

**MAINE DEPARTMENT OF TRANSPORTATION  
BRIDGE PROGRAM  
GEOTECHNICAL SECTION  
AUGUSTA, MAINE**

**GEOTECHNICAL DESIGN REPORT**

*For the Superstructure Replacement and  
Abutment Reuse of:*

**BUFFAM BRIDGE  
U.S. ROUTE 1 OVER WEBHANET RIVER  
WELLS, MAINE**

Laura Krusinski, P.E.  
Senior Geotechnical Engineer



York County  
WIN 23535.00

Soils Report 2019-56  
Bridge No. 2107

Fed. No. 2353500  
August 27, 2019

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## 1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical design recommendations for the reuse of the existing concrete gravity abutments at Buffam Bridge, which carries U.S. Route 1 over Webhanet River in Wells, Maine.

The existing bridge was constructed in 1931 and is a single slab on concrete gravity abutments. The historical bridge plans indicate the abutments are supported on timber piles. However, the 1931 Construction Diary indicates the upstream half of each abutment footing is founded on bedrock and the downstream halve of each footing is not bearing on bedrock and are instead bearing on 3 to 6 feet of gravel.

According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the superstructure is rated 4 for poor condition and the substructure is rated 6 for satisfactory condition. The inspection report notes an  $\frac{1}{8}$ " crack the full height of the Abutment No. 2 extending into two granite blocks, and an  $\frac{1}{16}$ " thick full height vertical crack in Abutment No. 1 that extends into one granite block. Abutment No. 1 is scoured due to the stream alignment. There is heavy cracking and efflorescence at the southeast corner of the structure. The Sufficiency Rating of the bridge is 52.7.

The MaineDOT has elected to repurpose the existing abutments and replace the superstructure. Using the geotechnical parameters provided in Appendix D – Geotechnical Memorandum, the MaineDOT designer completed evaluations for bearing resistance, sliding, and overturning (eccentricity).

## 2.0 GEOLOGIC SETTING

The existing structure carries U.S. Route 1 over Webhanet River as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Wells Quadrangle Open-File No. 99-104 (1999) indicates the surficial soils in the vicinity of the bridge project consist of muck, peat, silt, and sand from wetlands and saltmarsh.

The MGS Bedrock Map of Maine (1985) indicates that the bedrock at the project site is calcareous feldspathic sandstone of the Kittery Formation. Bedrock cores retrieved at the site are identified as metasandstone.

## 3.0 SUBSURFACE INVESTIGATION

Six test borings were drilled at the site: BB-WWR-101, BB-WWR-101A were drilled at existing Abutment No. 1 and BB-WWR-102, BB-WWR-102A, BB-WWR-102B, and BB-WWR-103 was drilled at existing Abutment No. 2. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The test borings were drilled between October 10 and 13, 2018 by MaineDOT. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring and rock coring techniques. Soil samples were typically obtained in 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon sampler. The automatic hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in June of 2018. All N-values discussed in this report are corrected values computed by applying the corresponding average energy transfer factor of 0.928 to the raw field N-values. The hammer efficiency factor (0.928) and both the raw field N-values and the corrected N-values are shown on the boring logs provided in Appendix A – Boring Logs and Sheet 3 – Boring Logs.

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

#### **4.0 LABORATORY TESTING**

A laboratory testing program was conducted on one soil sample recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soil, and geologic assessment of the project site. Laboratory testing consisted of one standard grain size analysis with natural water content. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

## 5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of granular fill, concrete, and gravel underlain by bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The following paragraphs discuss the subsurface conditions and foundation bearing conditions encountered:

### 5.1 Abutment No. 1

Two test borings explored Abutment No. 1 and the foundation bearing conditions. Boring BB-WWR-101 was terminated above Abutment No. 1. Boring BB-WWR-101A was drilled through the Abutment and terminated in a layer of rounded gravel underlying the abutment footing.

The fill layers and structural concrete encountered were, in order of increasing depth:

- 8 inches HMA
- 1 inch soil layer;
- 7-inch thick concrete slab;
- 6-inch to 1-foot void;
- brown, damp, coarse sand;
- several pieces of coarse gravel;
- concrete cored from 12.0 to 17.3 ft bgs;
- possible abutment heel roller coned from 14.0 to 15.5 ft bgs;
- rounded gravel.

One SPT  $N_{60}$ -value in the coarse gravel backfill was 29 blows per foot (bpf), indicating the layer is medium dense.

### 5.2 Abutment No. 2

Four test borings explored Abutment No. 2 and the foundation bearing conditions. Boring BB-WWR-102 was terminated above the abutment backface. Boring BB-WWR-102A encountered the abutment backface at approximately 9.3 feet bgs but slipped down the back batter bending the casing. Five (5) feet of steel casing was abandoned in the borehole and the boring terminated. Boring BB-WWR-102B cored Abutment No. 2 concrete but the casing could not be advanced through the abutment. Therefore, a roller cone was advanced beneath the footing and through an unidentified layer (neither bedrock nor concrete) to greater resistance (possible bedrock). Boring BB-WWR-103 was drilled behind Abutment No. 2 and cored bedrock.

The fill layers and structural concrete encountered were, in order of increasing depth:

- 8 inches HMA
- 9 inches concrete slab;
- brown sand;
- dark brown, wet, sandy gravel, little silt;
- black to reddish brown, wet, gravel, some sand, little silt, piece of wood;
- 9.0 feet of abutment concrete in BB-WWR-102B.

SPT N<sub>60</sub>-values in the gravel backfill ranged from 5 to 42 bpf, indicating the gravel is loose to dense in consistency. One grain size analysis of a sample from the gravel unit resulted in a soil classification of A-2-4 under the AASHTO Soil Classification System and SM under the Unified Soil Classification System (USCS). The natural water content of the sample tested was approximately 16 percent.

### 5.3 Bedrock

Bedrock was encountered and cored in one boring. Table 1 summarizes approximate depth to bedrock, corresponding approximate top of the bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD R1, R2 (%)
BB-WWR-103	1612+24.3	12.3 Rt.	21.8	-5.2	69, 52

**Table 1** – Summary of Approximate Bedrock Depths and Elevations

The bedrock recovered from the boring is generally identified as grey to light grey, fine grained, Metasandstone, very hard, fresh, horizontal to low angle joints with several steep quartz filled joints and veins along many of the joints, close spacing, closed, with a shatter zone from 22.5-23.1 ft bgs. The RQD of the bedrock ranged from 52 to 69 percent correlating to a rock quality of fair. Detailed bedrock descriptions and the RQD of each core run are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

### 5.4 Groundwater

Groundwater was not observed in the boreholes. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

## **6.0 ABUTMENT FOUNDATION CONDITIONS**

### **6.1 Inferred Footing Conditions**

Abutment footing bearing conditions, as inferred from the 2019 subsurface explorations are detailed above in Sections 5.1 and 5.2.

