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GEOTECHNICAL
ENVIRONMENTAL
ECOLOGICAL
WATER
CONSTRUCTION MANAGEMENT

707 Sable Oaks Drive
Suite 150
South Portland, ME 04106
T: 207.879.9190
F: 207.536.1173
www.gza.com

MEMORANDUM

To: Laura Krusinski, Maine Department of Transportation

From: Michael Smith, P.E. (MA), Andrew Blaisdell, P.E., Christopher Snow, P.E.

Date: August 26, 2022

File No.: 09.0026141.00

Re: Geotechnical Addendum #1
Armstrong Road Bridge over I-95, Bridge No. 5815
Maine Department of Transportation WIN 23505.00
Waterville, Maine

STATE OF MAINE
Andrew
R.
Blaisdell
10957
LICENSED
PROFESSIONAL ENGINEER
08/26/2022

GZA GeoEnvironmental, Inc. (GZA) has prepared this memorandum summarizing proposed revisions of geotechnical recommendations contained in the Geotechnical Design Report prepared by MaineDOT for the proposed substructure design for the subject project. Our services were completed in accordance with Bridge Program Assignment Letter #4 (dated January 5, 2022) associated with Multi-PIN Project Contract Number 20200603000000000709 between MaineDOT and GZA dated August 19, 2020. This memorandum is subject to the *Limitations* included in **Appendix A**.

BACKGROUND

The overall project consists of replacing the Armstrong Road Bridge over Interstate 95 (I-95) in Waterville. The project consists of a 300-foot long, two-span integral abutment bridge carrying Rice Rips Road over I-95.

Subsurface explorations, geotechnical evaluations and recommendations were completed for the bridge replacement and presented in a Geotechnical Design Report (GDR) prepared by MaineDOT dated October 7, 2020. In response to MaineDOT's request and subsequent discussions between GZA and MaineDOT, GZA completed a peer review of the GDR and completed additional engineering analyses of specific aspects of the proposed foundation design. This addendum presents recommended revisions to the geotechnical design recommendations resulting from our review and analyses. Evaluations and recommendations that are not discussed in this addendum remain unchanged from the October 7, 2020 GDR.

5.0 REVISED SUBSURFACE CONDITIONS

Additional subsurface exploration data were located on the MaineDOT 1963 bridge drawings that will be useful in interpretation of subsurface conditions at the site. Seven test borings, designated as B-23, B-24, B-26, B-27, B-31, B-33 and B-34, were included in the drawings. Approximately 5 feet of bedrock was cored in each of these borings. Encountered bedrock elevations were documented as El. 257.9 to 262.6 near Abutment 1, El. 256.1 to 252.4 near the piers, and El. 247.9 to 248.0 near Abutment 2.

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Based on the range in encountered elevations in the historic borings and MaineDOT's BB-WRRR-100 series borings, we recommend that bedrock elevations of El. 261 and El. 248 be utilized for lateral pile foundation evaluations, as noted below. The historic subsurface data included in the 1963 drawings is attached in **Appendix B**. We understand that MaineDOT updated the Boring Location Plan and Interpretive Subsurface Profile drawings to include these data, which are presented in **Appendix C**.

Groundwater depths were documented as 9 and 15 feet below ground surface in borings BB-WRRR-101 and BB-WRRR-105, respectively, corresponding to El. 269.0 and El. 259.0, based on measurements taken immediately after the borings. However, none of the soil was observed to be wet in any borings, and laboratory water contents are not indicative of saturated conditions. Considering these observations and that the ditches along the highway are at El. 255 to El. 251, we anticipate stabilized groundwater is deeper than the measurements taken in the borings.

7.0 REVISED GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.2 AXIAL PILE RESISTANCE – STRENGTH LIMIT STATE

Abutment No, 1 – West Abutment, Geotechnical Resistance *(replace current section with the following)*: A grouted rock socket detail is proposed that includes a steel plate welded to the tip of the pile and detailed such that grout can be reliably placed below and around the pile tip and promote full, uniform load transfer to end bearing in bedrock. For this condition, the end bearing resistance may be calculated using methodology for a drilled shaft bearing in bedrock. Using the average of values calculated based on AASHTO LRFD equations 10.8.3.5.4C-1 (intact rock) and 10.8.3.5.4C-2 (jointed rock mass), and an unconfined compressive strength of 7,620 psi, a bearing area of 225 square inches (15-inch square plate) provides a nominal geotechnical axial compression resistance of 2,003 kips, resulting in a factored resistance of 1,002 kips.

We also checked the tip resistance of 5,000 psi grout, using the same methodology but assuming an unjointed condition, which is consistent with a uniform grout material. The nominal bearing resistance for sound 5,000-psi grout is 2,813 kips, which is greater than the value calculated above for jointed rock. Therefore, 5,000-psi grout is suitable.

A summary of the calculated factored axial compressive structural and geotechnical resistances of the H-pile sections at strength limit state is provided in Table 4A below:

Table 4A: Factored Axial Compressive Resistances for Strength Limit State - H-Piles with Steel End Plates Installed in Bedrock Sockets at Abutment No. 1

Strength Limit State Factored Axial Pile Resistance – Abutment No. 1			
Pile Section	**Structural Resistance ϕ_c=0.60 (kips)[1]**	**Static Geotechnical Resistance ϕ_{static}=0.50 (kips)[2]**	**Governing Axial Pile Resistance (kips)**
HP 14 x 89	679	1,002	679
HP 14 x 117	899	1,002	899

Notes:
1. *Considers axial loading in compression only; resistance factor per AASHTO LRFD 6.5.4.2*
2. *Static geotechnical resistance is based on drilled shaft tip resistance in rock; resistance factor per AASHTO LRFD Table 10.5.5.2.4-1.*
3. *It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (l) and effective length factors (K) determined from LPile.*

Abutment No. 2 – East Abutment, Geotechnical Resistance *(replace Table 5 with Table 5A below, incorporating the preliminary analysis for D36-32 Hammer only)*:

Table 5A: Factored Axial Compressive Resistances for Strength Limit State - H-Piles End Bearing on Bedrock at Abutment No. 2

Strength Limit State Factored Axial Pile Resistance – Abutment No. 2					
Pile Section	**Modeled Hammer**	**Structural Resistance, Combined Bending (kips)[1]**	**Maximum Geotechnical Resistance (Hard Rock) ϕ_c=0.50 (kips)[2]**	**Drivability Resistance ϕ_{dyn}=0.65 (kips)[3]**	**Governing Axial Pile Resistance (kips)**
HP 14 x 89	D 19 x 42	652	566	409	409
HP 14 x 117	D 36 x 32	860	749	546	546

Notes:
1. *Considers combined axial and compression resistance using K-values in the upper and lower sections of piles based on end conditions and f_c=0.70 for the upper section (undamaged pile) and f_c=0.50 for the lower section (subject to damage due to hard driving); resistance factors per AASHTO LRFD 6.5.4.2.*
2. *Considers axial loading only for pile subject to damage due to hard driving; resistance factor per AASHTO LRFD 6.5.4.2*
3. *Considers driving criteria established by dynamic testing; resistance factor per AASHTO LRFD Table 10.5.5.2.3-1.*
4. *It is the responsibility of the structural engineer to check the nominal axial structural compressive resistance (P_n) based on unbraced lengths (l) and effective length factors (K) determined from LPile against the combined stresses calculated in LPile.*

7.3 AXIAL PILE RESISTANCE – SERVICE AND EXTREME LIMIT STATE

Abutment No, 1 – West Abutment *(replace Table 6 with Table 6A below, updated Geotechnical Resistance)*:

Table 6A: Factored Axial Compressive Resistances for Service and Extreme Limit State - H-Piles with Steel End Plates Installed in Bedrock Sockets at Abutment No. 1

