Bridge Rating of Atwater Avenue Culvert (M-02-021)

Atwater Avenue over Saw Mill Brook

Manchester-by-the-Sea, Massachusetts

Prepared for: Cell Signaling Technology

Behalf of: 3 Trask Lane

Danvers, MA 01923

Prepared by: **TEC, Inc.**

282 Merrimack Street

2nd Floor

Lawrence, MA 01843



TABLE OF CONTENTS

TABLE OF CONTENTS
LOAD RATING SUMMARY AND RESULTS
APPENDICES
 Appendix A – Design Calculations I. Structural Steel Plate Arch Culvert Rating, thirteen (13) page(s).
II. Structural Steel Plate Arch Culvert Rating with 50% Section Loss, thirteen (13) page(s). page(s).
III. Structural Steel Plate Arch Culvert Rating with 57% Section Loss, thirteen (13) page(s).
 Appendix B – Design References
 Proposed Bridge Plans Service Road Sation 14+72.0 Over Saw Mill Brook, dated May 1952, one (1) page(s).
II. MassDOT Structures Inspection Field Report dated October 1, 2024, nine (9) page(s).
III. Test Boring Log by GZA GeoEnvironmental Inc., dated September 3, 2024, four (4) page(s).
IV. AASHTO LRFD Bridge Design Specification, 9th Edition, 2020, eighteen (18) page(s).
V. MassDOT 2020 LRFD Bridge Manual 100th Anniversary Edition, six (6) page(s).
VI. The Manual for Bridge Evaluation, Third Edition with 2019 Revisions, four (4) page(s).

LOAD RATING SUMMARY AND RESULTS

Cell Signaling Technology Inc. has retained TEC, Inc. to perform calculations to determine the load carrying capacity of the existing corrugated steel plate arch culvert and its ability to support various certified legal loads. Load ratings are expressed as a Rating Factor (RF), or as tonnage for a particular vehicle. A RF that is greater than 1.0 demonstrates a bridge's ability to safely carry the given load. Calculations were completed considering the as-inspected levels of deficiency from the latest culvert inspection performed by MassDOT, dated 10/1/2024. Additional calculations were completed considering 50% section loss of the steel culvert thickness, and lastly, calculations were performed to determine the greatest amount of section loss to the steel culvert thickness before receiving a failing rating, which would lead to a load posting.

The culvert is comprised of corrugated galvanized steel arch plates that are anchored to concrete abutments, spans 15'-0" and is 57'-8" in length. The existing culvert is owned and maintained by the town of Manchester-by-the-Sea and is routinely inspected by MassDOT every two years. The culvert was constructed in 1952 and then the galvanized steel arch was replaced in 2006. In the most recent MassDOT inspection report (10/1/24), the culvert condition was rated as a five (5). A five (5) rating is described as Fair, with moderate to major deteriorations and in a metal culvert, significant corrosion. The corrugated steel arch has an original thickness of 7/32". It was noted that the bottom of both sides of the culvert have minor laminar rust with minor section loss and pitting that is up to 1/16" deep along the entire length of the culvert. TEC, Inc. conservatively rated the culvert by assuming that the section loss was consistent throughout the bottom of both sides of the entire structure. The rating is also based on available plans, inspection reports by MassDOT, sample rating reports for similar structure types.

For the as-inspected condition, all bridge elements rate above statutory levels for H20, Type 3, Type3S2, SU4, SU5, SU6, SU7, EV2, and EV3 truck loadings. The loading diagrams for each vehicle configuration are provided in Appendix B. The bridge element controlling the bridge inventory and bridge operation rating in the culvert's existing condition is the Arch Seam Strength.

TEC, Inc. completed an additional rating considering a 50% loss to the original steel thickness. Additionally, TEC performed calculations to determine the amount of section loss that the culvert can endure before a load posting would be recommended. It was determined that there could be a 57% loss to the original steel thickness (corresponding to approximately 3/32" remaining) before RF's were less than 1.0. These additional calculations demonstrate that continued section loss can occur (assuming no preventative maintenance), and the culvert still holds ample reserve capacity.

The results of all three load rating scenario calculations have been presented on the sheets below. In summary, the culvert in its existing condition has ample reserve capacity to safely handle all certified legal loads. Without any maintenance of preservation activities, TEC has determined that the bridge can continue to safely handle all certified legal loads with up to 57% section loss to the original steel thickness. TEC, Inc. recommends routine inspection and maintenance tasks to the entire culvert on a regular basis to ensure the current condition of the culvert.

Rating for Existing Culvert Conditions

					Inve	ntory LRI	FR Rating	Factors				
Br	ridge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
	Axial	2.93	3.41	3.03	4.62	4.95	4.62	4.62	4.62	4.62	2.95	3.08
CMP	Buckling	2.93	3.41	3.03	4.62	4.95	4.62	4.62	4.62	4.62	2.95	3.08
_	Seam Strength	2.91	3.38	3.00	4.58	4.91	4.58	4.58	4.58	4.58	2.92	3.06

					Oper	ating LR	FR Rating	g Factors				
		HL-93	HL-93									
		TRUCK	TANDEM &	H20	TYPE	TYPE	SU4	SU5	SU6	SU7	EV2	EV3
Br	Bridge Element	& LANE	LANE	П20	3	3S2	304	303	300	307	EVZ	EV3
		LOAD	LOAD									
Д ч	Axial	3.80	4.42	3.92	5.99	6.42	5.99	5.99	5.99	5.99	3.82	4.00
CMP	Buckling	3.80	4.42	3.92	5.99	6.42	5.99	5.99	5.99	5.99	3.82	4.00
∪ ▼	Seam Strength	3.77	4.38	3.89	5.94	6.36	5.94	5.94	5.94	5.94	3.79	3.96

					Invento	•	by LRFR h Tons)	R Method				
Br	idge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
— ч	Axial	105.64	85.21	60.53	115.56	178.27	124.80	143.29	160.63	179.12	84.68	132.65
CMP	Buckling	105.64	85.21	60.53	115.56	178.27	124.80	143.29	160.63	179.12	84.68	132.65
⊃ ₹	Seam Strength	104.74	84.48	60.01	114.57	176.74	123.74	142.07	159.25	177.59	83.96	131.51

					Operati	_	g by LRFI sh Tons)	R Method				
Br	idge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
<u>а</u> ч	Axial	136.94	110.45	78.46	149.80	231.09	161.78	185.75	208.22	232.19	109.77	171.95
CMP	Buckling	136.94	110.45	78.46	149.80	231.09	161.78	185.75	208.22	232.19	109.77	171.95
⊃	Seam Strength	135.77	109.51	77.79	148.52	229.11	160.40	184.16	206.44	230.20	108.83	170.48

Rating for 50% Section Loss of Original Culvert Thickness

	Inventory LRFR Rating Factors												
					Inve	ntory LRI	FR Rating	Factors		_			
		HL-93	HL-93										
		TRUCK	TANDEM &	1120	TYPE	TYPE	CIIA	CI 15	CI I	CI 17	EVA	EV/2	
Br	Bridge Element		LANE	H20	3	3S2	SU4	SU5	SU6	SU7	EV2	EV3	
			LOAD										
д ч	Axial	2.01	2.34	2.08	3.17	3.40	3.17	3.17	3.17	3.17	2.02	2.12	
CMP	Buckling	1.55	1.80	1.60	2.45	2.62	2.45	2.45	2.45	2.45	1.56	1.63	
∪ ₹	Seam Strength	2.91	3.38	3.00	4.58	4.91	4.58	4.58	4.58	4.58	2.92	3.06	

					Oper	ating LR	FR Rating	g Factors				
		HL-93	HL-93									
	• • • • •	TRUCK & LANE	TANDEM &	H20	TYPE	TYPE	SU4	SU5	SU6	SU7	EV2	EV3
Br	Bridge Element		LANE	1120	3	3S2				50,	2,2	1
	_		LOAD									
_ A_ 4	Axial	2.61	3.03	2.69	4.11	4.41	4.11	4.11	4.11	4.11	2.62	2.75
CMP	Buckling	2.01	2.34	2.08	3.17	3.40	3.17	3.17	3.17	3.17	2.02	2.12
	Seam Strength	3.77	4.38	3.89	5.94	6.36	5.94	5.94	5.94	5.94	3.79	3.96

					Invent	•	g by LRF ish Tons)	R Method				
Br	idge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
Д ч	Axial	72.53	58.50	41.56	79.34	112.40	85.69	98.38	110.29	122.98	58.14	91.07
CMP	Buckling	55.95	45.12	32.06	61.20	94.41	66.09	75.89	85.07	94.86	44.85	70.25
	Seam Strength	104.74	84.48	60.01	114.57	176.74	123.74	142.07	159.25	177.59	83.96	131.51

					Opera	ting Ratin (Engl	g by LRF ish Tons)		d			
Bri	idge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
	Axial	94.02	75.83	53.87	102.85	158.66	111.08	127.53	142.96	159.42	75.37	118.06
CMP	Buckling	72.52	58.49	41.55	79.33	122.38	85.68	98.37	110.27	122.96	58.13	91.06
0 4	Seam Strength	135.77	109.51	77.79	148.52	229.11	160.40	184.16	206.44	230.20	108.83	170.48

Failed Rating for 57% Section Loss of Original Culvert Thickness

							FR Rating	Factors				
		HL-93	HL-93									
		TRUCK	TANDEM &	H20	TYPE	TYPE	SU4	SU5	SU6	SU7	EV2	EV3
Br	Bridge Element		LANE	п20	3	3S2	304	303	300	307	E V Z	EVS
			LOAD									
	Axial	1.72	2.00	1.77	2.71	2.90	2.71	2.71	2.71	2.71	1.72	1.81
CMP	Buckling	0.97	1.13	1.00	1.53	1.64	1.53	1.53	1.53	1.53	0.98	1.02
	Seam Strength	2.91	3.38	3.00	4.58	4.91	4.58	4.58	4.58	4.58	2.92	3.06

					Oper	ating LR	FR Rating	g Factors				
		HL-93	HL-93									
		TRUCK	TANDEM &	H20	TYPE	TYPE	SU4	SU5	SU6	SU7	EV2	EV3
Bridge Element		& LANE	LANE	1120	3	3S2	504	503	500	507	LVZ	LVJ
		LOAD	LOAD									
	Axial	2.23	2.59	2.30	3.51	3.76	3.51	3.51	3.51	3.51	2.24	2.34
CMP	Buckling	1.26	1.47	1.30	1.99	2.13	1.99	1.99	1.99	1.99	1.27	1.33
\(\frac{1}{2}\) \(\frac{1}{2}\)	Seam Strength	3.77	4.38	3.89	5.94	6.36	5.94	5.94	5.94	5.94	3.79	3.96

				Inve	ntory Rati	ing by LR	FR Metho	od (Englis	h Tons)			
		HL-93	HL-93									
		TRUCK	TANDEM &									
Bridge Element		& LANE	LANE	H20	TYPE	TYPE	SU4	SU5	SU6	SU7	EV2	EV3
		LOAD	LOAD		3	3S2						
д ч	Axial	61.84	49.88	35.43	67.64	104.35	73.06	83.88	94.03	104.85	49.57	77.65
CMP	Buckling	35.04	28.26	20.08	38.33	59.13	41.39	47.53	53.28	59.41	28.09	43.99
0 4	Seam Strength	104.74	84.48	60.01	114.57	176.74	123.74	142.07	159.25	177.59	83.96	131.51

			Operating Rating by LRFR Method (English Tons)									
Bri	idge Element	HL-93 TRUCK & LANE LOAD	HL-93 TANDEM & LANE LOAD	H20	TYPE 3	TYPE 3S2	SU4	SU5	SU6	SU7	EV2	EV3
	Axial	80.16	64.65	45.93	87.69	135.27	94.70	108.73	121.89	135.92	64.26	100.65
CMP	Buckling	45.42	36.63	26.02	49.68	76.64	53.66	61.61	69.06	77.01	36.41	57.03
○ ⋖	Seam Strength	135.77	109.51	77.79	148.52	229.11	160.40	184.16	206.44	230.20	108.83	170.48

Appendix A – Design Calculations

- I. Structural Steel Plate Arch Culvert Rating with Existing Conditions, thirteen (13) page(s).
- II. Structural Steel Plate Arch Culvert Rating with 50% Section Loss, thirteen (13) page(s).
- III. Structural Steel Plate Arch Culvert Rating with 57% Section Loss, thirteen (13) page(s).





Calculated by: M. Martell

Date: 10/17/2024

Task: Structural Steel Plate Arch Culvert Rating

Checked by: E. Caron Date: 10/22/2024

Lawrence, MA 01843

Client: Cell Signaling Technologies Inc.