Abutment footing bearing conditions documented in the 1931 Construction Diary are provided in Appendix C – Pages from 1931 Construction Diary.

## **7.0 PROJECT ALTERNATIVES**

### **7.1 Abutment Reuse**

Geotechnical design parameters for the evaluation of the existing abutments for a superstructure replacement were provided in a Geotechnical Design Memorandum of May 30, 2017, and revised on July 12, 2019. The memorandum is provided in Appendix D – Geotechnical Memorandum.

## **8.0 CLOSURE**

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

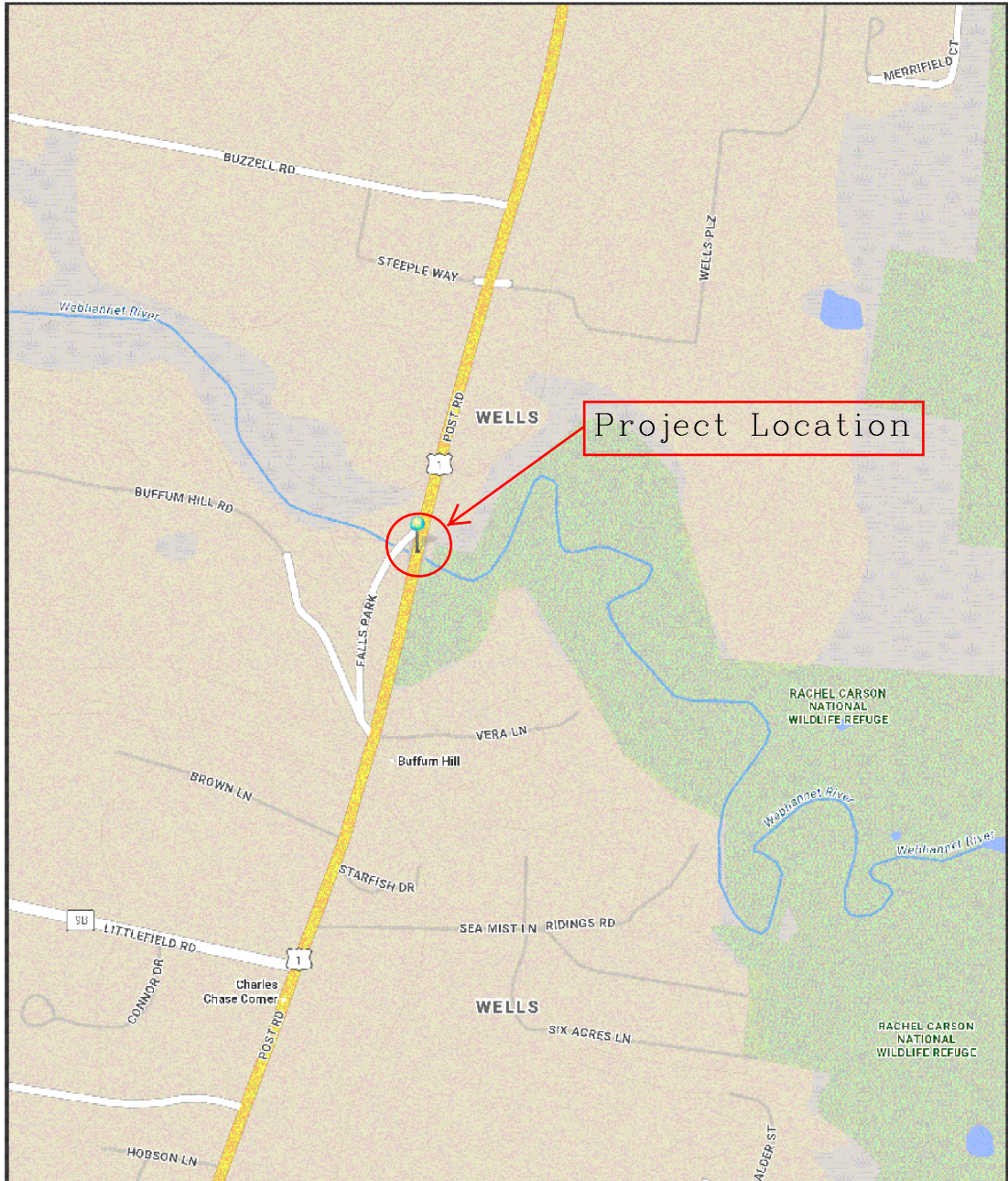
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes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

It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that geotechnical parameters for abutment reuse presented in this report are properly interpreted and implemented in the design and specifications.