Service and Extreme Limit State Factored Axial Pile Resistance – Abutment No. 1			
Pile Section	**Structural Resistance ϕ=1.0 (kips)**	**Static Geotechnical Resistance ϕ =1.0 (kips)**	**Governing Axial Pile Resistance (kips)**
HP 14 x 89	1,131	2,003	1,131
HP 14 x 117	1,498	2,003	1,498

Abutment No, 2 – East Abutment *(replace Table 7 with Table 7A below, incorporating the preliminary analysis for D36-32 Hammer only)*:

Table 7A: Factored Axial Compressive Resistances for Service and Extreme Limit State – Driven H-Piles End Bearing on Bedrock at Abutment No. 2

Service and Extreme Limit State Factored Axial Pile Resistance – Abutment No. 2					
Pile Section	**Modeled Hammer**	**Structural Resistance ϕ=1.0 (kips)**	**Controlling Geotechnical Resistance (Hard Rock) ϕ=1.0 (kips)**	**Drivability Resistance ϕ_{dyn}=1.0 (kips)**	**Governing Axial Pile Resistance (kips)**
HP 14 x 89	D 19 x 42	1,131	1,131	630	630
HP 14 x 117	D 36 x 32	1,498	1,498	840	840

7.4 LATERAL PILE RESISTANCE/BEHAVIOR

Recommended geotechnical parameters for generation of soil-resistance (p-y) curves in lateral pile analyses are presented in Tables 8A and 9A below. Top of rock elevations provided in the tables represent anticipated short pile lengths which control pile stress (Abutment 1) and fixity (Abutment 2) considerations.

Table 8A: Soil Parameters for Generation of Soil-Resistance (p-y) Curves for Abutment No. 1

LPile Input Parameters Abutment 1, Top of Pile El. = 269							
Soil Parameters							
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**		**γ_e (pcf)**	
Granular Borrow	Reese Sand	274	83	32		125	
Existing Fill	Reese Sand	269*	48	30		120	
Glacial Till (Strong Condition) [3]	Reese Sand	264	209	38		135	
Infill Material (Weak Condition) [3]	Reese Sand	264	83	32		120	
Rock Socket Parameters							
Stratum	**Rock Model**	**Top of Layer Elevation (NAVD88 ft)**	**UCS (ksi)**	**RQD (%)**	**Design Strain Factor, krm**	**Initial Modulus of Rock Mass, Kir (psi)**	**γ_e (pcf)**
Rock Socket	Weak Rock	261	2.0	49	0.000473	15,741	107

Notes:
1. * indicates the top of pile elevation, soil above is include to account for fill soils behind the abutment for expansion evaluations.
2. pci = pounds per cubic inch, deg = degrees, psi =pounds per square inch, γe = effective unit weight, pcf = pounds per square foot, UCS = unconfined compressive strength.
3. The soil parameters for the Glacial Till at El. 264 to 261 should be modeled as given for in-situ fill material and infill material to determine which controls for structural design of the socketed pile.

Table 9A: Soil Parameters for Generation of Soil-Resistance (p-y) Curves for Abutment No. 2

LPile Input Parameters Abutment 2, Top of Pile El. = 266					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci) / E50**	**ϕ' (deg)/ Su (psf)**	**γ_e (pcf)**
Granular Borrow	Reese Sand	271	83	32	125
Existing Fill	Reese Sand	266*	48	30	120
Glacial Till	Reese Sand	255	120	38	73
Top of Rock	--	248	--	--	--

Notes:
1. Pile tip elevation should be assumed to be top of Rock.
2. * indicates the top of pile elevation, soil above is included to account for fill soils behind the abutment for expansion evaluations.
3. pci = pounds per cubic inch, deg = degrees, psi =pounds per square inch, γe = effective unit weight, pcf = pounds per square foot.

CLOSING

We trust that this information meets current project needs. Please contact Andy Blaisdell at 207-358-5117 if you have any questions or required additional information.

MPS/ARB/CLS:erc
P:\09 Jobs\0026100s\09.0026141.00 - MEDOT Armstrong Bridge, Waterville\Report\FINAL 26141.00 Armstrong Waterville Geotech Addendum 08 26 22.docx

Attachments:

- Appendix A – Limitations
- Appendix B – Historic Subsurface Data
- Appendix C – Draft Revised Boring Location Plan and Interpretive Subsurface Profiles showing Historic Borings
- Appendix D – Calculations

APPENDIX A – LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions .

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal

recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

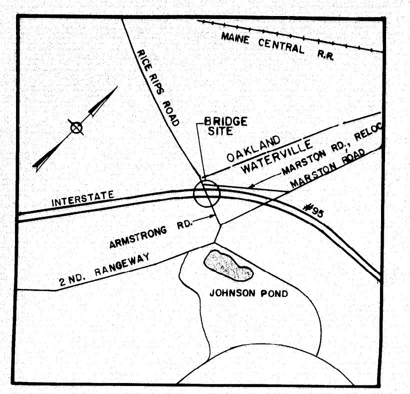
Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

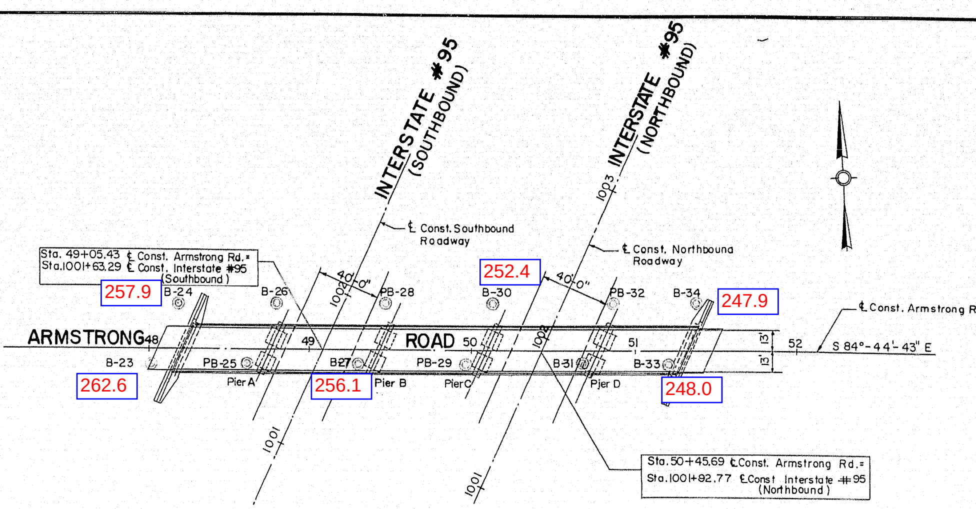
APPENDIX B – HISTORIC SUBSURFACE DATA

B.P.R. REG. NO.	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-6(22)	20	27

WATERVILLE INTERSTATE



LOCATION MAP
Not to scale



KEY PLAN
Scale: 1" = 50'-0"

GENERAL NOTES

FOUNDATION:

Foundations may be altered, if necessary, to suit conditions encountered in construction.

DESIGN:

In accordance with the Specifications of the American Association of State Highway Officials for H20-44 loading (1957 Edition).

Design Stresses: Structural Steel fs = 18,000 psi
Reinforcing Steel fs = 18,000 psi
Concrete (n=10) fc = 1,200 psi

CONSTRUCTION:

State of Maine Standard Specifications to be followed except as noted in Special Provisions

REINFORCEMENT:

All bars shall have deformations conforming to A.S.T.M. Designation A305.
Unless otherwise shown on plans, reinforcing bars shall be lapped 20 diameters to make a splice, except that main reinforcing bars near the top of slabs and beams having more than 12" of concrete under the bars shall be lapped 35 diameters to make a splice.

STRUCTURAL STEEL:

Wherever cover plates and/or shear connectors are welded to beams, beams and plates shall be weldable structural steel A.S.T.M. Designation A373.