Governing Specifications: AASHTO LRFD Bridge Design Specification, 9th Edition, 2020 (AASHTO)

MassDOT 2020 LRFD Bridge Manual 100th Anniversary Edition (B/M)

The Manual for Bridge Evaluation, Third Edition with 2019 Revisions (MBE)

Existing Bridge Plans, M-02-021, dated 1952 (Exist. Plans)

MassDOT Culvert inspection Report, dated 10/1/2024 (Inspection Report)

Test Boring Log by GZA GeoEnvironmental Inc., dated 9/3/2024 (Geotech)

Design Methodology: Load and Resistance Factor Design (LRFD)

<u>Purpose:</u> The following calculation is the load rating for the corrugated steel plate arch at Atwater Avenue over Sawmill Brook in Manchester, MA. The calculation below was done for all applicable design vehicles listed in Part I Chapter 7 of the MassDOT LRFD Bridge Manual.

General information:

Roadway elevation:	$EL_{road} := 148.45 ft$	Exist. Plans
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Culvert crown elevation:
$$EL_{cul.crown} := 147.08 \ ft$$
 Exist. Plans

Height of cover above top of
$$H_{cover} := EL_{road} - EL_{cul.crown} = 1.37 \text{ ft}$$

crown:

Highest elevation above culvert
$$EL_{high} := 149.19 \text{ ft}$$
 Exist. Plans crown:

 $EL_{low} := 148.33 ft$ Lowest elevation above the Exist. Plans

culvert crown:

Average height of soil:
$$H_{avg_soil} := \left(\frac{EL_{high} + EL_{low}}{2}\right) - EL_{cul.crown} = 1.68 \text{ ft}$$

Pavement thickness:
$$H_{pavement} := 0.44 \ ft$$
 Geotech

Span length:
$$l_{span} := 15 \text{ ft}$$
 Exist. Plans

Height of opening:
$$h_{open} := 79 \text{ in}$$
 Exist. Plans

Thickness of original corrugated Exist. Plans $t_0 := 0.218 \ in$ steel:

Thickness of remaining $t_c := 0.15625 in$ Inspection Report corrugated steel:

Bolted seam strength: $F_{u \text{ seam}} := 112 \text{ klf}$ *For 3/4-inch diameter bolt AASHTO Table A12-8

that is 0.218-inches thick Minimum yield stress of the $F_v := 33 \ ksi$ AASHTO Table A12-10

Minimum tensile stress of the $F_n := 45 \text{ ksi}$ AASHTO Table A12-10

metal: $E_s := 29000 \ ksi$ Modulus of elasticity of steel: AASHTO Table A12-10

 $A_{s,o} := 3.199 \ in^2$ Original cross sectional area AASHTO Table A12-3

 $I_{x.o} := 0.1269 \frac{in^4}{in}$ *For 6" x 2" corrugations Original moment of inertia of AASHTO Table A12-3 pipe:

Print Date:

metal:

of pipe:



Original radius of gyration of the corrugation:

$$r_{x.o}\!:=\!0.690~in$$

AASHTO Table A12-3

Ratio of remaining thickness to original thickness:

$$\rho_t := \frac{t_c}{t_o} = 0.72$$

Assuming that the remaining section's cross sectional area, moment of inertia, and radius of gyration can be approximated by multiplying the original section properties by the ratio of the remaining steel thickness to original thickness.

Cross sectional area of

$$A_s := A_{s.o} \cdot \rho_t = 2.293 \ in^2$$

Exist. Plans

remaining pipe:

$$I_x := I_{x.o} \cdot \rho_t = 0.091 \frac{in^4}{in}$$

Exist. Plans

Radius of gyration of the remaining corrugation:

$$r_x := r_{x.o} \cdot \rho_t = 0.495 \ in$$

Exist. Plans

Resistance factor for buried structures for wall area and buckling:

$$\phi_b := 1.00$$

AASHTO Table 12.5.5-1

Resistance factor for buried structures for seam strength:

$$\phi_s := 0.67$$

AASHTO Table 12.5.5-1

Soil Stiffness Factor:

$$k := 0.22$$

AASHTO 12.7.2.4

Design Vehicle Properties:

Live Load Distribution Factor transverse or parallel to span:

LLDF := 1.15

*for all buried structures excluding

 $i := 1 ... rows (VT_{matrix})$

AASHTO Table 3.6.1.2.6a-1

concrete pipes

"HL-93 Truck"

"SU7" "EV2" "EV3"

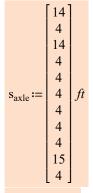
Vehicle type matrix title:

Wheel spacing for each vehicle type:

 $s_{\text{wheel}} := 6 ft$

AASHTO 3.6.1.2.6a-b

Axle spacing for each vehicle type:



*Assume most narrow axle spacing for

loading conditions.

AASHTO 3.6.1.2.2 & B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

Assumed tire pressure:

$$p_{tire} := 80 \ psi$$

B/M Pt. I - 7.2.5.9

Tire patch length:

$$l_{patch} := 10 in$$

B/M Pt. I - 7.2.5.9 & AASHTO 3.6.1.2.5

Number of design lanes:

 $N_{lanes} := 1$

*for traffic parallel to span

AASHTO 3.6.1.2.6a



Multiple presence factor: m := 1.2 AASHTO Table 3.6.1.1.2-1

Live load per wheel for each vehicle type (looking transversely):	$P_t :=$	[16 12.5 16 8.5 7.75 8.5 8.5 8.5 16.75 15.5	kip
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AASHTO 3.6.1.2.2, AASHTO 3.6.1.2.3, & B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

Density of soil: $\rho_{\text{soil}} := 120 \text{ pcf}$ AASHTO Table 3.5.1-1

Density of pavement: $\rho_{\text{pave}} := 140 \text{ pcf}$ AASHTO Table 3.5.1-1

Design live lane load: $\omega_{lane} := 0.64 \text{ klf}$ AASHTO 3.6.1.2.4

Width of design live lane load: $w_{lane} := 10 \text{ ft}$ AASHTO 3.6.1.2.4

Determine Vehicle Loading:

Vehicle tire contact area: $A_{\text{tire}} := \frac{P_{t}}{P_{\text{tire}}} = \begin{bmatrix} 200.00 \\ 156.25 \\ 200.00 \\ 106.25 \\ 96.88 \\ 106.25 \\ 106.25 \\ 106.25 \\ 106.25 \\ 209.38 \\ 193.75 \end{bmatrix} in^{2} B/M Pt. I - 7.2.5.9$

20.00

Vehicle wheel load interaction depth transverse to culvert: $H_{\text{int_t}} := \frac{s_{\text{wheel}} - w_{\text{patch}} - 0.06 \, l_{\text{span}}}{\text{LLDF}} = \begin{bmatrix} 3.30 \\ 2.99 \\ 3.66 \\ 3.73 \\ 3.66 \\ 3.66 \\ 3.66 \\ 3.66 \\ 2.92 \\ 3.03 \end{bmatrix}$

2.99

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The Engineering Corp

Project: Manchester, MA - T1199.02

Vehicle wheel live load patch width at depth H:

$$\begin{split} \mathbf{w_{w.LL_i}} \coloneqq & \text{if } \mathbf{H_{cover}} \!<\! \mathbf{H_{int_{-l_i}}} \\ & \qquad \qquad \| \mathbf{w_{patch_i}} \!+\! \mathbf{LLDF} \! \cdot \! \mathbf{H_{cover}} \! + \! 0.06 \ \mathbf{l_{span}} \\ & \qquad \qquad \qquad \\ & \qquad \qquad \\ & \qquad \qquad \qquad \\ & \qquad \qquad \\ & \qquad \qquad \qquad \\ & \qquad$$

AASHTO Eqs. 3.6.1.2.6b-2 & -3

$$\mathbf{w}_{\text{w.LL}} = \begin{bmatrix} 4.14 \\ 3.78 \\ 4.14 \\ 3.36 \\ 3.28 \\ 3.36 \\ 3.36 \\ 3.36 \\ 3.36 \\ 4.22 \\ 4.09 \end{bmatrix} ft$$

Vehicle axel load interaction depth parallel to culvert:

$$H_{\text{int_p}} := \frac{s_{\text{axle}} - l_{\text{patch}}}{\text{LLDF}} = \begin{bmatrix} 2.75 \\ 11.45 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 12.32 \\ 2.75 \end{bmatrix}$$

AASHTO Eq. 3.6.1.2.6b-4

Vehicle live load patch length at depth H:

AASHTO Eqs. 3.6.1.2.6b-5 & -6

Vehicle dynamic allowance for buried structures:

IM :=
$$\left(33 \cdot \left(1.0 - 0.125 \cdot \frac{H_{cover}}{ft}\right)\%\right) = 27.35\%$$

AASHTO Eq. 3.6.2.2-1



Vehicle live load rectangular patch area at depth H: $A_{LL_{i}} := w_{w.LL_{i}} \cdot l_{w.LL_{i}} = \begin{bmatrix} 9.98 \\ 9.10 \\ 9.98 \\ 8.10 \\ 7.91 \\ 8.10 \\ 8.10 \\ 8.10 \\ 8.10 \\ 10.17 \\ 9.85 \end{bmatrix}$ $AASHTO Eq. \ 3.6.1.2.6a-1$

Vehicle live load vertical crown pressure:

$$P_{L} := \frac{P_{t} \cdot \left(1 + \frac{IM}{100\%}\right) \cdot m}{A_{LL}} = \begin{bmatrix} 2.43 \\ 2.10 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{bmatrix} ksf$$

$$AASHTO Eq. \ 3.6.1.2.6b-7$$

Design Loads:

Unfactored dead load crown pressure due to soil:

$$P_{EV} := H_{avg_soil} \cdot \rho_{soil} = 0.20 \ ksf$$

Unfactored dead load crown pressure due to pavement:

$$P_{DW} := H_{pavement} \cdot \rho_{pave} = 0.06 \text{ ksf}$$

Unfactored design lane load:

$$\mathbf{P_{LL_{i}}} \coloneqq \mathbf{P_{L_{i}}} + \mathbf{P_{lane_{i}}} = \begin{vmatrix} 2.18 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{vmatrix} ksf$$



Load factors:

 $\gamma_{\text{EV}} \coloneqq 1.30$

 $\gamma_{\rm DW} := 1.50$

4.42

4.21

3.41 2.94 3.31

> 4.78 4.16 4.64

3.77 3.29

 $\gamma_{LL.inv} := 1.75$

 $\gamma_{LL.opr} := 1.35$

AASHTO Table 3.4.1-1, MBE Table 6A.4.2.2-1 & Table 6A.5.12.5-1

Factored dead load crown pressure due to soil:

$$P_{u.EV} := P_{EV} \cdot \gamma_{EV} = 0.26 \text{ ksf}$$

Factored dead load crown

$$P_{u.DW} := P_{DW} \cdot \gamma_{DW} = 0.09 \text{ ksf}$$

$$P_{u.LL.inv_{i}} := P_{LL_{i}} \cdot \gamma_{LL.inv} = \begin{vmatrix} 3.81 \\ 4.29 \\ 2.81 \\ 2.62 \\ 2.81 \\ 2.81 \\ 2.81 \\ 2.81 \\ 4.41 \end{vmatrix} ksf$$

Factored inventory vehicle live load crown pressure:

Factored operating vehicle live
$$P_{u.LI}$$
 load crown pressure:

$$P_{u.LL.opr_{i}} := P_{LL_{i}} \cdot \gamma_{LL.opr} = \begin{vmatrix} 2.17 \\ 2.02 \\ 2.17 \\ 2.17 \\ 2.17 \\ 2.17 \\ 2.17 \\ 3.40 \\ 3.25 \end{vmatrix} ksf$$

Combined factored loads when analyzing inventory vehicles:

$$P_{inv_{i}} := P_{u.EV} + P_{u.DW} + P_{u.LL.inv_{i}} = \begin{vmatrix} 3.16 \\ 2.98 \\ 3.16 \\ 3.16 \\ 3.16 \\ 3.16 \\ 4.76 \\ 4.56 \end{vmatrix} ksf$$

Combined factored loads when analyzing operating vehicles:

$$P_{opr_i} := P_{u.EV} + P_{u.DW} + P_{u.LL.opr_i} = \begin{vmatrix} 3.66 \\ 2.52 \\ 2.38 \\ 2.52 \\ 2.52 \\ 2.52 \\ 2.52 \\ 3.75 \\ 3.60 \end{vmatrix} ks$$

AASHTO Eq. 12.7.2.2-1 (modified)



Project: Manchester, MA - T1199.02

Determine Thrust:

Width of culvert on which live load is applied:	$\mathbf{C_{L_{i}}} \coloneqq min\left(\mathbf{l_{w.LL_{i}}}, \mathbf{l_{span}}\right) =$	2.41 2.41 2.41 2.41 2.41 2.41 2.41 2.41	1	. 12.7.2.2-2
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F_{min} value:
$$F_{min} := max \left(\frac{15}{12 \cdot \frac{l_{span}}{ft}}, 1.00 \right) = 1.00$$
 AASHTO Eq. 12.7.2.2-4

F₁ value:
$$F_{1_{i}} := \max \left(\frac{0.75 \cdot l_{span}}{l_{w.LL_{i}}}, F_{min} \right) = \begin{vmatrix} 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \end{vmatrix}$$
AASHTO Eq. 12.7.2.2-3

 $T_{L.EV} := \frac{P_{u.EV} \cdot l_{span}}{2} = 1.97 \ klf$

Factored thrust per unit length of wall due to dead load pressure caused by soil:

$$T_{L.DW} := \frac{P_{u.DW} \cdot l_{span}}{2} = 0.69 \text{ klf}$$
AASHTO Eq. 12.7.2.2-1 (modified)

Factored thrust per unit length of wall due to live load pressure caused by inventory vehicle:
$$T_{\text{L.inv}_{i}} = \frac{P_{\text{u.LL.inv}_{i}} \cdot F_{1_{i}} \cdot C_{L_{i}}}{2} = \begin{bmatrix} 21.42 \\ 24.12 \\ 15.79 \\ 14.74 \\ 15.79 \\ 15.79 \\ 15.79 \\ 15.79 \\ 24.79 \\ 23.67 \end{bmatrix} klf$$
AASHTO Eq. 12.7.2.2-1 (modified)

$$T_{L.opr_{i}} := \frac{P_{u.LL.opr_{i}} \cdot F_{1_{i}} \cdot C_{L_{i}}}{2} = \begin{bmatrix} 19.19 \\ 16.52 \\ 18.61 \\ 12.18 \\ 11.37 \\ 12.18 \\ 12.18 \\ 12.18 \\ 12.18 \\ 19.12 \\ 18.26 \end{bmatrix}$$

$$AASHTO Eq. \ 12.7.2.2-1 \ (modified)$$

Total factored thrust with inventory design vehicle: $T_{\text{L.inv}_{i}} \coloneqq T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.inv}_{i}} = \begin{bmatrix} 27.54 \\ 24.08 \\ 26.78 \\ 18.45 \\ 17.40 \\ 18.45 \\ 18.45 \\ 18.45 \\ 18.45 \\ 18.45 \\ 27.44 \\ 26.32 \end{bmatrix}$

Total factored thrust with operating design vehicle: $T_{\text{L.opr}_{i}} := T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.opr}_{i}} = \begin{vmatrix} 21.27 \\ 14.84 \\ 14.03 \\ 14.84 \\ 14.84 \\ 14.84 \\ 14.84 \\ 21.78 \end{vmatrix} \text{ klf}$

Determine Resistance:

Critical buckling stress:

$$\begin{split} f_{cr} \coloneqq & \text{if } l_{span} \!<\! \left(\frac{r_x}{k}\right) \! \cdot \! \sqrt{\frac{24 \cdot E_s}{F_u}} = 35.67 \text{ ksi} \\ & \left\| \frac{\left(\frac{F_u \cdot k \cdot l_{span}}{r_x}\right)^2}{48 \cdot E_s} \right\| = 35.67 \text{ ksi} \end{split}$$

$$= 35.67 \text{ ksi}$$

$$= 25.67 \text{ ksi}$$

21.85 19.18

The factored axial resistance per unit of wall:

$$R_{\text{n.axial}} := \left(\phi_b \cdot F_y \cdot A_s\right) \frac{1}{ft} = 75.66 \text{ klf}$$
AASHTO Eq. 12.7.2.3-1

The factored buckling resistance per unit of wall:

$$R_{\text{n.buckling}} := \phi_b \cdot \left(min \left(f_{cr}, F_y \right) \right) \cdot A_s \frac{1}{ft} = 75.66 \text{ klf}$$

$$AASHTO Eq. 12.7.2.3-1$$

The factored seam strength per unit of wall:

$$R_s := \phi_s \cdot F_{u_seam} = 75.04 \text{ klf}$$
 AASHTO 12.7.2.5



Design Vehicle Rating Factors:

 $Wt_{v} := \begin{bmatrix} 36 \\ 25 \\ 20 \\ 25 \\ 36 \\ 27 \\ 31 \\ 34.75 \\ 38.75 \\ 28.75 \\ 43 \end{bmatrix} tonf$

AASHTO 3.6.1.2.2 & B/M Pt. I Ch. 7

vehicles:

Total weight of design

 $\phi_s := 1.00$

MBE Table 6A4.2.4-.1

Condition factor:

System factor:

 $\varphi_c := 1.00$

*condition factor taken as 1.00 since B/M states that they shall not be used

> 2.93 3.41

> > 2.93

MBE Table 6A4.2.3-.1 & B/M Pt. I 7.2.5.2A

Axial wall capacity:

$$C_{axial} := R_{n.axial} \cdot \varphi_s \cdot \varphi_c = 75.66 \ klf$$

MBE Eq. 6A4.2.1-2

Buckling capacity:

$$C_{\text{buckling}} := R_{\text{n.buckling}} \cdot \varphi_s \cdot \varphi_c = 75.66 \text{ klf}$$

MBE Eq. 6A4.2.1-2

Seam strength capacity:

$$C_{seam} := R_s \cdot \varphi_s \cdot \varphi_c = 75.04 \ klf$$

MBE Eq. 6A4.2.1-2

Inventory load rating factor for axial wall resistance:

$$LR_{axial.inv} := \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{bmatrix} 3.03 \\ 4.62 \\ 4.95 \\ 4.62 \\ 4.62 \\ 4.62 \\ 4.62 \\ 2.95 \\ 3.08 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

Inventory load rating factor for buckling resistance:

$$LR_{buckling.inv} := \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{vmatrix} 3.41 \\ 3.03 \\ 4.62 \\ 4.95 \\ 4.62 \\ 4.62 \\ 4.62 \\ 4.62 \\ 2.95 \\ 3.08 \end{vmatrix}$$

MBE Eq. 6A4.2.1-1

Inventory load rating factor for seam strength:	$LR_{\text{seam.inv}} := \frac{C_{\text{seam}} - T_{\text{L.EV}} - T_{\text{L.DW}}}{T_{\text{L.inv}}}$	$\begin{bmatrix} 2.91 \\ 3.38 \\ 3.00 \\ 4.58 \\ 4.91 \\ = 4.58 \\ 4.58 \\ 4.58 \\ 4.58 \\ 2.92 \\ 3.06 \end{bmatrix}$	MBE Eq. 6A4.2.1-1
		[280]	

Operating load rating factor for axial wall resistance:

$$LR_{axial.opr} := \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{bmatrix} 3.92\\ 5.99\\ 6.42\\ 5.99\\ 5.99\\ 5.99\\ 5.99\\ 3.82\\ 4.00 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

Operating load rating factor for buckling resistance:
$$LR_{buckling.opr} := \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{bmatrix} 3.92\\ 5.99\\ 6.42\\ 5.99\\ 5.99\\ 5.99\\ 3.82\\ 4.00 \end{bmatrix}$$

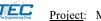
MBE Eq. 6A4.2.1-1

$$LR_{\text{seam.opr}} := \frac{C_{\text{seam}} - T_{\text{L.EV}} - T_{\text{L.DW}}}{T_{\text{L.opr}}} = \begin{bmatrix} 3.89 \\ 5.94 \\ 6.36 \\ 5.94 \\ 5.94 \\ 5.94 \\ 5.94 \\ 3.79 \\ 3.96 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

4.42

4.38



Inventory load rating factor for axial wall resistance in tons:

$$TLR_{axial.inv_{i}} := LR_{axial.inv_{i}} \cdot Wt_{v_{i}} = \begin{bmatrix} 105.64 \\ 85.21 \\ 60.53 \\ 115.56 \\ 178.27 \\ 124.80 \\ 143.29 \\ 160.63 \\ 179.12 \\ 84.68 \\ 132.65 \end{bmatrix}$$

Inventory load rating factor for buckling resistance in tons:

$$TLR_{buckling.inv_{i}} := LR_{buckling.inv_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 60.53 \\ 115.56 \\ 178.27 \\ 124.80 \\ 143.29 \\ 160.63 \\ 179.12 \\ 84.68 \\ 132.65 \end{vmatrix} tonf$$

105.64 85.21

Inventory load rating factor for seam strength in tons:

$$TLR_{seam.inv_{i}} := LR_{seam.inv_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 114.57 \\ 176.74 \\ 123.74 \\ 142.07 \\ 159.25 \\ 177.59 \\ 83.96 \\ 131.51 \end{vmatrix} tonf$$

Operating load rating factor for axial wall resistance in tons:

$$TLR_{axial.opr_{i}} := LR_{axial.opr_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 78.46 \\ 149.80 \\ 231.09 \\ 161.78 \\ 185.75 \\ 208.22 \\ 232.19 \\ 109.77 \\ 171.95 \end{vmatrix} tony$$

136.94 110.45



Operating load rating factor for buckling resistance in tons: $TLR_{buckling.opr_{i}} := LR_{buckling.opr_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 78.46 \\ 149.80 \\ 231.09 \\ 161.78 \\ 185.75 \\ 208.22 \\ 232.19 \\ 109.77 \\ 171.95 \end{vmatrix}$

Operating load rating factor for seam strength in tons:

$$TLR_{seam.opr_{i}} := LR_{seam.opr_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 77.79 \\ 148.52 \\ 229.11 \\ 160.40 \\ 184.16 \\ 206.44 \\ 230.20 \\ 108.83 \\ 170.48 \end{vmatrix} tonf$$

Summary Tables of Rating Factors:

Summary of Inventory Load Rating Factors.

Heading_{inv} := ["Loading" "Inv. Yielding" "Inv. Buckling" "Inv. Seam Strength"]

135.77 109.51

 $Table_{inv} \coloneqq stack \left(Heading_{inv}, augment \left(VT_{matrix}, LR_{axial.inv}, LR_{buckling.inv}, LR_{seam.inv} \right) \right)$

136.94 110.45

Table _{inv} =	"Loading" "HL-93 Truck" "HL-93 Tandem" "H20" "Type 3" "Type 3S2" "SU4" "SU5" "SU6"	"Inv. Yielding" 2.93 3.41 3.03 4.62 4.95 4.62 4.62 4.62	"Inv. Buckling" 2.93 3.41 3.03 4.62 4.95 4.62 4.62 4.62	"Inv. Seam Strength" 2.91 3.38 3.00 4.58 4.91 4.58 4.58 4.58	
Table _{inv} =	"SU4" "SU5" "SU6" "SU7"	4.62 4.62 4.62 4.62	4.62 4.62 4.62 4.62	4.58 4.58 4.58 4.58	
	"EV2" "EV3"	2.95 3.08	2.95 3.08	2.92 3.06	

Summary of Operating Load Rating Factors.

Heading_{opr}:= ["Loading" "Opr. Yielding" "Opr. Buckling" "Opr. Seam Strength"]

 $Table_{opr} \coloneqq stack \left(Heading_{opr}, augment \left(VT_{matrix}, LR_{axial.opr}, LR_{buckling.opr}, LR_{seam.opr} \right) \right)$

	"Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"]
	"HL-93 Truck"	3.80	3.80	3.77
	"HL-93 Tandem"	4.42	4.42	4.38
	"H20"	3.92	3.92	3.89
	"Type 3"	5.99	5.99	5.94
Table -	"Type 3S2"	6.42	6.42	6.36
Table _{opr} =	"SU4"	5.99	5.99	5.94
	"SU5"	5.99	5.99	5.94
	"SU6"	5.99	5.99	5.94
	"SU7"	5.99	5.99	5.94
	"EV2"	3.82	3.82	3.79
	"EV3"	4.00	4.00	3.96

Summary of Inventory Load Rating Factors in Tons.

Heading_{t.inv}:= ["Loading" "Inv. Yielding" "Inv. Buckling" "Inv. Seam Strength"]

$$Table_{t.inv} \coloneqq stack \left(Heading_{t.inv}, augment \left(VT_{matrix}, \frac{TLR_{axial.inv}}{\textit{ton}f}, \frac{TLR_{buckling.inv}}{\textit{ton}f}, \frac{TLR_{seam.inv}}{\textit{ton}f} \right) \right)$$

$Table_{t.inv} =$	"Loading" "HL-93 Truck" "HL-93 Tandem" "H20" "Type 3" "Type 3S2" "SU4" "SU5" "SU6" "SU6"	105.64 85.21 60.53 115.56 178.27 124.80 143.29 160.63	105.64 85.21 60.53 115.56 178.27 124.80 143.29 160.63	"Inv. Seam Strength" 104.74 84.48 60.01 114.57 176.74 123.74 142.07 159.25 177.59
	200			
	"EV2" "EV3"	84.68 132.65	84.68 132.65	83.96 131.51

Summary of Operating Load Rating Factors in Tons.