## **Sheets**



# WELLS, MAINE

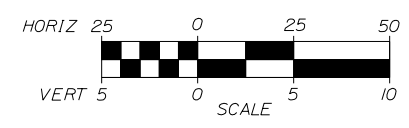
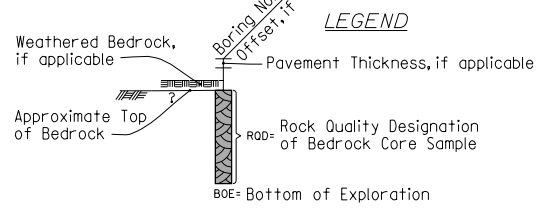
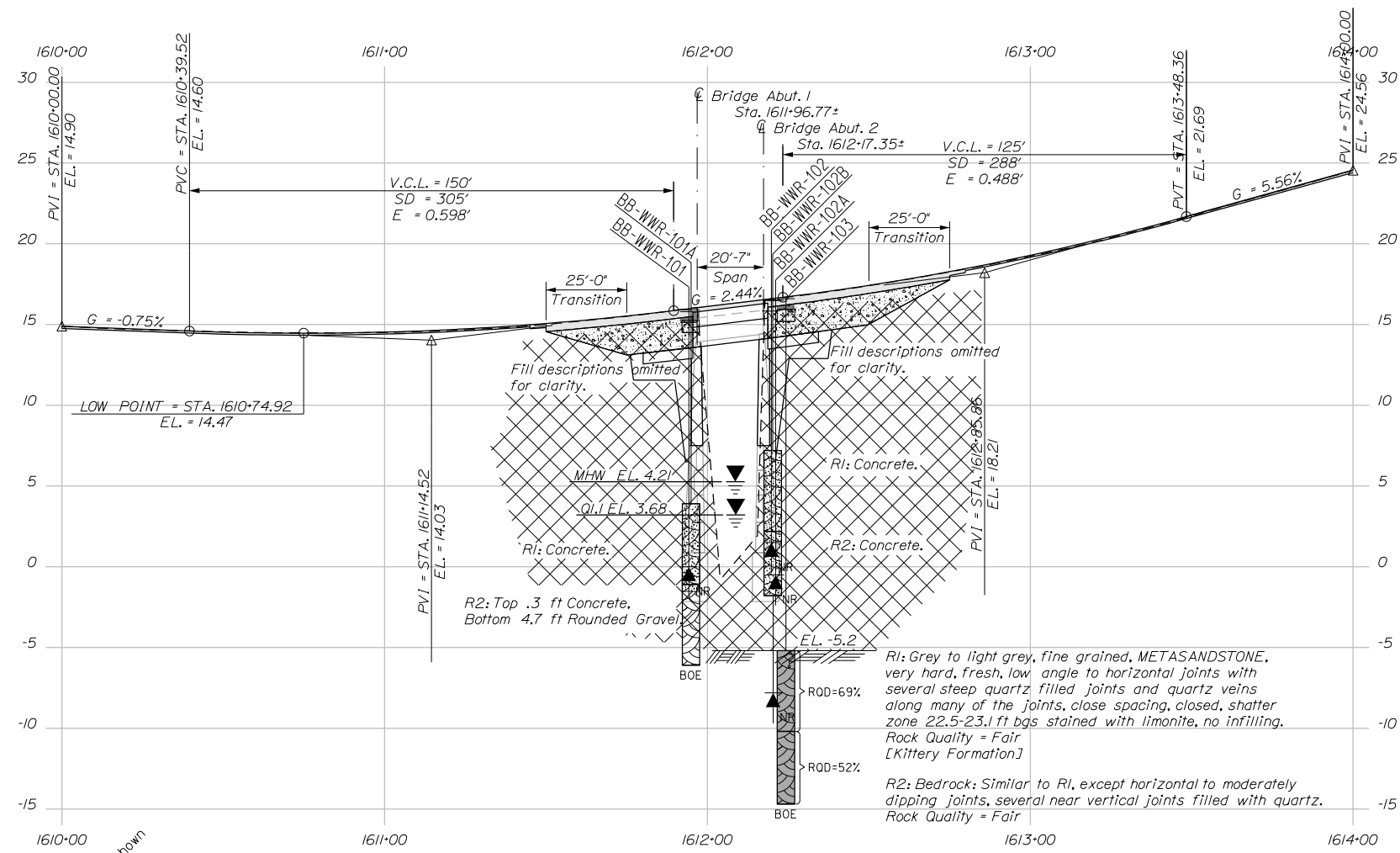
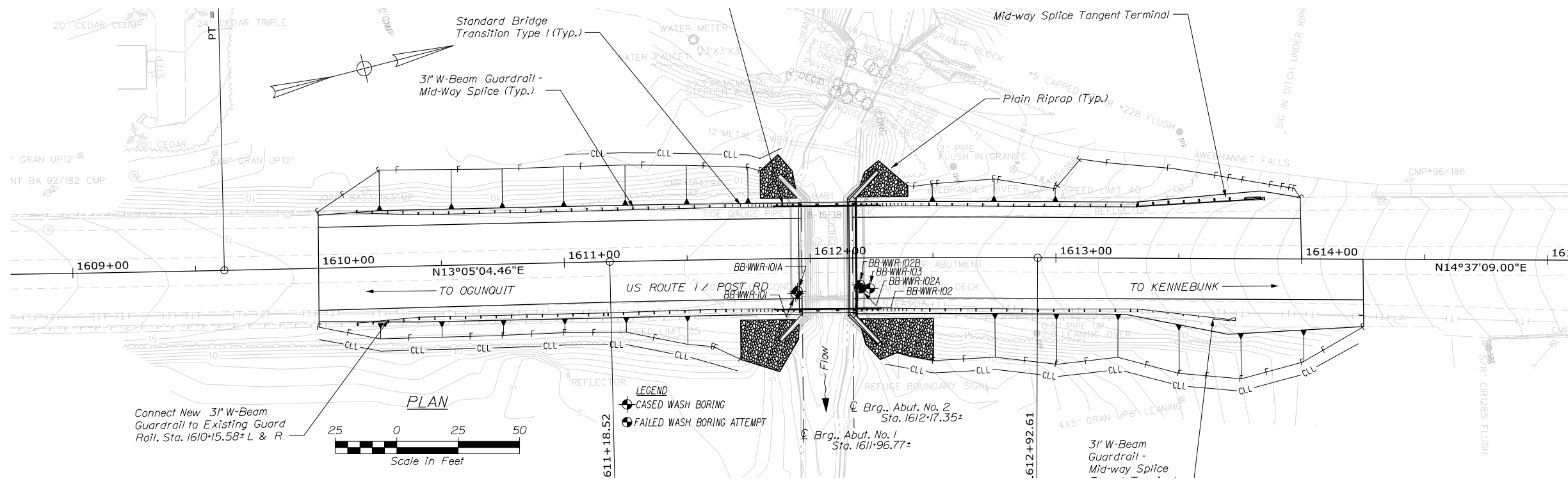


The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information.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