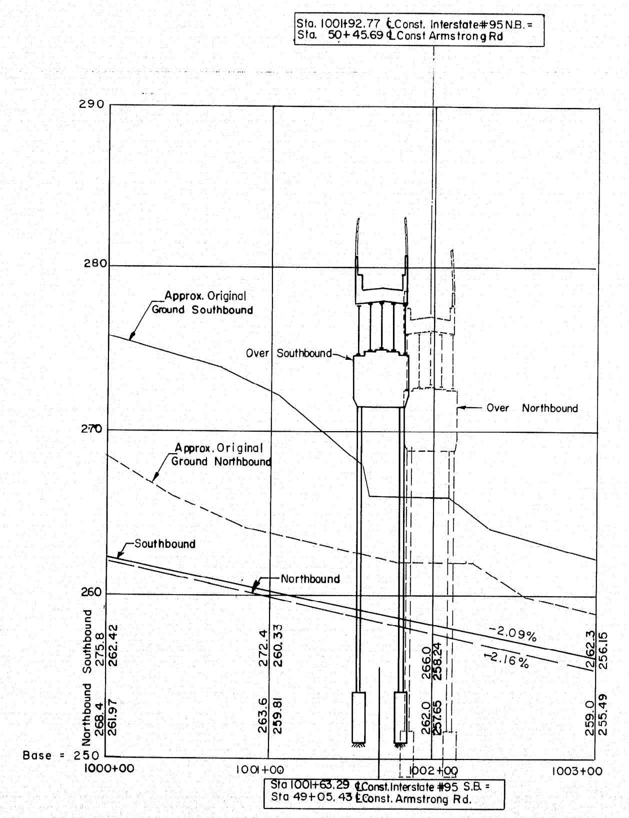
BENCH MARK:

BM-5 R.R. Spike in intersection of ℄'s Armstrong Rd. and Second Rangeway. Elevation 259.20 U.S.G.S. Datum.

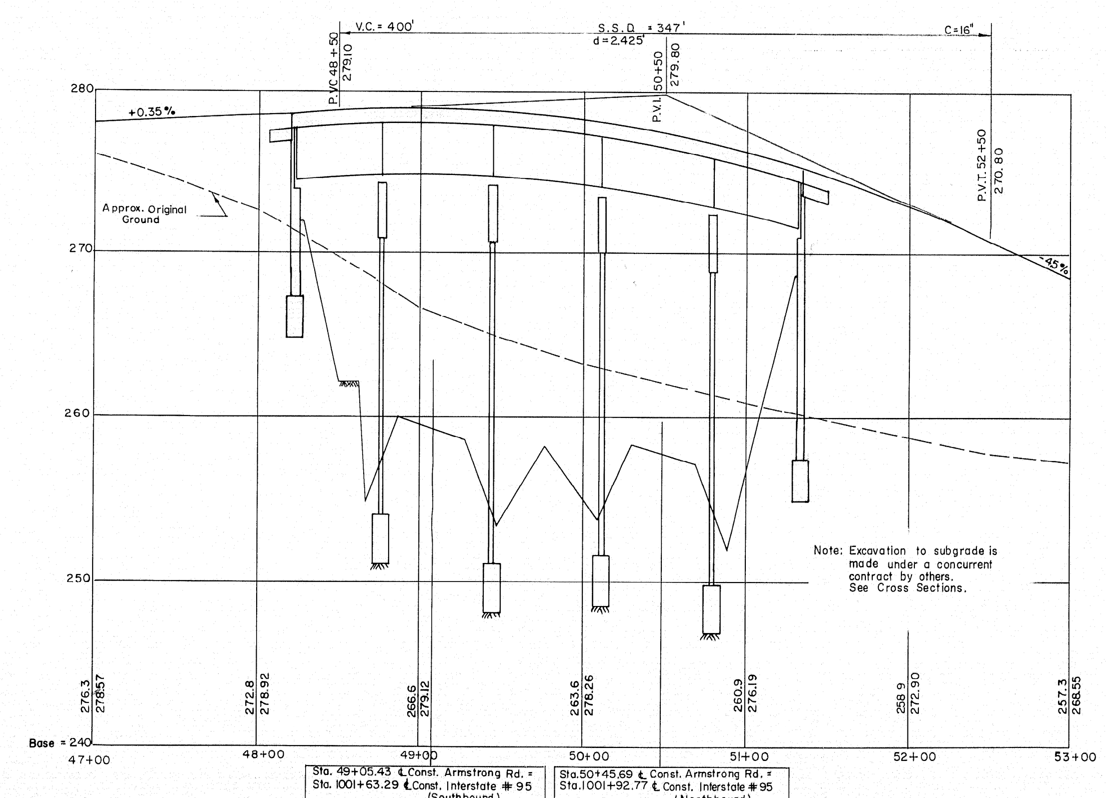
~~ESTIMATED~~ QUANTITIES

~~(Not Guaranteed)~~

STRUCTURAL EARTH EXCAVATION, ABUTMENTS AND RETAINING WALLS	~~330~~	CU. YDS. 548.9
STRUCTURAL EARTH EXCAVATION, PIERS	~~80~~	CU. YDS. 71.6
STRUCTURAL ROCK EXCAVATION, PIERS	~~130~~	CU. YDS. 181.94
GRAVEL BASE COURSE – IN PLACE MEASUREMENT	~~180~~	CU. YDS. 237.4
BITUMINOUS CONCRETE SURFACE COURSE, TYPE "A"	110	TONS
MEMBRANE WATERPROOFING (3 PLY)	~~910~~	SQ. YDS. 900.28
PORTLAND CEMENT CONCRETE, ABUTMENTS AND RETAINING WALLS	~~210~~	CU. YDS. 247.01
PORTLAND CEMENT CONCRETE, PIERS	~~170~~	CU. YDS. 194.46
PORTLAND CEMENT CONCRETE, ROADWAY AND SIDEWALK SLABS ON STEEL BRIDGES	~~310~~	CU. YDS. 327.73
PORTLAND CEMENT	~~1,040~~	BBLS. 1210.1
BRIDGE DRAINAGE	1	LUMP SUM
STRUCTURAL STEEL, FABRICATED AND DELIVERED	~~305,100~~	LBS. 305,846
STRUCTURAL STEEL, ERECTION	~~305,100~~	LBS. 305,846
REINFORCING STEEL, DELIVERED	~~108,300~~	LBS. 103,452
REINFORCING STEEL, PLACING	~~108,300~~	LBS. 103,452
SHEAR CONNECTORS, DELIVERED AND PLACED	1	LUMP SUM
FRENCH DRAINS	~~200~~	CU. YDS. 96.25
ALUMINUM RAILING	~~626~~	LIN. FT. 622.9
SLOPE PAVING FOR BRIDGES	~~370~~	SQ. YDS 474.7



PROFILE ALONG INTERSTATE #95
Scale: Hor. 1" = 50'-0" Vert. 1" = 5'-0"



PROFILE ALONG ARMSTRONG ROAD
Scale: Hor. 1" = 50'-0" Vert. 1" = 5'-0"

DESIGN	CHECK H.P.	BRIDGE NO.
DRAWN D.A.T.	APPROVED WAH-CJM	SURVEY PLOT

STATE HIGHWAY COMMISSION
INTERSTATE #95
UNDER
ARMSTRONG ROAD
IN THE CITY OF
WATERVILLE
KENNEBEC COUNTY
KEY PLAN & PROFILES
SHEET 1 OF 8 AUGUSTA, MAINE