 $Heading_{t.opr} \coloneqq \begin{bmatrix} \text{``Loading'' '`Opr. Yielding'' '`Opr. Buckling'' '`Opr. Seam Strength''} \end{bmatrix}$

$$Table_{t.opr} \coloneqq stack \left(Heading_{t.opr}, augment \left(VT_{matrix}, \frac{TLR_{axial.opr}}{tonf}, \frac{TLR_{buckling.opr}}{tonf}, \frac{TLR_{seam.opr}}{tonf} \right) \right)$$

	_			_
	"Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"
	"HL-93 Truck"	136.94	136.94	135.77
	"HL-93 Tandem"	110.45	110.45	109.51
	"H20"	78.46	78.46	77.79
	"Type 3"	149.80	149.80	148.52
Table -	"Type 3S2"	231.09	231.09	229.11
$Table_{t.opr} =$	"SU4"	161.78	161.78	160.40
	"SU5"	185.75	185.75	184.16
	"SU6"	208.22	208.22	206.44
	"SU7"	232.19	232.19	230.20
	"EV2"	109.77	109.77	108.83
	"EV3"	171 95	171 95	170 48





282 Merrimack Street Lawrence, MA 01843 Project: Manchester, MA - T1199.02

Calculated by: M. Martell

Date: 10/28/2024

Rating

<u>Checked by</u>: E. Caron <u>Date:</u> 10/28/2024

Exist. Plans

Task: Structural Steel Plate Arch Culvert Rating

with 50% Section Loss

<u>Client:</u> Cell Signaling Technologies Inc.

Governing Specifications: AASHTO LRFD Bridge Design Specification, 9th Edition, 2020 (AASHTO)

MassDOT 2020 LRFD Bridge Manual 100th Anniversary Edition (B/M)

The Manual for Bridge Evaluation, Third Edition with 2019 Revisions (MBE)

Existing Bridge Plans, M-02-021, dated 1952 (Exist. Plans)

Test Boring Log by GZA GeoEnvironmental Inc., dated 9/3/2024 (Geotech)

Design Methodology: Load and Resistance Factor Design (LRFD)

<u>Purpose:</u> The following calculation is the load rating for the corrugated steel plate arch at Atwater Avenue over Sawmill Brook in Manchester, MA with 50% section loss in the thickness of the steel. The calculation below was done for all applicable design vehicles listed in Part I Chapter 7 of the MassDOT LRFD Bridge Manual.

General information:

Culvert crown elevation:
$$EL_{cul crown} := 147.08 \ ft$$
 Exist. Plans

Height of cover above top of
$$H_{cover} := EL_{road} - EL_{cul.crown} = 1.37 \text{ ft}$$
 crown:

Highest elevation above culvert
$$EL_{high} := 149.19 \text{ } ft$$
 crown:

Lowest elevation above the
$$EL_{low} := 148.33 \ ft$$
 Exist. Plans

culvert crown:

Average height of soil:
$$H_{\text{avg_soil}} \coloneqq \left(\frac{\text{EL}_{\text{high}} + \text{EL}_{\text{low}}}{2}\right) - \text{EL}_{\text{cul.crown}} = 1.68 \text{ ft}$$

Pavement thickness:
$$H_{pavement} := 0.44 \text{ ft}$$
 Geotech

Span length:
$$l_{span} := 15 \text{ ft}$$
 Exist. Plans

Height of opening:
$$h_{open} := 79 \text{ in}$$
 Exist. Plans

Thickness of original corrugated
$$t_0 := 0.218 \text{ in}$$
 Exist. Plans

steel:

$$50\%$$
 of the thickness of original $t_0 := 0.50 \cdot t_0 = 0.11$ in

Bolted seam strength:
$$F_{u_seam} := 112 \text{ klf}$$
 *For 3/4-inch diameter bolt that is 0.218-inches thick

Minimum yield stress of the metal:

$$F_y := 33 \text{ ksi}$$

metal:

AASHTO Table A12-10

Minimum tensile stress of the metal:

$$F_u := 45 \text{ ksi}$$

metal:

Modulus of elasticity of steel:
$$E_s = 29000 \text{ ksi}$$
 AASHTO Table A12-10

Original cross sectional area of pipe:

$$A_{s.o} := 3.199 \text{ in}^2$$

AASHTO Table A12-3

Original moment of inertia of pipe:
$$I_{x.o} := 0.1269 \frac{in^4}{in} \text{*For 6" x 2" corrugations}$$

$$AASHTO Table A12-3$$

Original radius of gyration of the corrugation:

$$r_{x.o} := 0.690 \text{ in}$$

AASHTO Table A12-3

Exist. Plans



Project: Manchester, MA - T1199.02

Ratio of remaining thickness to original thickness:

$$\rho_t := \frac{t_c}{t_o} = 0.50$$

Assuming that the remaining section's cross sectional area, moment of inertia, and radius of gyration can be approximated by multiplying the original section properties by the ratio of the remaining steel thickness to original thickness.

Cross sectional area of $A_s := A_{s,o} \cdot \rho_t = 1.600 \ in^2$

remaining pipe:

Moment of inertia of $I_x := I_{x,o} \cdot \rho_t = 0.063 \frac{in^4}{in}$ Exist. Plans remaining pipe:

Radius of gyration of the $r_x := r_{x.o} \cdot \rho_t = 0.345$ in Exist. Plans remaining corrugation:

Resistance factor for buried structures for wall area and $\phi_b := 1.00$

buckling:

Resistance factor for buried $\phi_s := 0.67$ structures for seam strength:

Soil Stiffness Factor: k = 0.22 AASHTO 12.7.2.4

Design Vehicle Properties:

Live Load Distribution Factor transverse or parallel to span:

*for all buried structures excluding concrete pipes

*for all buried structures excluding concrete pipes

 $Vehicle type matrix title: VT_{matrix} := \begin{bmatrix} \text{"HL-93 Truck"} \\ \text{"H20"} \\ \text{"Type 3"} \\ \text{"Type 3S2"} \\ \text{"SU4"} \\ \text{"SU5"} \\ \text{"SU6"} \\ \text{"SU6"} \\ \text{"EV2"} \\ \text{"EV2"} \\ \text{"EV3"} \end{bmatrix} i := 1 \dots \text{rows} \left(VT_{\text{matrix}} \right)$

Wheel spacing for each vehicle type: $s_{\text{wheel}} := 6 \text{ ft}$ AASHTO 3.6.1.2.6a-b

 $s_{axle} := \begin{bmatrix} 14 \\ 4 \\ 4 \\ 4 \end{bmatrix}$ ft*Assume most narrow axle spacing for loading conditions.

14

Axle spacing for each vehicle type:

Assumed tire pressure: $p_{tire} := 80 \ psi$ $B/M \ Pt. \ I - 7.2.5.9$

Tire patch length: $l_{\text{patch}} := 10 \text{ in}$ B/M Pt. I - 7.2.5.9 & AASHTO 3.6.1.2.5

Number of design lanes: $N_{lanes} := 1$ *for traffic parallel to span AASHTO 3.6.1.2.6a

Multiple presence factor: m := 1.2 AASHTO Table 3.6.1.1.2-1

AASHTO 3.6.1.2.2 &

B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

10/28/2024



Live load per wheel for each vehicle type (looking transversely):	$P_t :=$	16 12.5 16 8.5 7.75 8.5 8.5 8.5 8.5 16.75 15.5	kip

AASHTO 3.6.1.2.2, AASHTO 3.6.1.2.3, & B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

Density of soil:

$$\rho_{\text{soil}} := 120 \ pcf$$

AASHTO Table 3.5.1-1

Density of pavement:

$$\rho_{\text{pave}} := 140 \ pcf$$

AASHTO Table 3.5.1-1

Design live lane load:

$$\omega_{lane} := 0.64 \ klf$$

AASHTO 3.6.1.2.4

Width of design live lane load:

$$w_{lane} := 10 ft$$

AASHTO 3.6.1.2.4

Determine Vehicle Loading:

Vehicle tire contact area:

$$A_{\text{tire}} := \frac{P_{\text{t}}}{P_{\text{tire}}} = \begin{vmatrix} 156.25 \\ 200.00 \\ 106.25 \\ 96.88 \\ 106.25 \\ 106.25 \\ 106.25 \\ 106.25 \\ 209.38 \\ 193.75 \end{vmatrix} in^{2}$$

200.00

20.00

B/M Pt. I - 7.2.5.9

Vehicle tire patch width:

$$v_{\text{patch}} := \frac{A_{\text{tire}}}{I_{\text{patch}}} = \begin{vmatrix} 15.63 \\ 20.00 \\ 10.63 \\ 9.69 \\ 10.63 \\ 10.63 \\ 10.63 \\ 10.63 \\ 20.94 \\ 19.38 \end{vmatrix} in$$

B/M Pt. I - 7.2.5.9 & AASHTO 3.6.1.2.5

Vehicle wheel load interaction depth transverse to culvert:

$$\mathbf{H}_{\text{int_t}} \coloneqq \frac{\mathbf{s}_{\text{wheel}} - \mathbf{w}_{\text{patch}} - 0.06 \ \mathbf{l}_{\text{span}}}{\text{LLDF}} = \begin{bmatrix} 3.30 \\ 2.99 \\ 3.66 \\ 3.73 \\ 3.66 \\ 3.66 \\ 3.66 \\ 3.66 \\ 3.66 \end{bmatrix} ft$$

2.99

2.923.03

AASHTO Eq. 3.6.1.2.6b-1

TEC
The Engineering Corp

Project: Manchester, MA - T1199.02

Vehicle wheel live load patch width at depth H:

$$\begin{split} \mathbf{w_{w.LL}_{i}} &\coloneqq \text{if } \mathbf{H_{cover}} \!<\! \mathbf{H_{int_t_{i}}} \\ & \qquad \qquad \| \mathbf{w_{patch}_{i}} \!+\! \mathbf{LLDF} \! \cdot \! \mathbf{H_{cover}} \! + 0.06 \ \mathbf{l_{span}} \\ & \qquad \qquad \text{else} \\ & \qquad \| \mathbf{w_{patch}_{i}} \!+\! \mathbf{s_{wheel}} \!+\! \mathbf{LLDF} \! \cdot \! \mathbf{H_{cover}} \! + 0.06 \ \mathbf{l_{span}} \end{split}$$

$$\mathbf{w}_{\text{w.LL}} = \begin{bmatrix} 4.14 \\ 3.78 \\ 4.14 \\ 3.36 \\ 3.28 \\ 3.36 \\ 3.36 \\ 3.36 \\ 3.36 \\ 4.22 \\ 4.09 \end{bmatrix} ft$$

$$H_{\text{int_p}} := \frac{s_{\text{axle}} - l_{\text{patch}}}{\text{LLDF}} = \begin{bmatrix} 11.45 \\ 2.75 \\ 11.45 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 12.32 \end{bmatrix} ft$$

$$AASHTO Eq. \ 3.6.1.2.6b-4$$

Vehicle axel load interaction depth parallel to culvert:

Vehicle live load patch length at depth H:

$$\begin{split} \mathbf{l_{w.LL}}_{i} &\coloneqq \text{if } \mathbf{H_{cover}} < \mathbf{H_{int_p_i}} \\ & \quad \| \mathbf{l_{patch}} + \mathbf{LLDF} \cdot \mathbf{H_{cover}} \\ & \quad \text{else} \\ & \quad \| \mathbf{l_{patch}} + \mathbf{s_{axle_i}} + \mathbf{LLDF} \cdot \mathbf{H_{cover}} \\ \\ & \quad \mathbf{l_{w.LL}} = \begin{bmatrix} 2.41 \\ 3.1 \\$$

AASHTO Eqs. 3.6.1.2.6b-5 & -6

Vehicle dynamic allowance for buried structures:

IM :=
$$\left(33 \cdot \left(1.0 - 0.125 \cdot \frac{H_{cover}}{ft}\right)\%\right) = 27.35\%$$

AASHTO Eq. 3.6.2.2-1



Vehicle live load rectangular patch area at depth H:	$\mathbf{A}_{\mathrm{LL}_{i}}\!:=\!\mathbf{w}_{\mathrm{w.LL}_{i}}\!\bullet\!\mathbf{l}_{\mathrm{w.LL}_{i}}\!=\!$	$\begin{bmatrix} 9.98 \\ 9.10 \\ 9.98 \\ 8.10 \\ 7.91 \\ 8.10 \\ 8.10 \\ 8.10 \\ 8.10 \\ 10.17 \\ 9.85 \end{bmatrix}$	AASHTO Eq. 3.6.1.2.6a-1
--	--	---	-------------------------