0.1 Miles  
1 inch = 0.11 miles

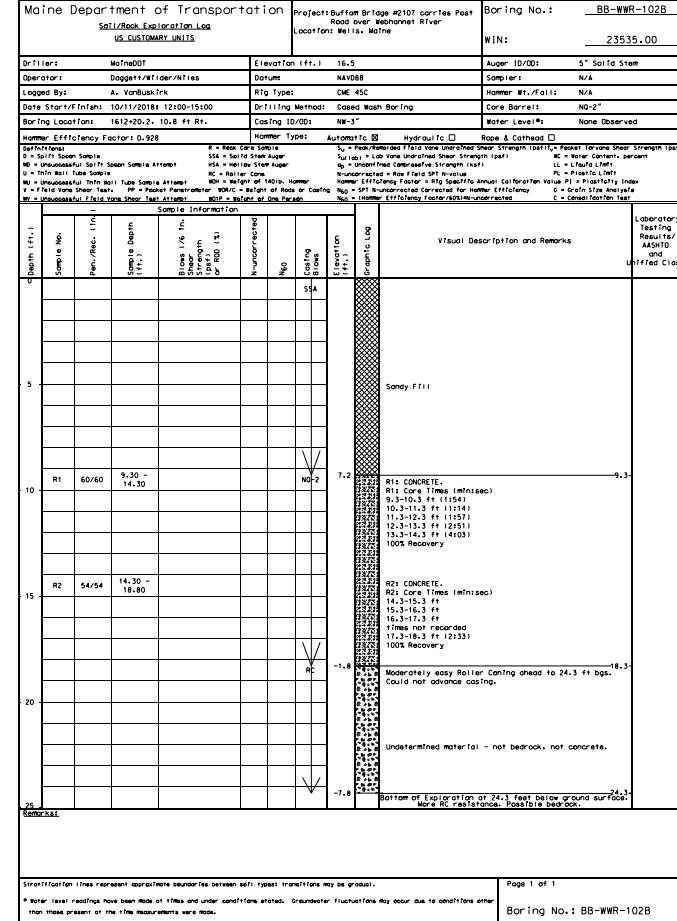
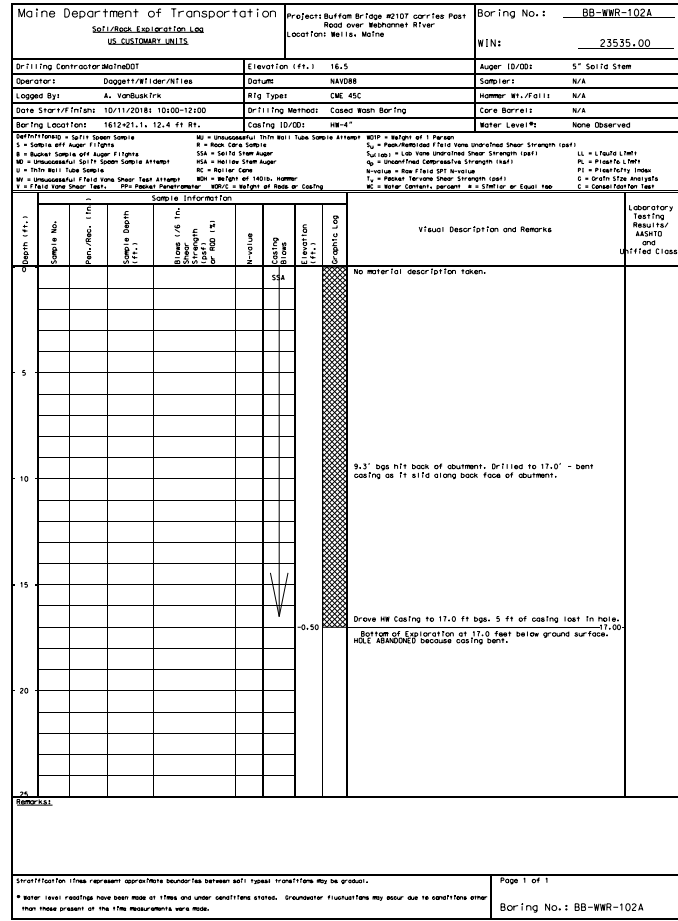
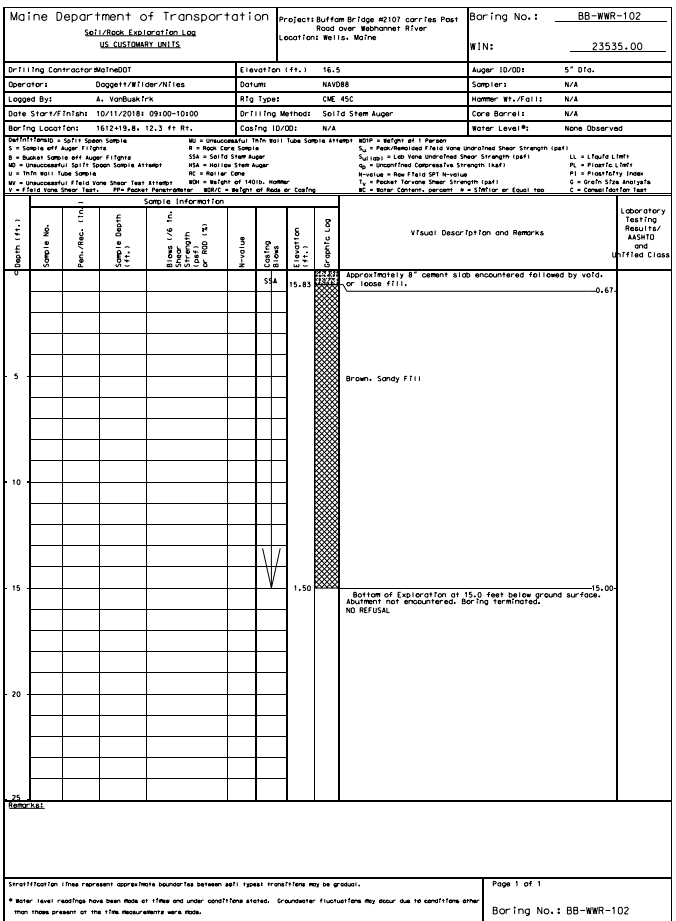
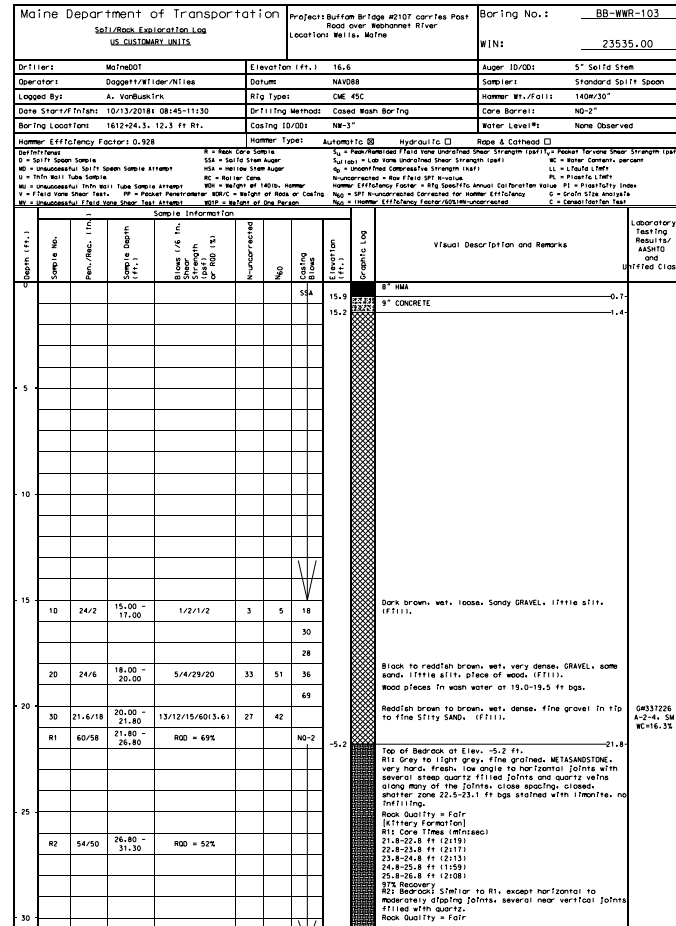
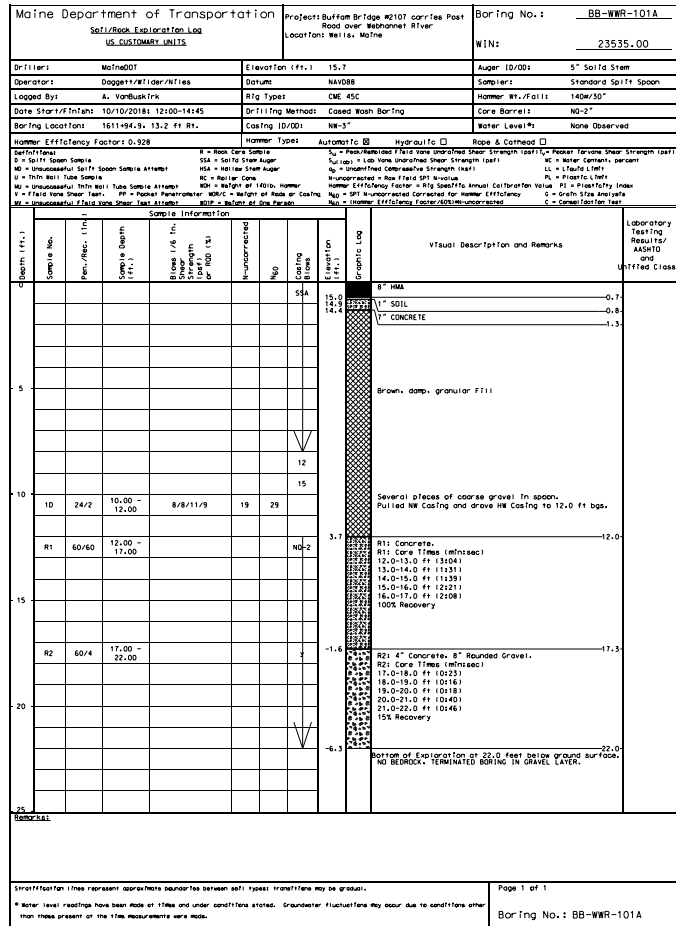
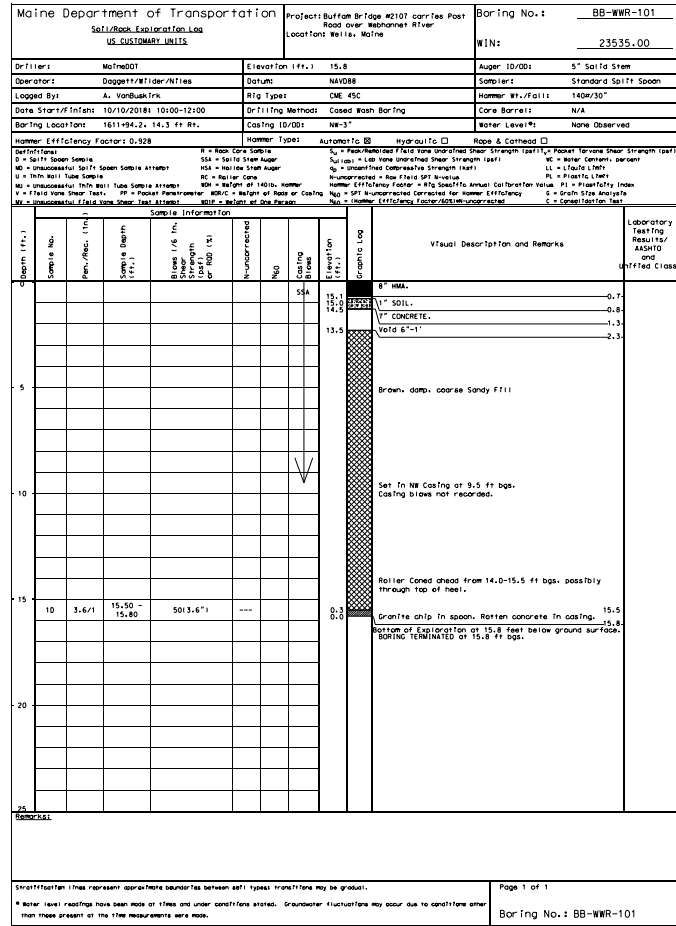
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SHEET NUMBER  <b>1</b>	BUFFUM BRIDGE WEBHANNET RIVER		STATE OF MAINE DEPARTMENT OF TRANSPORTATION	
	WELLS	YORK COUNTY	<b>023535.00</b>	
OF 3	<b>LOCATION MAP</b>		<b>WIN</b>	<b>23535.00</b>
			BRIDGE NO. 2107	BRIDGE PLANS