APPROVED BY Wm. A. Henderson DATE 7/23/58

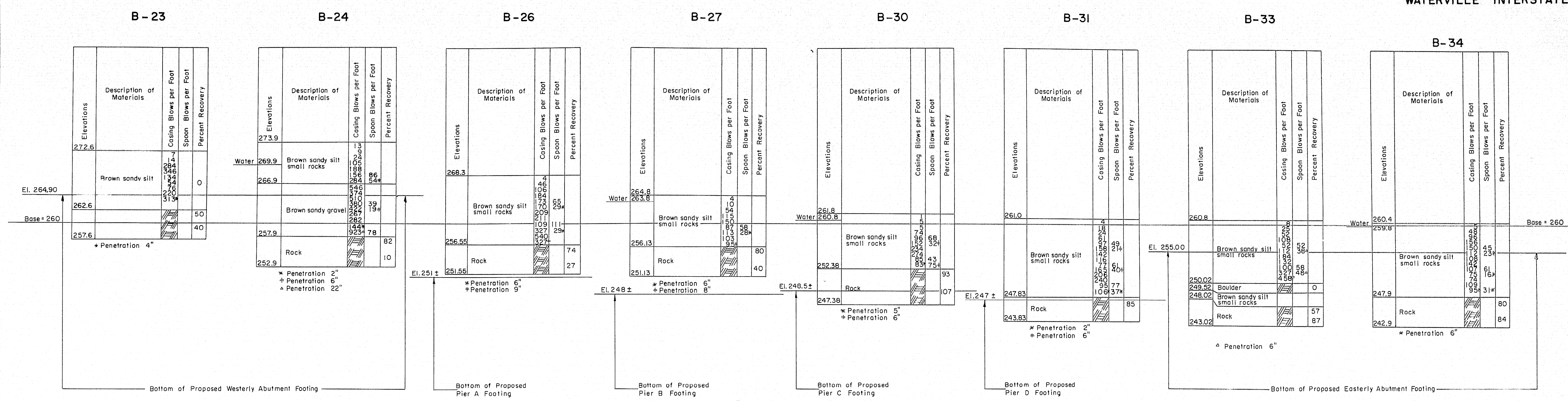
THE CLARKESON ENGINEERING CO., INC.
CONSULTING ENGINEERS
BOSTON MASSACHUSETTS

73-62

0 1 2 3 4 5 INCHES

B.P.R. REG. NO.	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS
1	MAINE	I-95-6(22)	21	27

WATERVILLE INTERSTATE



BORING LOG

Scale: 1/8" = 1'-0"

NOTES:

1. Location of borings are shown on Key Plan thus ● B-23.
   Punchings are shown on Key Plan thus ● PB-25.
2. Borings and punchings are taken for the purpose of design and show condition at boring and punching points only, but do not necessarily show nature of materials to be encountered during construction.
3. The Contractor is to form his own opinion of the character of the materials, and to make his own interpretation of borings and punchings.
4. The Engineer does not warrant the findings as being accurate or complete.
5. Figures in boring columns indicate blows per foot on 2 1/2" casing or 1" spoon produced by a 356 # hammer with a fall of 14" and 12" respectively.
6. Punchings made by 1" rod and a 60 # hammer with an 18" drop.
7. Borings and punchings were taken by the Maine State Highway Commission during the months of March and April, 1958.

PUNCHING LOG

Punching No.	Ground Elevation	Refusal @
PB-25	270.4	8.4'
PB-28	263.4	12'
PB-29	261.8	10'
PB-32	261.2	5.5'

THE CLARKESON ENGINEERING CO., INC.

DESIGN	CHECK H.P.	BRIDGE NO.
DRAWN V.S.	APPROVED W.A.H. C.J.M.	SURVEY PLOT

STATE HIGHWAY COMMISSION

INTERSTATE #95

UNDER

ARMSTRONG ROAD

IN THE CITY OF

WATERVILLE

KENNEBEC COUNTY

BORING DATA

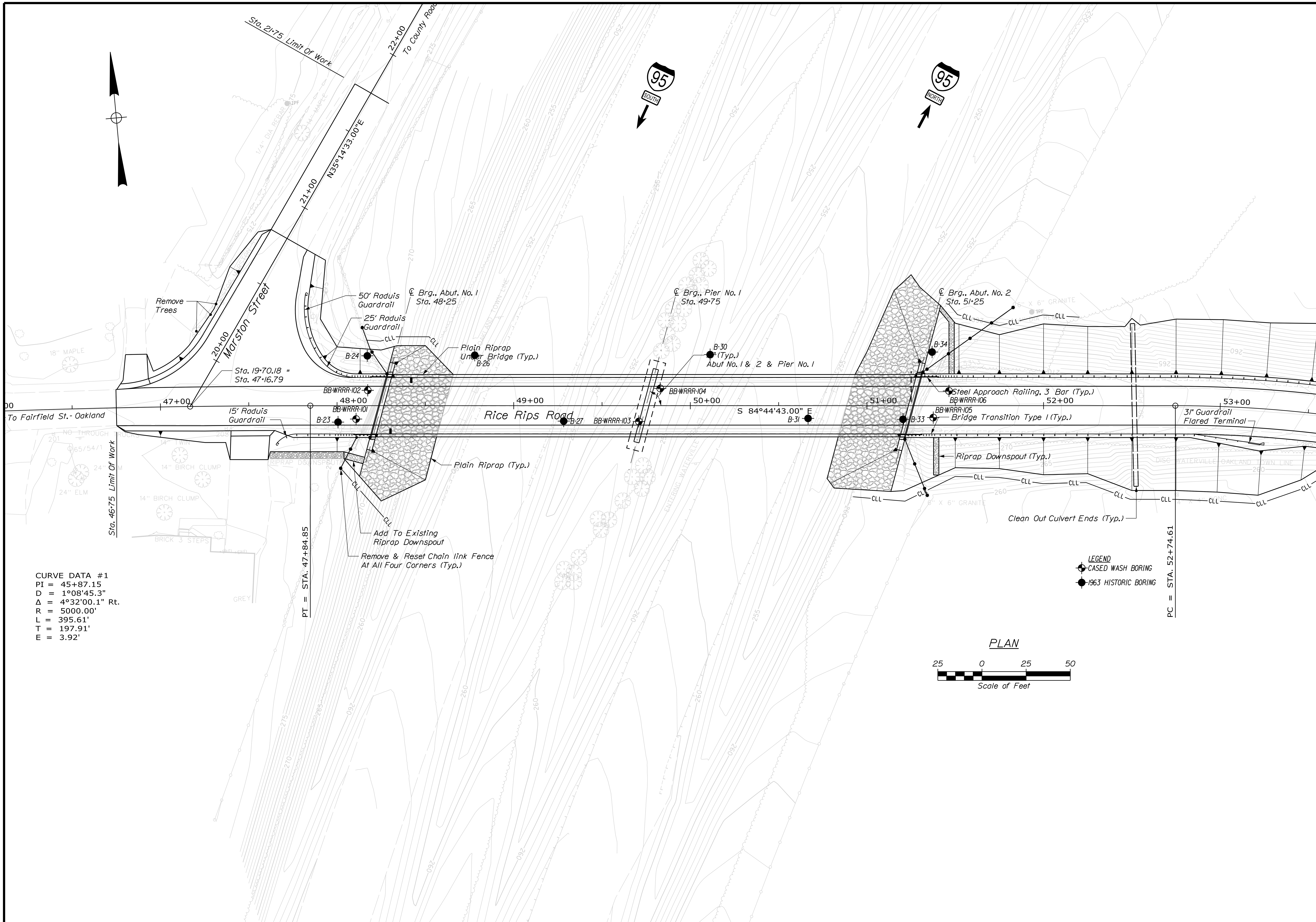
SHEET 2 OF 8 AUGUSTA, MAINE

73-63

0 1 2 3 4 5 INCHES

APPENDIX C – DRAFT REVISED BORING LOCATION PLAN AND INTERPRETIVE SUBSURFACE PROFILES SHOWING HISTORIC BORINGS

Filename: ...\MSTA\006_BLP1 HISTORIC .dgn   Division: GEOTECH   Username: terry.white   Date:8/2/2022



CURVE DATA #1
PI = 45+87.15
D = 1°08'45.3"
Δ = 4°32'00.1" Rt.
R = 5000.00'
L = 395.61'
T = 197.91'
E = 3.92'