Vehicle live load vertical crown pressure:

$$P_{L} := \frac{P_{t} \cdot \left(1 + \frac{IM}{100\%}\right) \cdot m}{A_{LL}} = \begin{bmatrix} 2.10 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{bmatrix} ksf$$

$$AASHTO Eq. \ 3.6.1.2.6b-7$$

Design Loads:

Unfactored dead load crown pressure due to soil:

$$P_{EV} := H_{avg_soil} \cdot \rho_{soil} = 0.20 \text{ ksf}$$

Unfactored dead load crown pressure due to pavement:

$$P_{DW} := H_{pavement} \cdot \rho_{pave} = 0.06 \ ksf$$

Unfactored design lane load:

$$\mathbf{P}_{\mathrm{LL}_{i}} \coloneqq \mathbf{P}_{\mathrm{L}_{i}} + \mathbf{P}_{\mathrm{lane}_{i}} = \begin{bmatrix} 2.53 \\ 2.18 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{bmatrix} ksf$$

Load factors:

$$\gamma_{\rm EV} := 1.30$$

$$\gamma_{\rm DW} := 1.50$$

$$\gamma_{LL.inv} := 1.75$$

$$\gamma_{LL.opr} := 1.35$$

AASHTO Table 3.4.1-1, MBE Table 6A.4.2.2-1 & Table 6A.5.12.5-1

Factored dead load crown pressure due to soil:

$$P_{u.EV} := P_{EV} \cdot \gamma_{EV} = 0.26 \text{ ksf}$$

Factored dead load crown pressure due to pavement:

$$P_{u.DW} := P_{DW} \cdot \gamma_{DW} = 0.09 \text{ ksf}$$

$$P_{\text{u.LL.inv}_{i}} := P_{\text{LL}_{i}} \cdot \gamma_{\text{LL.inv}} = \begin{bmatrix} 4.42\\3.81\\4.29\\2.81\\2.62\\2.81\\2.81\\2.81 \end{bmatrix} ks$$

2.81 4.41 4.21

3.41 2.94

> 4.78 4.16 4.64

3.77 3.29 3.66

Factored inventory vehicle live load crown pressure:

$$P_{\text{u.LL.opr}_{i}} := P_{\text{LL}_{i}} \cdot \gamma_{\text{LL.opr}} = \begin{vmatrix} 3.31 \\ 2.17 \\ 2.02 \\ 2.17 \\ 2.17 \\ 2.17 \\ 2.17 \\ 3.40 \\ 3.25 \end{vmatrix} ksf$$

Combined factored loads when analyzing inventory vehicles:

$$P_{inv_{i}} := P_{u.EV} + P_{u.DW} + P_{u.LL.inv_{i}} = \begin{vmatrix} 3.16 \\ 2.98 \\ 3.16 \\ 3.16 \\ 3.16 \\ 3.16 \\ 4.76 \\ 4.56 \end{vmatrix} ksf$$

Combined factored loads when analyzing operating vehicles:

$$P_{opr_i} := P_{u.EV} + P_{u.DW} + P_{u.LL.opr_i} = \begin{vmatrix} 2.52 \\ 2.38 \\ 2.52 \\ 2.52 \\ 2.52 \\ 2.52 \\ 3.75 \\ 3.60 \end{vmatrix} ks_i$$

Determine Thrust:

Determine Thrust:		
Width of culvert on which live load is applied:	$C_{L_{i}} := min \left(l_{w.LL_{i}}, l_{span} \right) = \begin{bmatrix} 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \\ 2.41 \end{bmatrix} ft$ $F_{min} := max \left(\frac{15}{1000}, 1.00 \right) = 1.00$	AASHTO Eq. 12.7.2.2-2
F _{min} value:	$F_{\min} := \max \left(\frac{15}{12 \cdot \frac{l_{\text{span}}}{ft}}, 1.00 \right) = 1.00$	AASHTO Eq. 12.7.2.2-4
F ₁ value:	$\mathbf{F}_{1_{i}} \coloneqq \max \left(\frac{0.75 \cdot \mathbf{l}_{\text{span}}}{\mathbf{l}_{\text{w.LL}_{i}}}, \mathbf{F}_{\text{min}} \right) = \begin{bmatrix} 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \end{bmatrix}$	AASHTO Eq. 12.7.2.2-3
Factored thrust per unit length of wall due to dead load pressure caused by soil:	$T_{\text{L.EV}} := \frac{P_{\text{u.EV}} \cdot l_{\text{span}}}{2} = 1.97 \text{ klf}$	AASHTO Eq. 12.7.2.2-1 (modified)
Factored thrust per unit length of wall due to dead load pressure	$T_{L.DW} := \frac{P_{u.DW} \cdot l_{span}}{2} = 0.69 \ klf$	AASHTO Eq. 12.7.2.2-1 (modified)
Factored thrust per unit length of wall due to live load pressure caused by inventory vehicle:	$T_{\text{L.inv}_{i}} := \frac{P_{\text{u.LL.inv}_{i}} \cdot F_{1_{i}} \cdot C_{\text{L}_{i}}}{2} = \begin{bmatrix} 24.88 \\ 21.42 \\ 24.12 \\ 15.79 \\ 14.74 \\ 15.79 \\ 15.79 \\ 15.79 \\ 15.79 \\ 24.79 \\ 23.67 \end{bmatrix} klf$	AASHTO Eq. 12.7.2.2-1 (modified)
Factored thrust per unit length of wall due to live load pressure caused by operating vehicle:	$T_{\text{L.opr}_{i}} := \frac{P_{\text{u.LL.opr}_{i}} \cdot F_{1_{i}} \cdot C_{L_{i}}}{2} = \begin{bmatrix} 19.19 \\ 16.52 \\ 18.61 \\ 12.18 \\ 11.37 \\ 12.18 \\ 12.18 \\ 12.18 \\ 12.18 \\ 19.12 \\ 18.26 \end{bmatrix} klf$	AASHTO Eq. 12.7.2.2-1 (modified)



Total factored thrust with inventory design vehicle:
$$T_{\text{L.inv}_{i}} \coloneqq T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.inv}_{i}} = \begin{bmatrix} 27.54 \\ 24.08 \\ 26.78 \\ 18.45 \\ 17.40 \\ 18.45 \\ 18.45 \\ 18.45 \\ 18.45 \\ 27.44 \\ 26.32 \end{bmatrix} \text{ kly}$$

Total factored thrust with operating design vehicle:

$$T_{\text{T.opr}_{i}} := T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.opr}_{i}} = \begin{vmatrix} 21.27 \\ 14.84 \\ 14.03 \\ 14.84 \\ 14.84 \\ 14.84 \\ 14.84 \\ 21.78 \\ 20.92 \end{vmatrix} klf$$

21.85 19.18

Determine Resistance:

Critical buckling stress:

$$f_{cr} \coloneqq \text{if } l_{span} < \left(\frac{r_x}{k}\right) \cdot \sqrt{\frac{24 \cdot E_s}{F_u}} = 25.83 \text{ ksi} \qquad AASHTO Eq. 12.7.2.4-1 & 12.7.2.4-2$$

$$\left\| \frac{\left(\frac{F_u \cdot k \cdot l_{span}}{r_x}\right)^2}{48 \cdot E_s} \right\|$$

$$else$$

$$\left\| \frac{12 \cdot E_s}{\left(\frac{k \cdot l_{span}}{r_x}\right)^2} \right\|$$

The factored axial resistance per unit of wall:

$$R_{\text{n.axial}} := \left(\phi_b \cdot F_y \cdot A_s \right) \frac{1}{ft} = 52.78 \text{ klf}$$

AASHTO Eq. 12.7.2.3-1

The factored buckling resistance per unit of wall:

$$R_{n.buckling} := \phi_b \cdot \left(min \left(f_{cr}, F_y \right) \right) \cdot A_s \frac{1}{ft} = 41.32 \ \textit{klf}$$

AASHTO Eq. 12.7.2.3-1

The factored seam strength per unit of wall:

$$R_s := \phi_s \cdot F_{u_seam} = 75.04 \text{ klf}$$

AASHTO 12.7.2.5



Design Vehicle Rating Factors:

 $Wt_{v} := \begin{bmatrix} 36 \\ 25 \\ 20 \\ 25 \\ 36 \\ 27 \\ 31 \\ 34.75 \\ 38.75 \\ 28.75 \\ 43 \end{bmatrix} tony$

AASHTO 3.6.1.2.2 & B/M Pt. I Ch. 7

System factor:

vehicles:

Total weight of design

$$\phi_s := 1.00$$

MBE Table 6A4.2.4-.1

Condition factor:

$$\varphi_c := 1.00$$

*condition factor taken as 1.00 since B/M states that they shall not be used

MBE Table 6A4.2.3-.1 & B/M Pt. I 7.2.5.2A

Axial wall capacity:

$$C_{axial} := R_{n.axial} \cdot \varphi_s \cdot \varphi_c = 52.78 \ klf$$

MBE Eq. 6A4.2.1-2

Buckling capacity:

$$C_{buckling} := R_{n.buckling} \cdot \varphi_s \cdot \varphi_c = 41.32 \ klf$$

MBE Eq. 6A4.2.1-2

Seam strength capacity:

$$C_{\text{seam}} := R_s \cdot \varphi_s \cdot \varphi_c = 75.04 \ klf$$

MBE Eq. 6A4.2.1-2

Inventory load rating factor for axial wall resistance:

$$LR_{axial.inv} := \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{bmatrix} 2.34 \\ 2.08 \\ 3.17 \\ 3.40 \\ 3.17 \\ 3.17 \\ 3.17 \\ 2.02 \\ 2.12 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

Inventory load rating factor for buckling resistance:

$$LR_{buckling.inv} := \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{bmatrix} 1.60 \\ 2.45 \\ 2.62 \\ 2.45 \\ 2.45 \\ 2.45 \\ 2.45 \\ 1.56 \\ 1.63 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

1.80



Inventory load rating factor for seam strength:	$LR_{\text{seam.inv}} := \frac{C_{\text{seam}} - T_{\text{L.EV}} - T_{\text{L.DW}}}{T_{\text{L.inv}}} =$	2.91 3.38 3.00 4.58 4.91 4.58 4.58 4.58 4.58 2.92 3.06	MBE Eq. 6A4.2.1-1
---	--	--	-------------------

Operating load rating factor
$$LR_{axial.opr} := \frac{C_{axial} - C_{axial}}{C_{axial}}$$

 $LR_{axial.opr} := \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{vmatrix} 3.03 \\ 2.69 \\ 4.11 \\ 4.41 \\ 4.11 \\ 4.11 \\ 4.11 \\ 4.11 \\ 2.62 \\ 2.75 \end{vmatrix}$ $MBE Eq. \ 6A4.2.1-1$

2.01 2.34

2.61

Operating load rating factor for buckling resistance:
$$LR_{buckling.opr} \coloneqq \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{bmatrix} 2.08\\3.17\\3.40\\3.17\\3.17\\2.02\\2.12 \end{bmatrix}$$

Operating load rating factor for seam strength: $LR_{\text{seam.opr}} := \frac{C_{\text{seam}} - T_{\text{L.EV}} - T_{\text{L.DW}}}{T_{\text{L.opr}}} = \begin{bmatrix} 3.77 \\ 4.38 \\ 3.89 \\ 5.94 \\ 6.36 \\ 5.94 \\ 5.94 \\ 5.94 \\ 5.94 \\ 3.79 \\ 3.96 \end{bmatrix}$ $MBE \ Eq. \ 6A4.2.1-1$

MBE Eq. 6A4.2.1-1



Inventory load rating factor for axial wall resistance in tons:
$$TLR_{axial.inv_{i}} := LR_{axial.inv_{i}} \cdot Wt_{v_{i}} = \begin{bmatrix} 72.53\\58.50\\41.56\\79.34\\122.40\\85.69\\98.38\\110.29\\122.98\\58.14\\91.07 \end{bmatrix} tonf$$

Inventory load rating factor for buckling resistance in tons:

$$TLR_{buckling.inv_{i}} := LR_{buckling.inv_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 32.06 \\ 61.20 \\ 94.41 \\ 66.09 \\ 75.89 \\ 85.07 \\ 94.86 \\ 44.85 \\ 70.25 \end{vmatrix} tonf$$

45.12

Inventory load rating factor for seam strength in tons:

$$TLR_{\text{seam.inv}_{i}} := LR_{\text{seam.inv}_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 60.01 \\ 114.57 \\ 176.74 \\ 123.74 \\ 142.07 \\ 159.25 \\ 177.59 \\ 83.96 \\ 131.51 \end{vmatrix} tonf$$

Operating load rating factor for axial wall resistance in tons:

$$TLR_{axial.opr_{i}} := LR_{axial.opr_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 75.83 \\ 53.87 \\ 102.85 \\ 158.66 \\ 111.08 \\ 127.53 \\ 142.96 \\ 159.42 \\ 75.37 \\ 118.06 \end{vmatrix} tonf$$

10/28/2024



Operating load rating factor for buckling resistance in tons: $TLR_{buckling.opr_{i}} := LR_{buckling.opr_{i}} \cdot Wt_{v_{i}} = \begin{cases} 58.49 \\ 41.55 \\ 79.33 \\ 122.38 \\ 85.68 \\ 98.37 \\ 110.27 \\ 122.96 \\ 58.13 \\ 91.06 \end{cases} tonf$

Operating load rating factor for seam strength in tons:

$$\text{TLR}_{\text{seam.opr}_{i}} \coloneqq \text{LR}_{\text{seam.opr}_{i}} \cdot \text{Wt}_{\text{v}_{i}} = \begin{vmatrix} 109.51 \\ 77.79 \\ 148.52 \\ 229.11 \\ 160.40 \\ 184.16 \\ 206.44 \\ 230.20 \\ 108.83 \\ 170.48 \end{vmatrix} tonf$$

Summary Tables of Rating Factors:

Summary of Inventory Load Rating Factors.