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

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION		023535.00	
WELLS		YORK COUNTY		BRIDGE NO. 2107	
BUFFUM BRIDGE		WEBHANNET RIVER		WIN 23535.00	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE		SHEET NUMBER		BRIDGE PLANS	
PROJ. MANAGER	DATE	BY	SIGNATURE	P.E. NUMBER	DATE
CHECKED-REVIEWED	NOV 2023	T. WHITE	[Signature]		
DESIGNS-DETAILED		L. KRUSINSKI			
DESIGNS-DETAILED					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					
2					
OF 3					



STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION  
 02535-00  
 BRIDGE NO. 2107  
 WIN  
 23535.00  
 BRIDGE PLANS

PROJECT: BUFFUM BRIDGE WEBHANNET RIVER  
 LOCATION: WELLS, MAINE

DESIGN-DETAILED  
 CHECKED-REVIEWED  
 DESIGNS-DETAILED  
 DESIGNS-DETAILED  
 REVISIONS 1  
 REVISIONS 2  
 REVISIONS 3  
 REVISIONS 4  
 FIELD CHANGES

NOV 2023  
 T. WHITE  
 L. KRUSINSKI

SIGNATURE  
 P.E. NUMBER  
 DATE

WELLS  
 YORK COUNTY  
 BORING LOGS  
 SHEET NUMBER  
 3  
 OF 3

## **Appendix A**

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	
	SANDS  (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	
		GC	Clayey gravels, gravel-sand-clay mixtures.		
		CLEAN SANDS	SW	Well-graded sands, Gravelly sands, little or no fines	
		(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.	
FINE-GRAINED SOILS  (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS  (liquid limit less than 50)	SM	Silty sands, sand-silt mixtures		
		SC	Clayey sands, sand-clay mixtures.		
		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.		
	SILTS AND CLAYS  (liquid limit greater than 50)	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.			
MH		Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.			
HIGHLY ORGANIC SOILS	Pt	CH	Inorganic clays of high plasticity, fat clays.		
		OH	Organic clays of medium to high plasticity, organic silts.		
		Pt	Peat and other highly organic soils.		
<b>Desired Soil Observations (in this order, if applicable):</b>				<b>Desired Rock Observations (in this order, if applicable):</b>	
Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level				Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))	
<b>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</b>				<b>Sample Container Labeling Requirements:</b>	
				WIN	Blow Counts
				Bridge Name / Town	Sample Recovery
				Boring Number	Date
				Sample Number	Personnel Initials
				Sample Depth	