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2350500

WIN 23505.00

BRIDGE NO. 5815

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	D. Eaton	
DESIGN-DETAILED	E. Brewer	
CHECKED-REVIEWED	M.R.P.	3/17/22
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

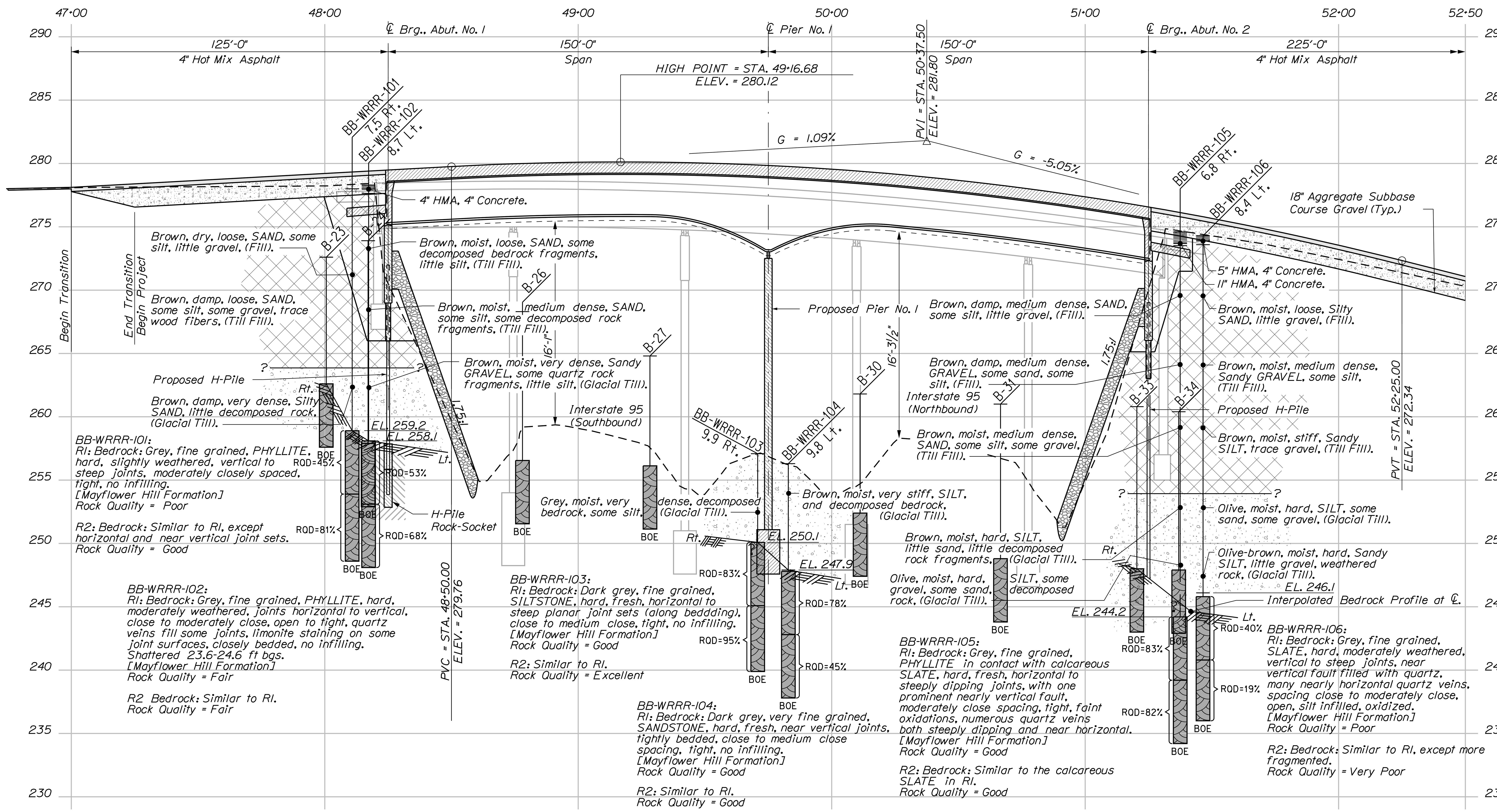
SIGNATURE
P.E. NUMBER
DATE

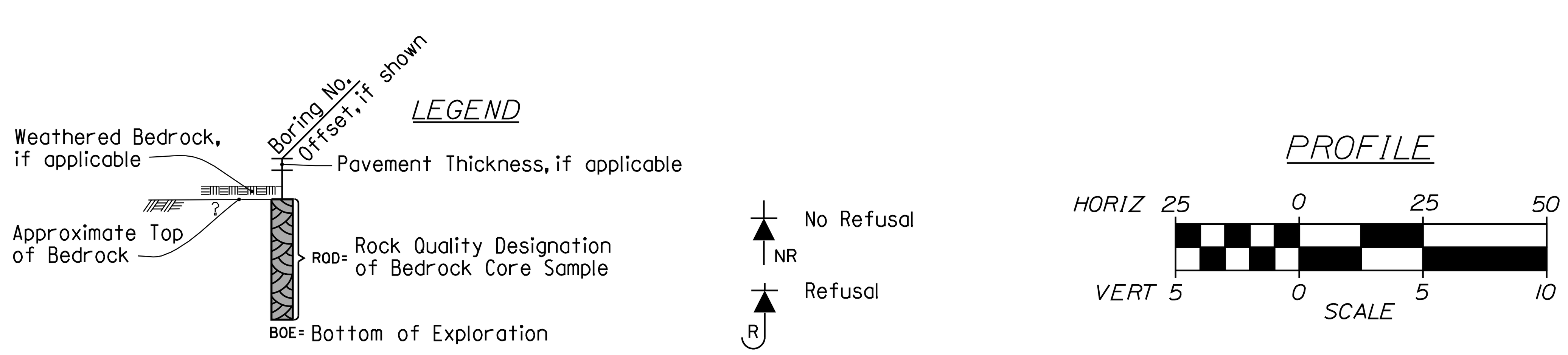
RICE RIPS ROAD
OVER INTERSTATE - 95
WATERVILLE KENNEBEC COUNTY

BORING LOCATION PLAN

SHEET NUMBER
6
OF 27

Filename: ...\MSTA\007_ISP1 HISTORIC.dgn Division: GEOTECH Username: terry.white Date:8/2/2022





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

2350500

BRIDGE NO. 5815

WIN 23505.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Eaton		
DESIGN-DETAILED	Kendra N.	M.R.P.	Mar. 2019
CHECKED-REVIEWED	L.KRUSINSKI	T.WHITE	SEP. 2020
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

ARMSTRONG ROAD BRIDGE
OVER INTERSTATE - 95
WATERVILLE KENNEBEC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
7
OF 27

APPENDIX D – CALCULATIONS

Socketed Pile Design Parameters in Bedrock

Project:	Armstrong Road Bridge #5815	**Project No.:**	09.0026141.00
Location:	Waterville, ME		
Calculated By:	B. Cardali	**Date:**	8/1/2022
Checked By:	A. Blaisdell	**Date:**	8/2/2022

Objective: Develop foundation design parameters for socketed piles end bearing in bedrock using AASHTO methodology .

Inputs:

1. Rock core data from BB-WRRR-100 series test borings, including rock type and RQD, and laboratory test data including unit weight, unconfined compressive strength were assessed as follows:

a. Rock was primarily classified as Phyllite at the proposed socketed pile abutment, Abutment 1.

b. The design RQD was selected as 49%.

c. Design Unconfined Compressive Strength and secant modulus were taken as average of three laboratory test results.

d. Poisson's ratio was taken a value from laboratory test result with strain measurements.