 $\label{eq:heading_inv} \textit{Heading} \texttt{``Inv. Yielding''} \texttt{``Inv. Buckling''} \texttt{``Inv. Seam Strength''}]$

135.77

 $Table_{inv} := stack \left(Heading_{inv}, augment \left(VT_{matrix}, LR_{axial.inv}, LR_{buckling.inv}, LR_{seam.inv} \right) \right)$

14.34

	"Loading"	"Inv. Yielding"	"Inv. Buckling"	"Inv. Seam Strength"	
	"HL-93 Truck"	2.01	1.55	2.91	
	"HL-93 Tandem"	2.34	1.80	3.38	
	"H20"	2.08	1.60	3.00	
	"Type 3"	3.17	2.45	4.58	
Table _{inv} =	"Type 3S2"	3.40	2.62	4.91	
	"SU4"	3.17	2.45	4.58	
	"SU5"	3.17	2.45	4.58	
	"SU6"	3.17	2.45	4.58	
	"SU7"	3.17	2.45	4.58	
	"EV2"	2.02	1.56	2.92	
	"EV3"	2.12	1.63	3.06	

Summary of Operating Load Rating Factors.

Heading_{opr}:= ["Loading" "Opr. Yielding" "Opr. Buckling" "Opr. Seam Strength"]

 $Table_{opr} \coloneqq stack \left(Heading_{opr}, augment \left(VT_{matrix}, LR_{axial.opr}, LR_{buckling.opr}, LR_{seam.opr} \right) \right)$

	"Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"]
	"HL-93 Truck"	2.61	2.01	3.77
	"HL-93 Tandem"	3.03	2.34	4.38
	"H20"	2.69	2.08	3.89
	"Type 3"	4.11	3.17	5.94
Table _{opr} =	"Type 3S2"	4.41	3.40	6.36
	"SU4"	4.11	3.17	5.94
	"SU5"	4.11	3.17	5.94
	"SU6"	4.11	3.17	5.94
	"SU7"	4.11	3.17	5.94
	"EV2"	2.62	2.02	3.79
	"EV3"	2.75	2.12	3.96

Summary of Inventory Load Rating Factors in Tons.

Heading_{t.inv}:=["Loading" "Inv. Yielding" "Inv. Buckling" "Inv. Seam Strength"]

$$Table_{t.inv} \coloneqq stack \left(Heading_{t.inv}, augment \left(VT_{matrix}, \frac{TLR_{axial.inv}}{tonf}, \frac{TLR_{buckling.inv}}{tonf}, \frac{TLR_{seam.inv}}{tonf} \right) \right)$$

Table _{t.inv} =	"HL-93 Truck" "HL-93 Tandem" "H20" "Type 3" "Type 3S2" "SU4" "SU5" "SU6" "SU7" "EV2"	72.53 58.50 41.56 79.34 122.40 85.69 98.38 110.29 122.98 58.14	55.95 45.12 32.06 61.20 94.41 66.09 75.89 85.07 94.86 44.85	"Inv. Seam Strength" 104.74 84.48 60.01 114.57 176.74 123.74 142.07 159.25 177.59 83.96
	"EV2"	58.14	44.85	83.96
	"EV3"	91.07	70.25	131.51

Summary of Operating Load Rating Factors in Tons.

Heading_{t.opr}:= ["Loading" "Opr. Yielding" "Opr. Buckling" "Opr. Seam Strength"]

$$Table_{t.opr} := stack \left(Heading_{t.opr}, augment \left(VT_{matrix}, \frac{TLR_{axial.opr}}{tonf}, \frac{TLR_{buckling.opr}}{tonf}, \frac{TLR_{seam.opr}}{tonf} \right) \right)$$

	"Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"	
	"HL-93 Truck"	94.02	72.52	135.77	
	"HL-93 Tandem"	75.83	58.49	109.51	
	"H20"	53.87	41.55	77.79	
Table _{t.opr} =	"Type 3"	102.85	79.33	148.52	
	"Type 3S2"	158.66	122.38	229.11	
	"SU4"	111.08	85.68	160.40	
	"SU5"	127.53	98.37	184.16	
	"SU6"	142.96	110.27	206.44	
	"SU7"	159.42	122.96	230.20	
	"EV2"	75.37	58.13	108.83	
	"EV3"	118.06	91.06	170.48	





282 Merrimack Street Lawrence, MA 01843 Project: Manchester, MA - T1199.02

Calculated by: M. Martell

Date: 10/28/2024

Task: Structural Steel Plate Arch Culvert Rating

with 57% Section Loss

Client: Cell Signaling Technologies Inc. Checked by: E. Caron Date: 10/28/2024

Governing Specifications: AASHTO LRFD Bridge Design Specification, 9th Edition, 2020 (AASHTO)

MassDOT 2020 LRFD Bridge Manual 100th Anniversary Edition (B/M)

The Manual for Bridge Evaluation, Third Edition with 2019 Revisions (MBE)

Existing Bridge Plans, M-02-021, dated 1952 (Exist. Plans)

Test Boring Log by GZA GeoEnvironmental Inc., dated 9/3/2024 (Geotech)

<u>Design Methodology:</u> Load and Resistance Factor Design (LRFD)

Purpose: The following calculation is the load rating for the corrugated steel plate arch at Atwater Avenue over Sawmill Brook in Manchester, MA with 50% section loss in the thickness of the steel. The calculation below was done for all applicable design vehicles listed in Part I Chapter 7 of the MassDOT LRFD Bridge Manual.

General information:

Culvert crown elevation:
$$EL_{cul crown} := 147.08 \ ft$$
 Exist. Plans

Height of cover above top of
$$H_{cover} := EL_{road} - EL_{cul.crown} = 1.37 \text{ ft}$$
 crown:

Lowest elevation above the culvert crown:
$$EL_{low} := 148.33 \text{ } ft$$

Average height of soil:
$$H_{avg_soil} := \left(\frac{EL_{high} + EL_{low}}{2}\right) - EL_{cul.crown} = 1.68 ft$$

 $E_s := 29000 \ ksi$

 $EL_{high} := 149.19 \, ft$

Pavement thickness:
$$H_{pavement} := 0.44 \, ft$$
 Geotech

Span length:
$$l_{snan} := 15 \text{ ft}$$
 Exist. Plans

Height of opening:
$$h_{open} := 79 \text{ in}$$
 Exist. Plans

Thickness of original corrugated
$$t_0 := 0.218 \text{ in}$$
 steel:

43% of the thickness of original $t_0 := 0.43 \cdot t_0 = 0.09$ in corrugated steel:

Bolted seam strength:
$$F_{u \text{ seam}} := 112 \text{ klf}$$
 *For 3/4-inch diameter bolt AASHTO Table A12-8

Minimum tensile stress of the
$$F_u := 45 \text{ ksi}$$
 AASHTO Table A12-10

Original cross sectional area
$$A_{s,o} := 3.199 \text{ in}^2$$

AASHTO Table A12-3

of pipe:

Original moment of inertia of

$$I_{x,o} := 0.1269 \frac{in^4}{in}$$

*For 6" x 2" corrugations

AASHTO Table A12-3

Original radius of gyration of the corrugation:

$$r_{x.o} := 0.690 \text{ in}$$

AASHTO Table A12-3

Print Date:

10/28/2024

Modulus of elasticity of steel:

AASHTO Table A12-10

Exist. Plans

Exist. Plans



Ratio of remaining thickness to original thickness:

$$\rho_t := \frac{t_c}{t_o} = 0.43$$

Assuming that the remaining section's cross sectional area, moment of inertia, and radius of gyration can be approximated by multiplying the original section properties by the ratio of the remaining steel thickness to original thickness.

Cross sectional area of

$$A_s := A_{s.o} \cdot \rho_t = 1.376 \ in^2$$

Exist. Plans

remaining pipe:

Moment of inertia of remaining pipe:

 $I_x := I_{x.o} \cdot \rho_t = 0.055 \frac{in^4}{in}$

Exist. Plans

Radius of gyration of the remaining corrugation:

 $r_x := r_{x.o} \cdot \rho_t = 0.297$ in

Exist. Plans

Resistance factor for buried structures for wall area and

buckling:

 $\phi_b := 1.00$

AASHTO Table 12.5.5-1

Resistance factor for buried structures for seam strength: $\phi_s := 0.67$

AASHTO Table 12.5.5-1

Soil Stiffness Factor:

k := 0.22

AASHTO 12.7.2.4

Design Vehicle Properties:

Live Load Distribution Factor transverse or parallel to span:

LLDF := 1.15

*for all buried structures excluding

AASHTO Table 3.6.1.2.6a-1

concrete pipes

"EV3"

 $i := 1 \dots rows (VT_{matrix})$

Print Date:

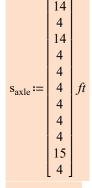
Wheel spacing for each vehicle type:

Vehicle type matrix title:

 $s_{\text{wheel}} := 6 ft$

AASHTO 3.6.1.2.6a-b

Axle spacing for each vehicle type:



*Assume most narrow axle spacing for

AASHTO 3.6.1.2.2 & B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

loading conditions.

Assumed tire pressure:

 $p_{tire} := 80 \ psi$

B/M Pt. I - 7.2.5.9

Tire patch length:

 $l_{\text{patch}} := 10 \ in$

B/M Pt. I - 7.2.5.9 & AASHTO 3.6.1.2.5

Number of design lanes:

 $N_{lanes} := 1$

*for traffic parallel to span

AASHTO 3.6.1.2.6a

Multiple presence factor:

m := 1.2

AASHTO Table 3.6.1.1.2-1

10/28/2024



Live load per wheel for each vehicle type (looking transversely):	$P_t :=$	16 12.5 16 8.5 7.75 8.5 8.5 8.5 16.75 15.5	kip

AASHTO 3.6.1.2.2, AASHTO 3.6.1.2.3, & B/M Pt. I Ch. 7 Figures 7.52-10A, -11, & -12

Density of soil:

$$\rho_{\text{soil}} := 120 \ pcf$$

AASHTO Table 3.5.1-1

Density of pavement:

$$\rho_{\text{pave}} := 140 \ pcf$$

AASHTO Table 3.5.1-1

Design live lane load:

$$\omega_{lane} := 0.64 \ klf$$

AASHTO 3.6.1.2.4

Width of design live lane load:

$$w_{lane} := 10 ft$$

AASHTO 3.6.1.2.4

Determine Vehicle Loading:

$$A_{\text{tire}} := \frac{P_{\text{t}}}{P_{\text{tire}}} = \begin{vmatrix} 156.25 \\ 200.00 \\ 106.25 \\ 96.88 \\ 106.25 \\ 106.25 \\ 106.25 \\ 106.25 \\ 209.38 \\ 193.75 \end{vmatrix} in^{2}$$

200.00

20.00

B/M Pt. I - 7.2.5.9

Vehicle tire patch width:

$$w_{\text{patch}} \coloneqq \frac{A_{\text{tire}}}{l_{\text{patch}}} = \begin{vmatrix} 15.63 \\ 20.00 \\ 10.63 \\ 9.69 \\ 10.63 \\ 10.63 \\ 10.63 \\ 10.63 \\ 20.94 \\ 19.38 \end{vmatrix} in$$