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

**TERMS DESCRIBING DENSITY/CONSISTENCY**

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

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

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

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

**Rock Quality Designation (RQD):**  
 RQD (%) =  $\frac{\text{sum of the lengths of intact pieces of core}^* > 4 \text{ inches}}{\text{length of core advance}}$   
 \*Minimum NQ rock core (1.88 in. OD of core)

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









<b>Drilling Contractor:</b> MaineDOT	<b>Elevation (ft.):</b> 16.5	<b>Auger ID/OD:</b> 5" Solid Stem
<b>Operator:</b> Daggett/Wilder/Niles	<b>Datum:</b> NAVD88	<b>Sampler:</b> N/A
<b>Logged By:</b> A. VanBuskirk	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> N/A
<b>Date Start/Finish:</b> 10/11/2018; 12:00-15:00	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 1612+20.2, 10.8 ft Rt.	<b>Casing ID/OD:</b> NW-3"	<b>Water Level*:</b> None Observed

Definitions: D = Spilt Spoon Sample      MU = Unsuccessful Thin Wall Tube Sample Attempt      WO1P = Weight of 1 Person  
 S = Sample off Auger Flights              R = Rock Core Sample                              S<sub>u</sub> = Peak/Remolded Field Vane Undrained Shear Strength (psf)  
 B = Bucket Sample off Auger Flights      SSA = Solid Stem Auger                              S<sub>u(lab)</sub> = Lab Vane Undrained Shear Strength (psf)      LL = Liquid Limit  
 MD = Unsuccessful Split Spoon Sample Attempt      HSA = Hollow Stem Auger                              q<sub>p</sub> = Unconfined Compressive Strength (ksf)      PL = Plastic Limit  
 U = Thin Wall Tube Sample                  RC = Roller Cone                                      N-value = Raw Field SPT N-value                  PI = Plasticity Index  
 MV = Unsuccessful Field Vane Shear Test Attempt      WOH = Weight of 140lb. Hammer              T<sub>v</sub> = Pocket Torvane Shear Strength (psf)      G = Grain Size Analysis  
 V = Field Vane Shear Test      PP = Pocket Penetrometer      WOR/C = Weight of Rods or Casing              WC = Water Content, percent      ≐ = Similar or Equal too      C = Consolidation Test

Depth (ft.)	Sample Information								Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log						
0						SSA			Sandy Fill					
5														
7.2	R1	60/60	9.30 - 14.30			NQ-2	7.2			R1: CONCRETE. R1: Core Times (min:sec) 9.3-10.3 ft (1:54) 10.3-11.3 ft (1:14) 11.3-12.3 ft (1:57) 12.3-13.3 ft (2:51) 13.3-14.3 ft (4:03) 100% Recovery	9.3			
10														
15	R2	54/54	14.30 - 18.80								R2: CONCRETE. R2: Core Times (min:sec) 14.3-15.3 ft 15.3-16.3 ft 16.3-17.3 ft times not recorded 17.3-18.3 ft (2:33) 100% Recovery	18.3		
18.3						RC	-1.8					Moderately easy Roller Coning ahead to 24.3 ft bgs. Could not advance casing.  Undetermined material - not bedrock, not concrete.		
20														
24.3							-7.8						Bottom of Exploration at 24.3 feet below ground surface.	24.3
25														