Approach:

1. Evaluate Geologic Strength Index (GSI) in accordance with AASHTO methodology. See page 2.
2. Develop LPILE inputs for Weak Rock model based on bedrock structure characterization and laboratory test results. See red text on page 2.
3. Calculate tip resistance of drilled shaft socket in rock for socketed piles using AASHTO Eq. 10.8.3.5.4c-1 (intact or tightly jointed rock) and Eq. 10.8.3.5.4c-2 (random joint orientation). Considering generally Fair to Good rock quality and no infilling to minor silt infilling, use 2 times the resistance calculated using 10.8.3.5.4c-2 for the design unit tip resistance, which is still less than half of the intact value from 10.8.3.5.4c-1.

4. Calculate tip resistance using end area of plate welded to bottom of pile. Use resistance factor from AASHTO Table 10.5.5.2.4-1.
5. Check tip resistance of drilled shaft socket in rock for socketed piles using AASHTO Eq. 10.8.3.5.4C-2 using end area of plate welded to bottom of pile for direct bearing on grout, using unjointed rock equation and design grout unconfined compressive strength of 5 ksi.

Conclusion:

1. The nominal tip resistance for an H-pile with a 15" square plate is approximately 1,609 kips, resulting in a factored tip resistance of 805 kips, for the assumed bedrock conditions.
2. The factored tip resistance for end bearing on sound 5,000 psi grout is 1,406 kips. Therefore, the tip resistance in jointed rock controls.

Armstrong Road Bridge #5815
Waterville, ME
09.0026141.00

Calculated by B.Cardali Date: 8/1/2022
Checked by A.Blaisdell Date: 8/2/2022

Calculation: Rock Socket Lateral Parameters

References: 1) AASHTO LRFD Bridge Design Specifications, 9th Ed. (2020)

Parameter Description	**Parameter Symbol**	**Design Values**	**Reference**
Rock Type		Phyllite	Rock ID per boring logs/geologic mapping
Comparable Rock Type/Texture		Phyllite	AASHTO Table 10.4.6.4-1
Unconfined compressive strength of intact rock	q_u (psi)	7,620	Average of samples
Elastic Secant Modulus of Intact Rock	E_i (ksi)	8,053	Average of samples
Poisson's Ratio of Intact Rock	ν	0.22	From BB-WRRR-101, other samples did not have measured lateral strain.
LPILE Input Unconfined compressive strength	**$q_{u\text{-LPILE}}$ (psi)**	**2,000**	Limit UCS to 2,000 psi for LPILE
Calculated Strain Factor	krm,c	0.000473	Compression Strain at 50% of UCS
LPile Design Strain Factor	**krm**	**0.000473**	Use krm closest to lab value
Rock Total Unit Weight	γ' (psf)	169	Average value from lab UCS tests
Rock Effective Unit Weight	**γ' (psf)**	**106.6**	
Geological Strength Index	GSI	60	AASHTO FIG 10.4.6.4-1
Hoek - Brown 2002	D	0	Hoek - Brown 2002
Rock Quality Designation	**RQD (%)**	**49**	Average value, BB-WWWR-101 and -102, upper 5' core run.
Empirically determined rock mass parameter	s	0.011749	AASHTO Eq. 10.4.6.4-2
Empirically determined rock mass parameter	a	0.503	AASHTO Eq. 10.4.6.4-3
Rock group constant	m_i	7	AASHTO Table 10.4.6.4-1
Empirically determined rock mass parameter	m_b	1.68	AASHTO Eq. 10.4.6.4-4
Equivalent Rock Mass Modulus, Hoek & Brown	E_m1 (ksi)	1,869	AASHTO Table 10.4.6.5-1 - eq. from table
Equivalent Rock Mass Modulus, Yang	E_m2 (ksi)	1,279	AASHTO Table 10.4.6.5-1 - eq. from table
Design Basis Equivalent Rock Modulus	E_m (ksi)	1,574	
Initial Modulus of Rock Mass, Lpile K_{ir} Value	**K_{ir} (psi)**	**15,741**	Calculated as 1/100th of the calculated Design basis equivalent Rock Modulus, based on the KSDOT research.

Notes:
1. Yellow cell is user input, white is calculated.
2. Red text are input values for LPILE Weak Rock model.
3. A reduced modulus for use in LPILE was based on research by Kansas Department of Transportation (KSDOT) presented in a paper entitled " Lateral Capacity of Rock Sockets in Limestone Under Cyclic and Repeated Loading" dated August 2010.

Development of Parameters from Rock Laboratory Testing

		Laboratory Tests			
Parameter Description	**Parameter Symbol**	BB-WRRR-101, R1, 20-20.38	BB-WRRR-104, R1, 9.29-9.67	BB-WRRR-105, R1, 30.43-30.81	**Average**
Unconfined compressive strength of intact rock	q_u (psi)	10,577	5,236	7,047	7,620
Elastic Secant Modulus of Intact Rock	E_i (ksi)	7,650	5,810	10,700	8,053
Poisson's Ratio of Intact Rock	ν	0.22	--	--	0.22

Armstrong Road Bridge #5815
Waterville, ME
09.0026141.00

Calculated by B. Cardali Date: 8/1/2022
Checked by A. Blaisdell Date: 8/2/2022

Calculation: Drilled Shaft Tip Resistance in Rock

Note: Values imported from Bedrock Design Parameters

References: 1) AASHTO LRFD Bridge Design Specifications, 9th Ed. (2020)

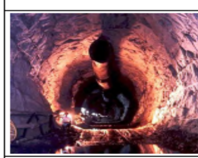
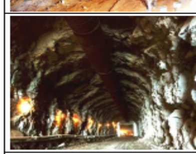
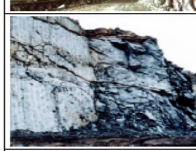
Parameter Description	**Parameter Symbol**	**Bedrock Values**	**Grout Values**	**Reference**
Pile Tip Size	*B (in)*	*15*	*15*	
Pile Tip Area	*At (in^2)*	*225*	*225*	
Unconfined compressive strength of intact rock	q_u (psi)	7,620	5,000	Lab data for rock. Recommended design UCS for grout.
Nominal Unit Tip Resistance, Intact Rock	$q_{p\ intact}$ (ksf)	2,743	1,800	AASHTO Eq. 10.8.3.5.4C-1
Geological Strength Index	GSI	60	--	AASHTO Fig. 10.4.6.4-1
Hoek - Brown 2002	D	0	--	Hoek - Brown 2002
Empirically determined rock mass parameter	s	0.012	--	AASHTO Eq. 10.4.6.4-2
Empirically determined rock mass parameter	a	0.503	--	AASHTO Eq. 10.4.6.4-3
Rock group constant	m_i	7	--	AASHTO Table 10.4.6.4-1
Empirically determined rock mass parameter	m_b	1.7	--	AASHTO Eq. 10.4.6.4-4
Vertical effective stress at the socket bearing elevation	$\sigma'_{v,b}$ (psf)	1944	--	Vertical effective stress
	$\sigma'_{v,b}$ (psi)	13.5	--	
Fracturing coefficient	A	927	--	AASHTO Eq. 10.8.3.5.4C-3 (Turner and Ramey, 2010)
Nominal Unit Tip Resistance, Jointed Rock Mass	$q_{p\ jointed}$ (ksf)	641	--	AASHTO Eq. 10.8.3.5.4C-2
Design Nominal Unit Tip Resistance	*$q_{p\ design}$ (ksf)*	*1,282*	*1,800*	*Use 2*Jointed value (less than 1/2 of intact) for rock based on primarily Fair quality*
Nominal Tip Resistance	***$R_{p,i}$ (kips)***	***2,003***	***2,813***	
Resistance Factor	ϕ_{qp}	0.5	0.5	AASHTO TABLE 10.5.5.2.5-1
Factored Tip Resistance	***$R_{R,i}$ (kips)***	***1,002***	***1,406***	AASHTO Eq. 10.8.3.5-1

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

Table 1: Guidelines for estimating disturbance factor D

From Hoek et al., 2002

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$\boxed{D = 0}$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