B/M Pt. I - 7.2.5.9 & AASHTO 3.6.1.2.5

Vehicle wheel load interaction depth transverse to culvert:

$$H_{\text{int_t}} := \frac{s_{\text{wheel}} - w_{\text{patch}} - 0.06 \ l_{\text{span}}}{\text{LLDF}} = \begin{vmatrix} 2.99 \\ 3.66 \\ 3.73 \\ 3.66 \\ 3.66 \\ 3.66 \\ 3.66 \end{vmatrix} ft$$

2.99 3.30

2.92 3.03 AASHTO Eq. 3.6.1.2.6b-1

TEC
The Engineering Corp

Project: Manchester, MA - T1199.02

Vehicle wheel live load patch width at depth H:

AASHTO Eqs. 3.6.1.2.6b-2 & -3

$$\mathbf{w_{w.LL}} = \begin{bmatrix} 4.14 \\ 3.78 \\ 4.14 \\ 3.36 \\ 3.28 \\ 3.36 \\ 3.36 \\ 3.36 \\ 3.36 \\ 4.22 \\ 4.09 \end{bmatrix} \mathit{ft}$$

Vehicle axel load interaction depth parallel to culvert:

$$H_{\text{int_p}} := \frac{s_{\text{axle}} - l_{\text{patch}}}{\text{LLDF}} = \begin{bmatrix} 2.73 \\ 11.45 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 2.75 \\ 12.32 \\ 2.75 \end{bmatrix} ft$$

AASHTO Eq. 3.6.1.2.6b-4

Vehicle live load patch length at depth H:

AASHTO Eqs. 3.6.1.2.6b-5 & -6

Vehicle dynamic allowance for buried structures:

IM :=
$$\left(33 \cdot \left(1.0 - 0.125 \cdot \frac{H_{cover}}{ft}\right)\%\right) = 27.35\%$$

AASHTO Eq. 3.6.2.2-1



Vehicle live load rectangular patch area at depth H:	$\mathbf{A_{LL}}_{i}\!:=\!\mathbf{w_{w.LL}}_{i}\!\bullet\!\mathbf{l_{w.LL}}_{i}\!=\!$	$\begin{bmatrix} 9.98 \\ 9.10 \\ 9.98 \\ 8.10 \\ 7.91 \\ 8.10 \\ 8.10 \\ 8.10 \\ 8.10 \\ 8.10 \\ 10.17 \\ 9.85 \end{bmatrix}$	AASHTO Eq. 3.6.1.2.6a-1
--	---	---	-------------------------

Vehicle live load vertical crown pressure:

$$P_{L} := \frac{P_{t} \cdot \left(1 + \frac{IM}{100\%}\right) \cdot m}{A_{LL}} = \begin{bmatrix} 2.10 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{bmatrix} ksf$$

$$AASHTO Eq. \ 3.6.1.2.6b-7$$

Design Loads:

Unfactored dead load crown pressure due to soil:

$$P_{EV} := H_{avg_soil} \cdot \rho_{soil} = 0.20 \text{ ksf}$$

Unfactored dead load crown pressure due to pavement:

$$P_{DW} := H_{pavement} \cdot \rho_{pave} = 0.06 \ ksf$$

Unfactored design lane load:

$$P_{LL_{i}} := P_{L_{i}} + P_{lane_{i}} = \begin{bmatrix} 2.53 \\ 2.18 \\ 2.45 \\ 1.60 \\ 1.50 \\ 1.60 \\ 1.60 \\ 1.60 \\ 1.60 \\ 2.52 \\ 2.40 \end{bmatrix} ksf$$

Print Date:



Load factors:

$$\gamma_{EV} := 1.30$$

$$\gamma_{\rm DW} := 1.50$$

$$\gamma_{\text{LL.inv}} := 1.75$$

$$\gamma_{LL.opr} := 1.35$$

AASHTO Table 3.4.1-1, MBE Table 6A.4.2.2-1 & Table 6A.5.12.5-1

Factored dead load crown pressure due to soil:

$$P_{u.EV} := P_{EV} \cdot \gamma_{EV} = 0.26 \text{ ksf}$$

Factored dead load crown pressure due to pavement:

$$P_{u.DW} := P_{DW} \cdot \gamma_{DW} = 0.09 \text{ ksf}$$

$$P_{u.LL.inv_{i}} := P_{LL_{i}} \cdot \gamma_{LL.inv} = \begin{bmatrix} 4.42 \\ 3.81 \\ 4.29 \\ 2.81 \\ 2.62 \\ 2.81 \\ 2.81 \\ 2.81 \end{bmatrix} ks$$

2.81 4.41 4.21

3.41 2.94

> 4.78 4.16 4.64

3.77 3.29 3.66

Factored inventory vehicle live load crown pressure:

$$P_{u.LL.opr_{i}} := P_{LL_{i}} \cdot \gamma_{LL.opr} = \begin{vmatrix} 3.31 \\ 2.17 \\ 2.02 \\ 2.17 \\ 2.17 \\ 2.17 \\ 2.17 \\ 3.40 \\ 3.25 \end{vmatrix} ksf$$

Combined factored loads when analyzing inventory vehicles:

$$P_{inv_{i}} := P_{u.EV} + P_{u.DW} + P_{u.LL.inv_{i}} = \begin{vmatrix} 3.16 \\ 2.98 \\ 3.16 \\ 3.16 \\ 3.16 \\ 3.16 \\ 4.76 \\ 4.56 \end{vmatrix} ksf$$

Combined factored loads when analyzing operating vehicles:

$$P_{opr_i} := P_{u.EV} + P_{u.DW} + P_{u.LL.opr_i} = \begin{vmatrix} 2.52 \\ 2.38 \\ 2.52 \\ 2.52 \\ 2.52 \\ 2.52 \\ 3.75 \\ 3.60 \end{vmatrix} ks$$

Determine Thrust:

Width of culvert on which live C load is applied:	$\mathbf{l}_{\mathbf{L}_{i}} := min\left(\mathbf{l}_{\mathbf{w.LL}_{i}}, \mathbf{l}_{\mathbf{span}}\right) = \mathbf{l}_{i}$	[2.41] [2.41] <td< th=""><th>AASHTO Eq. 12.7.2.2-2</th></td<>	AASHTO Eq. 12.7.2.2-2
---	--	---	-----------------------

F_{min} value:
$$F_{min} := max \left(\frac{15}{12 \cdot \frac{l_{span}}{ft}}, 1.00 \right) = 1.00$$
 AASHTO Eq. 12.7.2.2-4

F₁ value:
$$F_{1_{i}} := \max \left(\frac{0.75 \cdot l_{span}}{l_{w.LL_{i}}}, F_{min} \right) = \begin{pmatrix} 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \\ 4.67 \end{pmatrix}$$

Factored thrust per unit length of wall due to dead load pressure caused by soil:

$$T_{L.EV} := \frac{P_{u.EV} \cdot l_{span}}{2} = 1.97 \ klf$$

AASHTO Eq. 12.7.2.2-1 (modified)

Factored thrust per unit length of wall due to dead load pressure caused by pavement:

$$T_{L.DW} := \frac{P_{u.DW} \cdot l_{span}}{2} = 0.69 \ klf$$

AASHTO Eq. 12.7.2.2-1 (modified)

Factored thrust per unit length of wall due to live load pressure caused by inventory vehicle:

$$T_{L.inv_{i}} := \frac{P_{u.LL.inv_{i}} \cdot F_{1_{i}} \cdot C_{L_{i}}}{2} = \begin{vmatrix} 21.42 \\ 24.12 \\ 15.79 \\ 14.74 \\ 15.79 \\ 15.79 \\ 15.79 \\ 15.79 \\ 24.79 \\ 23.67 \end{vmatrix} klf$$

AASHTO Eq. 12.7.2.2-1 (modified)

Factored thrust per unit length of wall due to live load pressure caused by operating vehicle:

$$T_{L.opr_{i}} := \frac{P_{u.LL.opr_{i}} \cdot F_{1_{i}} \cdot C_{L_{i}}}{2} = \begin{bmatrix} 16.52\\18.61\\12.18\\11.37\\12.18\\12.18\\12.18\\12.18\\12.18 \end{bmatrix} klf$$

19.19

19.12 18.26 AASHTO Eq. 12.7.2.2-1 (modified)



	27.54 24.08 26.78 18.45	
Total factored thrust with inventory design vehicle:	$T_{\text{T.inv}_{i}} := T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.inv}_{i}} = \begin{bmatrix} 24.08 \\ 26.78 \\ 18.45 \\ 17.40 \\ 18.45 \\ 18.45 \\ 18.45 \\ 18.45 \\ 27.44 \\ 26.32 \end{bmatrix}$	klf
	[26.32]	

Total factored thrust with operating design vehicle:

$$T_{\text{T.opr}_{i}} := T_{\text{L.EV}} + T_{\text{L.DW}} + T_{\text{L.opr}_{i}} = \begin{bmatrix} 21.27 \\ 14.84 \\ 14.03 \\ 14.84 \\ 14.84 \\ 14.84 \\ 14.84 \\ 21.78 \\ 20.92 \end{bmatrix} klf$$

Determine Resistance:

Critical buckling stress:

$$\begin{split} f_{cr} \coloneqq & \text{if } l_{span} < \left(\frac{r_x}{k}\right) \cdot \sqrt{\frac{24 \cdot E_s}{F_u}} \right| = 19.54 \text{ ksi} \\ & \left\| F_u - \frac{\left(\frac{F_u \cdot k \cdot l_{span}}{r_x}\right)^2}{48 \cdot E_s} \right\| \\ & \text{else} \\ & \left\| \frac{12 \cdot E_s}{\left(\frac{k \cdot l_{span}}{r_x}\right)^2} \right| \end{split}$$

The factored axial resistance per unit of wall:

$$R_{\text{n.axial}} := (\phi_b \cdot F_y \cdot A_s) \frac{1}{ft} = 45.39 \text{ klf}$$

AASHTO Eq. 12.7.2.3-1

The factored buckling resistance per unit of wall:

$$R_{n.buckling} := \phi_b \cdot \left(min\left(f_{cr}, F_y\right) \right) \cdot A_s \frac{1}{\mathit{ft}} = 26.87 \ \mathit{klf}$$

AASHTO Eq. 12.7.2.3-1

The factored seam strength per unit of wall:

$$R_s := \phi_s \cdot F_{u_seam} = 75.04 \text{ klf}$$

AASHTO 12.7.2.5



Design Vehicle Rating Factors:

 $Wt_{v} := \begin{bmatrix} 36 \\ 25 \\ 20 \\ 25 \\ 36 \\ 27 \\ 31 \\ 34.75 \\ 38.75 \\ 28.75 \\ 43 \end{bmatrix} tony$

AASHTO 3.6.1.2.2 & B/M Pt. I Ch. 7

System factor:

vehicles:

Total weight of design

$$\phi_s := 1.00$$

MBE Table 6A4.2.4-.1

Condition factor:

 $\varphi_c := 1.00$

*condition factor taken as 1.00 since B/M states that they shall not be used

2.00

1.13

MBE Table 6A4.2.3-.1 & B/M Pt. I 7.2.5.2A

Axial wall capacity:

$$C_{axial} := R_{n.axial} \cdot \varphi_s \cdot \varphi_c = 45.39 \ klf$$

MBE Eq. 6A4.2.1-2

Buckling capacity:

$$C_{buckling} := R_{n.buckling} \cdot \varphi_s \cdot \varphi_c = 26.87 \ klf$$

MBE Eq. 6A4.2.1-2

Seam strength capacity:

$$C_{\text{seam}} := R_s \cdot \varphi_s \cdot \varphi_c = 75.04 \text{ klf}$$

MBE Eq. 6A4.2.1-2

Inventory load rating factor for axial wall resistance:

$$LR_{axial.inv} \coloneqq \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{bmatrix} 1.77 \\ 2.71 \\ 2.90 \\ 2.71 \\ 2.71 \\ 2.71 \\ 2.71 \\ 1.72 \\ 1.81 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

Inventory load rating factor for buckling resistance:

$$LR_{buckling.inv} := \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{vmatrix} 1.00 \\ 1.53 \\ 1.64 \\ 1.53 \\ 1.53 \\ 1.53 \\ 1.53 \\ 0.98 \\ 1.02 \end{vmatrix}$$

MBE Eq. 6A4.2.1-1



Inventory load rating factor for seam strength: $LR_{seam.inv} \coloneqq \frac{C_{seam} - T_{L.EV} - T_{L.DW}}{T_{L.inv}} = \begin{bmatrix} 3\\ 4\\ 4\\ 4\\ 4\\ 4 \end{bmatrix}$	2.91 3.38 3.00 4.58 4.91 4.58 4.58 4.58 4.58 4.58 2.92 3.06
--	--