**Remarks:**





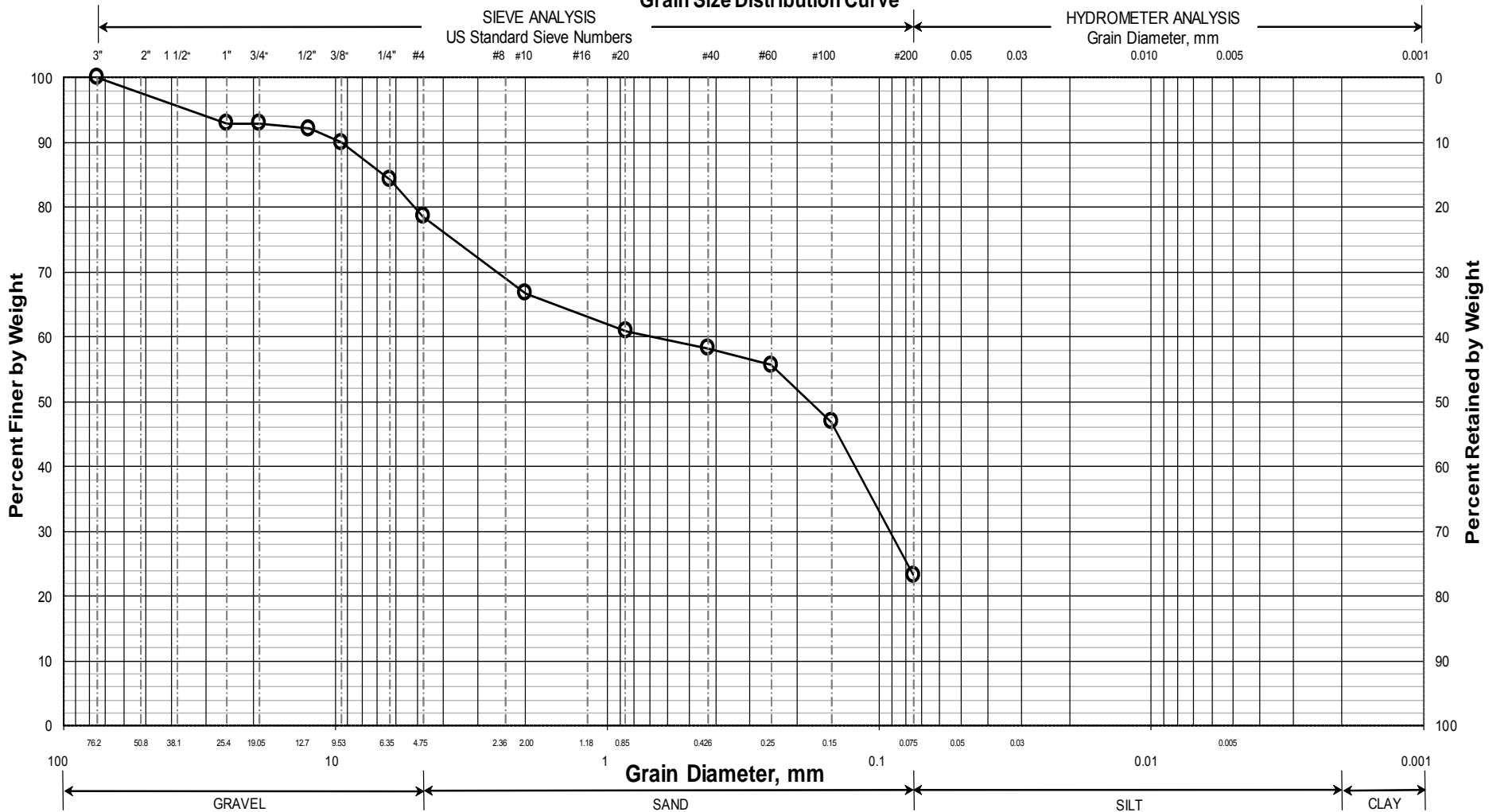


## **Appendix B**

Laboratory Test Results



## Maine Department of Transportation Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-WWR-103/3D	1612+24.3	12.3 RT	20.0-21.8	SAND, some silt, some gravel.	16.3			
◆									
■									
●									
▲									
X									

WIN
023535.00
Town
Wells
Reported by/Date
WHITE, TERRY A      1/22/2019

## **Appendix C**

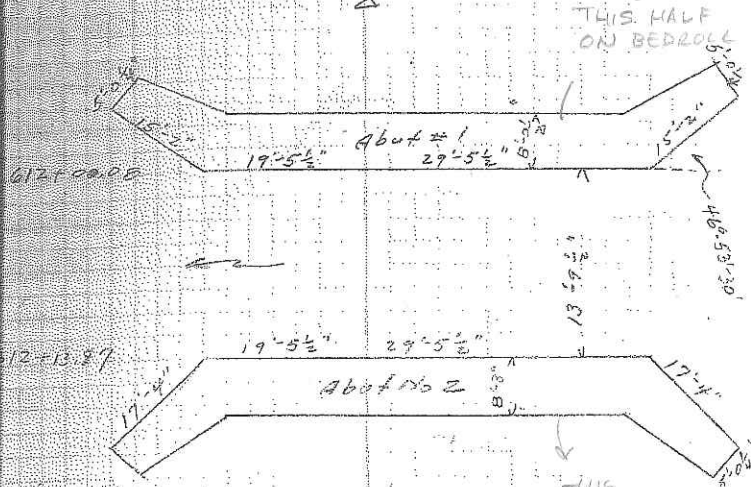
Pages from 1931 Construction Diary

612+50.70  
 13.7  
 612+42.63  
 612+13.87  
 28.76

Foresight Gable of house

Lay out of Footings

stakes at all corners



Spike in ground • sta. 612+42.63

Sta. Stangal  
 Royart  
 Lib. Harding

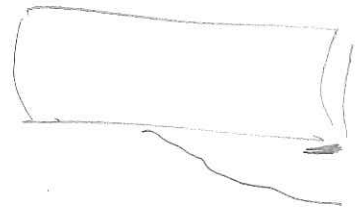
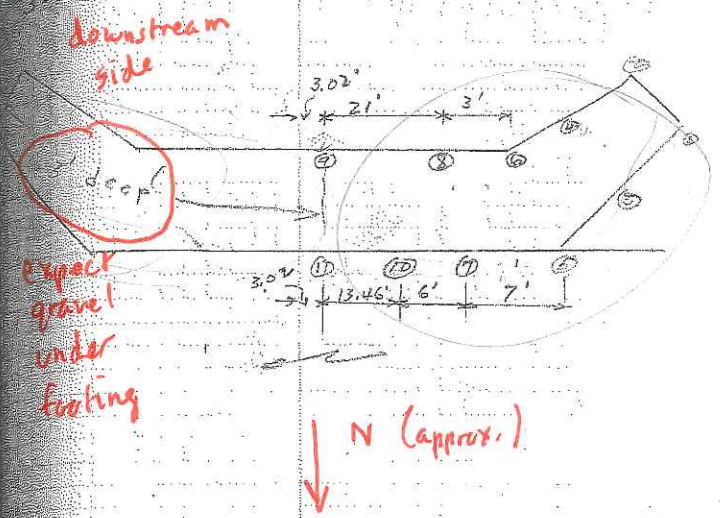
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Concrete in Footing No. 1

Point	Depth of ledge from top of footing
1	1.5'
2	1.17'
3	1.67'
4	1.67'
5	1.92'
6	1.50'
7	1.58'
8	2.0'
9	2.58'
10	2.58'
	2.58'
	3.0'

Footing No. 1



Core 17 of

5-14-51

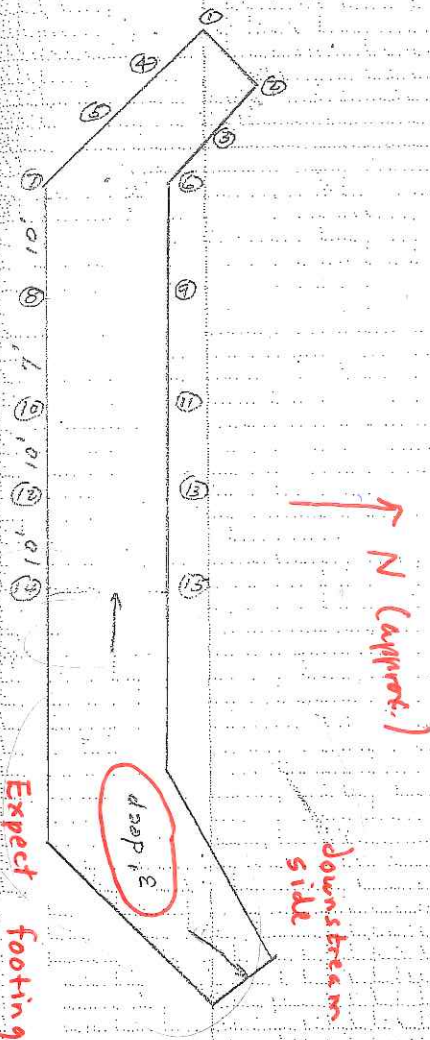
Footing Core

Footment No. 2

Footing No. 2

Point	Depth of ledge from top of footing	El. bottom of footing
1	1.08'	-0.24
2	1.17'	-0.33
3	1.17'	-0.33
4	1.17'	-0.33
5	1.00'	-0.16
6	1.17'	-0.33
7	1.00'	-0.16
8	1.00'	-0.16
9	1.00'	-0.16
10	1.58'	-0.74
11	1.58'	-0.74
12	2.17'	-1.33
13	2.17'	-1.33
14	2.50'	-1.66
15	2.50'	-1.66
	3.00'	-2.16

Top of footing 0.84



Expect footing on gravel

deep 1.8

downstream side

N (approx.)