No Blasting, No Disturbance

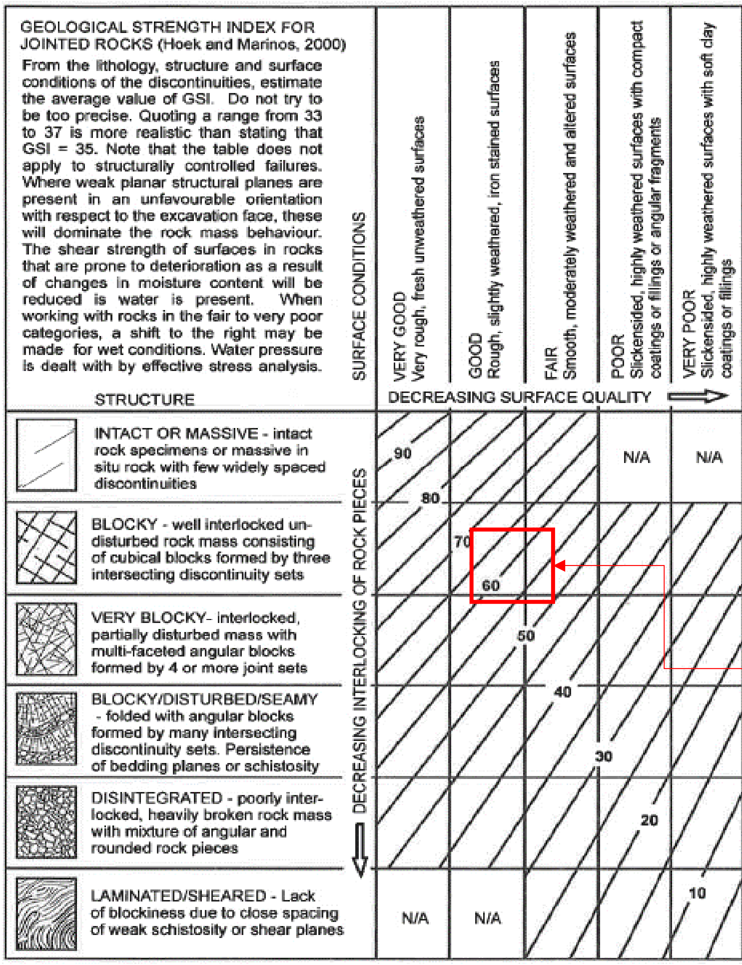


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

Best Estimate
Structure: Blocky
Surface Conditions: Fair to Good
Select GSI=60

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Phyllite,
use mean value

LPile Input Parameters
MaineDOT - Armstrong Bridge
Waterville, Maine

GZA FILE NO.	09.0026141.00
CALCULATED BY	M. Jarrar 8/1/2022
CHECKED BY	M. Smith 8/2/2022

Objective: To estimate the horizontal modulus of subgrade reaction (k) or E50 of subsurface strata for use in lateral analyses. K values are estimated using strata internal friction angles (ϕ') or shear strength.

Methods Correlations between the horizontal modulus of subgrade reaction and the soil internal friction angle of a given stratum are based on Figure 3-34 presented in the 2019 LPile Technical Manual.

Given Information: SPT measurements and subsurface conditions in borings BB-WRRR-100 series borings and bedrock elevations from original bridge borings B-23, B-33 and B-34.

Abutment 1, Top of Pile El. = 269

Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Granular Borrow	Reese Sand	274	83	32	125
Existing Fill	Reese Sand	269	48	30	120
Glacial Till (Strong Condition)	Reese Sand	264	209	38	135
Infill Material (Weak Condition)	Reese Sand	264	83	32	120
Rock Socket	Weak Rock	261	--	--	107

Notes: 1. Weak Rock Model Parameters: Unconfined Compressive Strength - 2 ksi, RQD=49%, krm=0.0005, Kir= 15,700 psi. See socketed pile design parameter calculation.

Abutment 2, Top of Pile El. = 266

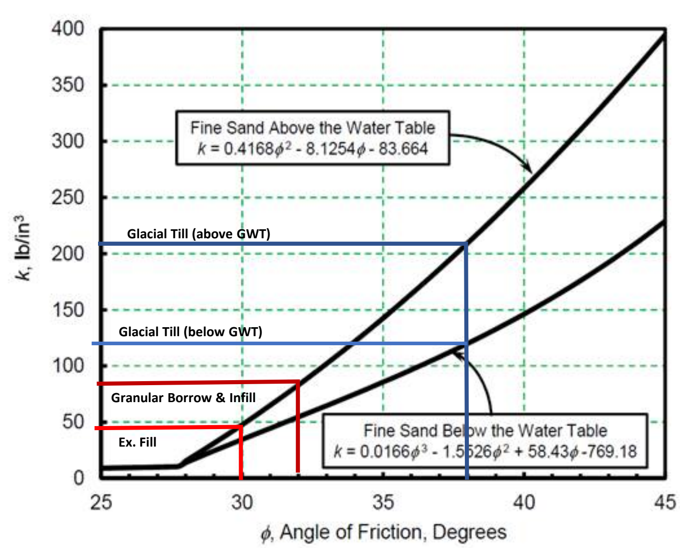
Stratum	Soil Model	Top of Layer Elevation (NAVD88 ft)	k (pci) / E50	ϕ' (deg)/ Su (psf)	γ_e (pcf)
Granular Borrow	Reese Sand	271	83	32	125
Existing Fill	Reese Sand	266	48	30	120
Glacial Till**	Reese Sand	255	120	38	73
Top of Rock	--	248	--	--	--

Notes: 1. Pile tip elevation should be assumed to be top of Rock.

2. * indicates the top of pile elevation, soil above include to account for fill soils behind the abutment for expansion evaluations.

3. ** indicates the top of layer is the approximate ground water elevation based on the boring logs.

4. pci = pounds per cubic inch, deg = degrees, psi =pounds per square inch, γe = effective unit weight, pcf = pounds per square foot.



Correlation of SPT-N Values to ϕ Worksheet

Project:	09.0026141.00 - MEDOT Armstrong Bridge		
Location:	Waterville, Maine		
Calculated By:	MFJ	**Date:**	8/1/2022
Checked By:	MPS	**Date:**	8/2/2022

Purpose: Estimate ϕ value for granular soils based on data from the test borings using the Peck, Hanson, and Thornburn correlation(attached). Correlations are made using N_{field} and $(N1)_{60}$

References:
1) Peck, Hanson, and Thornburn; "Foundation Engineering" 2nd ed., Wiley, New York, 1974
2) M.Carter and S.P.Bentley (1991), Correlations of soil properties, Pentech Press Publishers, London, UK
3) Hatanaka, M., Uchida, A. (1996). Empirical correlation between penetration resistance and effective friction of sandy soil. Soils & Foundations, Vol. 36 (4), 1-9, Japanese Geotechnical Society.

Instructions:
- Create separate tab for each boring, add/delete rows to accommodate boring depth
- Edit "Strata" column on right side of sheet to correspond with boring log, denote granular strata with S1, S2….SN
- Input CE value in cell B6 from table on right side of sheet, CB and CS values correspond to borehole diameters and sampler configurations, and are not likely to change
- Input groundwater level in cell B9. Use bottom of borehole if none encountered.
- User input required in columns A, C, J, and P. Also, ground surface elevation (if known) in cell C11
- Copy cells K12:O12 and paste at each sample depth.
- Update/edit formulas at bottom of sheet to average f values for each strata, transfer value(s) to "Summary" sheet

Assumptions: 1) The empirical relationships are applicable for N values less than or equal to 70. N = 70 was used for SPT N_field values equal or greater than 70.
2) Cohesive soils (i.e. Silty Clay, Peat, etc.) are omitted from this calculation. Assume phi=0 for these soils.

