5	10–90	1
Blaine anhydrite		43.4	29.4	0–203	3
Inada biotite granite	0.4	55.2	47.7	0.1–98	7
Stone Mountain granite	0.2	55.1	51.0	0–68.9	8
Nevada Test Site basalt	4.6	66.2	31.0	3.4–34.5	10
Schistose gneiss					
90 degrees to schistocity	0.5	46.9	28.0	0–69	2
30 degrees to schistocity	1.9	14.8	27.6	0–69	2

[a] Data from Kulhawy (1975), (Ref. 5)

[b] 1. Dayre, M., Dessene, J. L., and Wack, B. (1970) *Proc. 2nd Congress of ISRM*, Belgrade, Vol. 1, pp. 373–381.
2. DeKlotz, E., Heck, W. J., and Neff, T. L. (1964) First Interim Report, *MRD Lab Report 64/493*, U. S. Army Corps of Engineers, Missouri River Division.
3. Handin, J., and Hager, R. V. (1957) *Bull. A.A.P.G.*., Vol. 41, pp. 1–50.
4. Handin, J., Hager, R. V., Friedman, M., and Feather, J. N. (1963) *Bull. A.A.P.G.*, Vol. 47, pp. 717–755.
5. Kulhawy, F. (1975) *Eng. Geol.*, Vol. 9, pp. 327–350.
6. McLamore, R. T. (1966) Strength-deformation characteristics of anisotropic sedimentary rocks, Ph.D. Thesis, University of Texas, Austin.

Typical values of the peak shear strength intercep internal friction ϕ for a representative set of rock spec The ratio of unconfined compressive to tensile stren rock types is given in Table 3.1.

3.7 THE EFFECT OF WATER

Some rocks are weakened by the addition of water, deterioration of the cement or clay binder. A friable sa 15% of its strength by mere saturation. In extreme case clay shales, saturation is totally destructive. In most ca of pore and fissure water pressure that exerts the greates If drainage is impeded during loading, the pores or contained water, raising its pressure.

Development of pore pressure and consequent loss vanian shale tested in triaxial compression is shown in Gibala (1972). Two separate test results are presented represent triaxial compression of a saturated specimen excess pore pressures could drain away rather than a ditions"); the triangles represent a saturated shale drainage, so that excess pore pressures that develop mu conditions"). The curve of differential axial stress versus test displays a peak and then a descending tail as depict mean stress increases simultaneously with the axial st curve of volumetric strain shown in Figure 3.17 is the pression (Figure 3.6) and dilatancy behavior (Figure 3 decreases by hydrostatic compression until the specime upon the rate of volume decrease slows, eventually bec the volume increases on subsequent load increments. tendency for volume change cannot be fully realized be voids undergoes compression rather than drainage. As a (p_w) inside the pores begins to increase. This dramatically flattens the postpeak curve.

Many investigators have confirmed the validity of Ter for rocks, which states that a pressure of p_w in the pore

7. Mogi, K. (1964) *Bull. Earthquake Res. Inst.*, Tokyo, Vol. 42,
8. Schwartz, A. E. (1964) *Proc. 6th Symp. on Rock Mech.*, Rol
9. Sinclair, S. R. and Brooker, E. W. (1967) *Proc. Geotech. C perties of Natural Soils and Rocks*, Oslo, Vol. 1, pp. 295–299
10. Stowe, R. L. (1969) U. S. Army Corps of Engineers Wate Vicksburg, *Misc. Paper C-69-1*.