## **Appendix D**

Geotechnical Memorandum

To: Roger Naous, P.E.  
From: Laura Krusinski, P.E.  
Cc: Garrett Gustafson, P.E.  
Subject: Preliminary Geotechnical Design Parameters for Abutment Reuse  
Buffam Bridge #2107  
Date: May 30, 2019 – **REVISED July 12, 2019**  
WIN: 23535.00  
Town: Wells

This memorandum provides preliminary geotechnical design parameters to evaluate repurposing the existing concrete cast-in-place abutments of Buffam Bridge in Wells, Maine. These design recommendations are in accordance with AASHTO LRFD Bridge Design Specifications 8<sup>th</sup> Edition, 2017 and MaineDOT Bridge Design Guide (BDG) Section 10.6 - Substructure Rehabilitation and BDG Section 10.7 - Substructure Reuse.

## 1.0 ABUTMENT REUSE

Repurposed abutments shall be evaluated to ensure that they meet current AASHTO LRFD standards for sliding, eccentricity, and bearing resistance. Repurposed and 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 bearing resistance, sliding and eccentricity.

### 1.1 Sliding

Based on the historical bridge plans (State Highway Commission, dated 1931) the existing South and North Abutments consist of cast-in-place concrete abutments on timber piles; but a handwritten note on the plans states “No Pile Driven”. Construction Diaries indicate that the upstream portions of both abutments were cast directly on bedrock and the downstream portions (approximately half) were not constructed on bedrock. Based on the borings conducted at the site in 2018, it is inferred that the downstream portions of both abutments are founded on gravel. Table 1 presents the resistance factors,  $\phi_{\tau}$ , for sliding analyses for the abutments.

Substructure	Bearing Material	Condition	Limit State	Sliding Resistance Factor $\phi_\tau$	LRFD Reference
Abutments 1 and 2 Upstream half	Bedrock	Cast-in-place footings	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
Abutments 1 and 2 Downstream half	Gravel	Cast-in-place footings	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

**Table 1 – Resistance Factors for Sliding**

The sliding resistance of the Abutments footing to lateral loads shall be calculated using the maximum coefficient of friction provided in Table 2:

Substructure	Interface Materials	Limit State	Friction Angle, $\delta_{soil}$	Coefficient of Friction, $\tan \delta^1$ (dim.)	$C^2$	LRFD Reference
Abutments 1 and 2 Upstream half	Mass concrete on clean sound rock	All	35°	0.70	1.0	Table C3.11.5.3-1
Abutments 1 and 2 Downstream half	Mass concrete on clean gravel, gravel-sand mixtures, and coarse sand	All	29°	0.55	1.0	Table C3.11.5.3-1

<sup>1</sup> Use a  $\tan \phi_f = \tan \delta$  in LRFD Equation 10.6.3.4-2 to calculate resistance to sliding

<sup>2</sup> Per LRFD Article 10.6.3.4 and Equation 10.6.3.4-2,  $C=1.0$  for concrete footings cast against soil

**Table 2 – Maximum Coefficients of Friction for Sliding**

Passive resistance of soil at the toe of the abutment footings should be neglected for resisting sliding.

**1.2 Bearing Resistance and Eccentricity**

Based on the historical bridge plans, construction diaries and the borings conducted at the site it is inferred that existing Abutments No. 1 and 2 are founded on both bedrock and gravel. Repurposed abutments shall be checked to ensure that they meet current LRFD standards against bearing capacity failure.

Application of permanent and transient loads is specified in LRFD Article 11.5.6. The stress distribution at both abutments may be assumed to be a distributed pressure over the effective base as shown in LRFD Figures 11.6.3.2-1 and -2. Table 4 summarizes the factored bearing resistances for the existing abutments:

Substructure	Assumed Bearing Material	Limit State	Resistance Factor, $\phi_b$	Factored Bearing Resistance (ksf)	LRFD Reference
Abutments 1 and 2 Upstream portion <sup>1</sup>	Bedrock	Service	1.0	20	Article 10.5.5.1
		Strength	0.45	47	Table 10.5.5.2.2-1
		Extreme	0.8	83	Article C11.5.8
Abutments 1 and 2 Downstream portion <sup>1</sup>	Gravel	Service	1.0	6	Article 10.5.5.1
		Strength	0.45	13.5	Table 10.5.5.2.2-1
		Extreme	0.8	24	Article C11.5.8

<sup>1</sup> Footing width of 8 feet assumed per State Highway Commission Plans.

**Table 4 – Bearing Resistances**

For each abutment footing, the eccentricity limits are presented in Table 5. LRFD Figures C11.5.6-2 and C11.5.6-4 illustrate the typical load factors to produce the extreme factored effect for evaluating eccentricity.

Substructure	Assumed Bearing Material	Location of Resultant Forces	LRFD Reference
Abutments 1 and 2 Upstream portion	Bedrock	Within the middle nine-tenths (9/10) of the base width	Article 11.6.3.3
Abutments 1 and 2 Downstream portion	Gravel	Within the middle two-thirds (2/3) of the base width	Article 11.6.3.3

**Table 5 – Eccentricity Limits**

### 1.3 Earth Pressures and Surcharge Forces

Repurposed abutments should be evaluated for active earth pressure over the abutment height. Coulomb wedge theory applies for gravity and semi-gravity walls. In designing for active pressure, a Coulomb active earth pressure coefficient,  $K_{ac}$ , is recommended. Coulomb Active Earth Pressure coefficients were calculated assuming a level backfill ( $\beta=0^\circ$ ), a soil-to-concrete friction angle ( $\delta$ ) of  $19.5^\circ$  and a  $\theta$  of  $73.7^\circ$  as illustrated in LRFD Figure 3.11.5.3-1. These coefficients are as follows:

- $K_{ac} = 0.414$  (Active Earth Pressure)
- $K_{ach} = 0.336$  (Active Earth Pressure – horizontal component)
- $K_{acv} = 0.242$  (Active Earth Pressure – vertical component)

Refer to LRFD Figure 3.11.5.3-1 and BDG Figure 3-1 for guidance on applying Coulomb Earth Pressure on the abutment back face.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG)) Section 3.6.1) for granular backfill soil properties. The backfill properties are as follows:  $\phi = 32^\circ$ ,  $\gamma = 125$  pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for the abutments if an approach slab is not present. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ( $h_{eq}$ ) taken from Table 6.

Abutment Height (feet)	$h_{eq}$ (feet)
5	4.0
10	3.0
$\geq 20$	2.0

**Table 6 - Equivalent Height of Soil for Estimating Live Load Surcharge Of Abutments Perpendicular to Traffic**