3) Unit weights arere assumed for each stratum for the purpose of estimating the effective stress to be used in the empirical friction angle correlations.

Stratum	Assumed Unit Weight (pcf)
Existing Fill	120
Glacial Till	135

Results: Recommended values of friction angle for design are provided below for the soil strata encountered at each site, based on the empirical correlations shown on the attached calculations for each site.

(SPT-Based)

Abutment	Strata	Recommended γ (pcf)	Recommended ϕ	
Abutment No.1 (BB-WRRR-101 & BB-WBRRR102)	Existing Fill	120	30	°
	Glacial Till	135	38	°
Abutment No.2 (BB-WRRR-105 & BB-WBRRR106)	Existing Fill	120	30	°
	Glacial Till	135	38	°

Empirical Correlations of SPT N-Value and Internal Frictional Angle
09.0026141.00 - MEDOT Armstrong Bridge - Waterville, Maine

Boring ID: **BB-WRRR-101**

CE	0.93	(Auto - efficiency per MaineDOT boring log)
CB	1.0	(borehole diameter 60-115mm)
CS	1.0	(sampler without liner)
gw level (ft):	9	(below ground surface)

Depth (feet)	Elev in ft (NAVD 88)	Unit Weight (pcf)	Stress Increment (psf)	Total Stress (psf)	Pore Water Pressure (psf)	Effective Stress σv' (tsf)	Cn	Cr	N_{field} (blows/ft)	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Carter & Bentley (1991) Φ' based on N_{field} < 70	PH&T 1974 Φ' based on Min. Value of N_{60} < 73	Estimated Fines Content (%)	Strata	Depth for C_R (meters)
0.0	278	120	0	0	0	0.0000	-	0.75								0.00
6.0	272.0	120	720	720	0	0.3600	1.67	0.75	6	4	7	28.8	28.0		S1: Fill	1.83
11.0	267.0	120	600	1320	124.8	0.5976	1.29	0.80	4	3	4	28.0	28.0			3.35
16.0	262.0	135	675	1995	436.8	0.7791	1.13	0.85	100	79	89	44.0	44.3		S2: GT	4.88
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00

	Carter & Bentley (1991)	PH&T 1974
S1 Average ϕ' =	**28.4**	**28.0**
S2 Average ϕ' =	**44.0**	**44.3**

Notes:

100 number requires manual input

25 groundwater table (if not measured, use bottom of boring depth instead)

Empirical Correlations of SPT N-Value and Internal Frictional Angle
09.0026141.00 - MEDOT Armstrong Bridge - Waterville, Maine

Boring ID: **BB-WRRR-102**

CE	0.93	(Auto - efficiency per MaineDOT boring log)
CB	1.0	(borehole diameter 60-115mm)
CS	1.0	(sampler without liner)
gw level (ft):	9	(below ground surface)

Depth (feet)	Elev in ft (NAVD 88)	Unit Weight (pcf)	Stress Increment (psf)	Total Stress (psf)	Pore Water Pressure (psf)	Effective Stress σv' (tsf)	Cn	Cr	N_{field} (blows/ft)	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Carter & Bentley (1991) Φ' based on N_{field} < 70	PH&T 1974 Φ' based on Min. Value of N_{60} < 73	Estimated Fines Content (%)	Strata	Depth for C_R (meters)
0.0	278.3	120	0	0	0	0.0000	-	0.75								0.00
6.0	272.3	120	720	720	0	0.3600	1.67	0.75	6	4	7	28.8	28.0		S1: Fill	1.83
11.0	267.3	120	600	1320	124.8	0.5976	1.29	0.80	10	7	10	30.0	29.3			3.35
16.0	262.3	135	675	1995	436.8	0.7791	1.13	0.85	85	67	76	44.0	43.8		S2: GT	4.88
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00

	Carter & Bentley	PH&T 1974
S1 Average ϕ' =	29.4	28.6
S2 Average ϕ' =	44.0	43.8

Notes:

100 number requires manual input

25 groundwater table (if not measured, use bottom of boring depth instead)

Empirical Correlations of SPT N-Value and Internal Frictional Angle
09.0026141.00 - MEDOT Armstrong Bridge - Waterville, Maine

Boring ID: **BB-WRRR-105**

CE	0.93	(Auto - efficiency per MaineDOT boring log)
CB	1.0	(borehole diameter 60-115mm)
CS	1.0	(sampler without liner)
gw level (ft):	9	(below ground surface)

Depth (feet)	Elev in ft (NAVD 88)	Unit Weight (pcf)	Stress Increment (psf)	Total Stress (psf)	Pore Water Pressure (psf)	Effective Stress σv' (tsf)	Cn	Cr	N_{field} (blows/ft)	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Carter & Bentley (1991) Φ' based on $N_{field} < 70$	PH&T 1974 Φ' based on Min. Value of $N_{60} < 73$	Estimated Fines Content (%)	Strata	Depth for C_R (meters)
0.0	274.6	120	0	0	0	0.0000	-	0.75								0.00
6.0	268.6	120	720	720	0	0.3600	1.67	0.75	13	9	15	31.0	29.8		S1: Fill	1.83
11.0	263.6	120	600	1320	124.8	0.5976	1.29	0.80	12	9	12	30.5	29.8			3.35
16.0	258.6	120	600	1920	436.8	0.7416	1.16	0.85	10	8	9	30.0	29.5			4.88
21.0	253.6	135	675	2595	748.8	0.9231	1.04	0.95	42	37	39	39.3	37.8		S2: Glacial Till	6.40
26.0	248.6	135	675	3270	1060.8	1.1046	0.95	0.95	41	36	34	39.0	37.5			7.92
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00
																0.00

	Carter & Bentley	PH&T 1974
S1 Average ϕ' =	**30.5**	**29.7**
S2 Average ϕ' =	**39.1**	**37.6**

Notes:

100 number requires manual input

25 groundwater table (if not measured, use bottom of boring depth instead)

Empirical Correlations of SPT N-Value and Internal Frictional Angle
09.0026141.00 - MEDOT Armstrong Bridge - Waterville, Maine

Boring ID: **BB-WRRR-106**

CE	0.93	(Auto - efficiency per MaineDOT boring log)
CB	1.0	(borehole diameter 60-115mm)
CS	1.0	(sampler without liner)
gw level (ft):	9	(below ground surface)

Depth (feet)	Elev in ft (NAVD 88)	Unit Weight (pcf)	Stress Increment (psf)	Total Stress (psf)	Pore Water Pressure (psf)	Effective Stress σv' (tsf)	Cn	Cr	N_{field} (blows/ft)	N_{60} (blows/ft)	$(N_1)_{60}$ (blows/ft)	Carter & Bentley (1991) Φ' based on N_{field} < 70	PH&T 1974 Φ' based on Min. Value of N_{60} < 73	Estimated Fines Content (%)
0.0	274.3	120	0	0	0	0.0000	-	0.75						
6.0	268.3	120	720	720	0	0.3600	1.67	0.75	6	4	7	28.8	28.0	
11.0	263.3	120	600	1320	124.8	0.5976	1.29	0.80	13	10	12	31.0	30.0	
16.0	258.3	120	600	1920	436.8	0.7416	1.16	0.85	10	8	9	30.0	29.5	
21.0	253.3	135	675	2595	748.8	0.9231	1.04	0.95	42	37	39	39.3	37.8	
26.0	248.3	135	675	3270	1060.8	1.1046	0.95	0.95	48	42	40	40.3	39.3	
											S1 Average ϕ' =	29.9	29.2	
											S2 Average ϕ' =	39.8	38.5	

Strata	Depth for C_R (meters)
	0.00
S1: Fill	1.83
	3.35
	4.88
S2: Glacial Till	6.40
	7.92
	0.00
	0.00
	0.00
	0.00
	0.00
	0.00
	0.00
	0.00
	0.00

Notes:

100 number requires manual input

25 groundwater table (if not measured, use bottom of boring depth instead)