2.23 2.59

> 1.26 1.47

3.77 4.38

$$LR_{axial.opr} := \frac{C_{axial} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{bmatrix} 2.30 \\ 3.51 \\ 3.76 \\ 3.51 \\ 3.51 \\ 3.51 \\ 2.24 \\ 2.34 \end{bmatrix}$$

$$LR_{buckling.opr} := \frac{C_{buckling} - T_{L.EV} - T_{L.DW}}{T_{L.opr}} = \begin{bmatrix} 1.30 \\ 1.99 \\ 2.13 \\ 1.99 \\ 1.99 \\ 1.99 \\ 1.99 \\ 1.27 \\ 1.33 \end{bmatrix}$$

Operating load rating factor for seam strength:

$$LR_{\text{seam.opr}} := \frac{C_{\text{seam}} - T_{\text{L.EV}} - T_{\text{L.DW}}}{T_{\text{L.opr}}} = \begin{bmatrix} 3.89 \\ 5.94 \\ 6.36 \\ 5.94 \\ 5.94 \\ 5.94 \\ 5.94 \\ 3.79 \\ 3.96 \end{bmatrix}$$

MBE Eq. 6A4.2.1-1

28.26

64.65

Inventory load rating factor for axial wall resistance in tons:
$$TLR_{axial.inv_{i}} := LR_{axial.inv_{i}} \cdot Wt_{v_{i}} = \begin{cases} 01.84 \\ 49.88 \\ 35.43 \\ 67.64 \\ 104.35 \end{cases}$$

$$73.06 \\ 83.88 \\ 94.03 \\ 104.85 \\ 49.57 \\ 77.65 \end{cases}$$

Inventory load rating factor for buckling resistance in tons:

$$TLR_{buckling.inv_{i}} := LR_{buckling.inv_{i}} \cdot Wt_{v_{i}} = \begin{cases} 20.08 \\ 38.33 \\ 59.13 \\ 41.39 \\ 47.53 \\ 53.28 \\ 59.41 \\ 28.09 \end{cases}$$

$$ton$$

Inventory load rating factor for seam strength in tons:
$$TLR_{seam.inv_{i}} := LR_{seam.inv_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 60.01 \\ 114.57 \\ 176.74 \\ 123.74 \\ 142.07 \\ 159.25 \\ 177.59 \\ 83.96 \\ 131.51 \end{vmatrix} tonf$$

Print Date:

Operating load rating factor for buckling resistance in tons: $TLR_{buckling.opr_{i}} := LR_{buckling.opr_{i}} \cdot Wt_{v_{i}} = \begin{bmatrix} 36.63 \\ 26.02 \\ 49.68 \\ 76.64 \\ 53.66 \\ 61.61 \\ 69.06 \\ 77.01 \\ 36.41 \\ 57.03 \end{bmatrix} tonf$

 $TLR_{seam.opr_{i}} := LR_{seam.opr_{i}} \cdot Wt_{v_{i}} = \begin{vmatrix} 109.51 \\ 77.79 \\ 148.52 \\ 229.11 \\ 160.40 \\ 184.16 \\ 206.44 \\ 230.20 \\ 108.83 \end{vmatrix} tonf$

Operating load rating factor for seam strength in tons:

Summary Tables of Rating Factors:

Summary of Inventory Load Rating Factors.

Heading_{inv}:=["Loading" "Inv. Yielding" "Inv. Buckling" "Inv. Seam Strength"]

170.48

135.77

 $Table_{inv} := stack \left(Heading_{inv}, augment \left(VT_{matrix}, LR_{axial.inv}, LR_{buckling.inv}, LR_{seam.inv} \right) \right)$

45.42

Table _{inv} =	"HL-93 Truck" "HL-93 Tandem" "H20" "Type 3" "Type 3S2" "SU4" "SU5" "SU6" "SU6"	1.72 2.00 1.77 2.71 2.90 2.71 2.71 2.71 2.71	0.97 1.13 1.00 1.53 1.64 1.53 1.53 1.53	"Inv. Seam Strength" 2.91 3.38 3.00 4.58 4.91 4.58 4.58 4.58 4.58
	"EV3"	1.81	1.02	3.06

Summary of Operating Load Rating Factors.

 $Heading_{opr} \coloneqq \begin{bmatrix} \text{``Loading''} & \text{``Opr. Yielding''} & \text{``Opr. Buckling''} & \text{``Opr. Seam Strength''} \end{bmatrix}$

 $Table_{opr} := stack \left(Heading_{opr}, augment \left(VT_{matrix}, LR_{axial.opr}, LR_{buckling.opr}, LR_{seam.opr} \right) \right)$

-	, -	•	-		
	["Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"	
	"HL-93 Truck"	2.23	1.26	3.77	
	"HL-93 Tandem"	2.59	1.47	4.38	
	"H20"	2.30	1.30	3.89	
	"Type 3"	3.51	1.99	5.94	
Toble -	"Type 3S2"	3.76	2.13	6.36	
Table _{opr} =	"SU4"	3.51	1.99	5.94	
	"SU5"	3.51	1.99	5.94	
	"SU6"	3.51	1.99	5.94	
	"SU7"	3.51	1.99	5.94	
	"EV2"	2.24	1.27	3.79	
	"EV3"	2.34	1.33	3.96	

Print Date:

Summary of Inventory Load Rating Factors in Tons.

Heading_{t.inv}:=["Loading" "Inv. Yielding" "Inv. Buckling" "Inv. Seam Strength"]

$$\mathsf{Table}_{\mathsf{t.inv}} \coloneqq \mathsf{stack}\left(\mathsf{Heading}_{\mathsf{t.inv}}, \mathsf{augment}\left(\mathsf{VT}_{\mathsf{matrix}}, \frac{\mathsf{TLR}_{\mathsf{axial.inv}}}{\mathit{tonf}}, \frac{\mathsf{TLR}_{\mathsf{buckling.inv}}}{\mathit{tonf}}, \frac{\mathsf{TLR}_{\mathsf{seam.inv}}}{\mathit{tonf}}\right)\right)$$

	F			7
	"Loading"	"Inv. Yielding"	"Inv. Buckling"	"Inv. Seam Strength"
	"HL-93 Truck"	61.84	35.04	104.74
	"HL-93 Tandem"	49.88	28.26	84.48
	"H20"	35.43	20.08	60.01
	"Type 3"	67.64	38.33	114.57
Table _{t.inv} =	"Type 3S2"	104.35	59.13	176.74
rabic _{t.inv} —	"SU4"	73.06	41.39	123.74
	"SU5"	83.88	47.53	142.07
	"SU6"	94.03	53.28	159.25
	"SU7"	104.85	59.41	177.59
	"EV2"	49.57	28.09	83.96
	"EV3"	77.65	43.99	131.51

Summary of Operating Load Rating Factors in Tons.

 $Heading_{t.opr} \coloneqq \begin{bmatrix} \text{``Loading'' '`Opr. Yielding'' '`Opr. Buckling'' '`Opr. Seam Strength''} \end{bmatrix}$

$$Table_{t.opr} \coloneqq stack \left(Heading_{t.opr}, augment \left(VT_{matrix}, \frac{TLR_{axial.opr}}{tonf}, \frac{TLR_{buckling.opr}}{tonf}, \frac{TLR_{seam.opr}}{tonf} \right) \right)$$

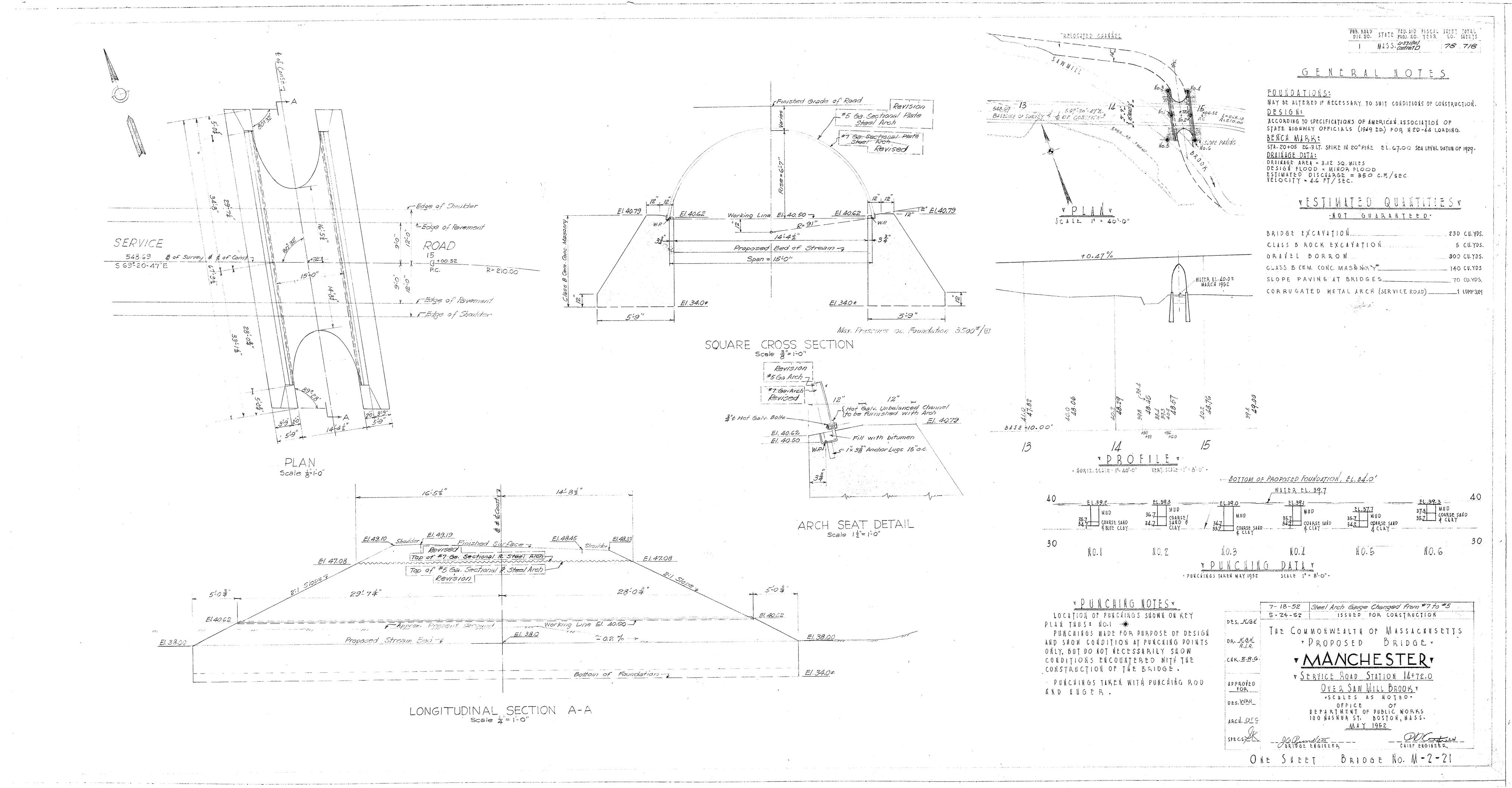
	"Loading"	"Opr. Yielding"	"Opr. Buckling"	"Opr. Seam Strength"]
	"HL-93 Truck"	80.16	45.42	135.77
	"HL-93 Tandem"	64.65	36.63	109.51
	"H20"	45.93	26.02	77.79
	"Type 3"	87.69	49.68	148.52
Table _{t.opr} =	"Type 3S2"	135.27	76.64	229.11
Table _{t.opr} —	"SU4"	94.70	53.66	160.40
	"SU5"	108.73	61.61	184.16
	"SU6"	121.89	69.06	206.44
	"SU7"	135.92	77.01	230.20
	"EV2"	64.26	36.41	108.83
	"EV3"	100.65	57.03	170.48

Print Date:

10/28/2024

Appendix B - Design References

- I. Proposed Bridge Plans Service Road Sation 14+72.0 Over Saw Mill Brook, dated May 1952, one (1) page(s).
- II. MassDOT Structures Inspection Field Report dated October 1, 2024, nine (9) page(s).
- III. Test Boring Log by GZA GeoEnvironmental Inc., dated September 3, 2024, four (4) page(s).
- IV. AASHTO LRFD Bridge Design Specification, 9th Edition, 2020, eighteen (18) page(s).
- V. MassDOT 2020 LRFD Bridge Manual 100th Anniversary Edition, six (6) page(s).
- VI. The Manual for Bridge Evaluation, Third Edition with 2019 Revisions, four (4) page(s).



MASSACHUSETTS DEPARTMENT OF TRANSPORTATION PAGE 1 OF 8

STRUCTURES INSPECTION FIELD REPORT

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B.I.N.

BR. DEPT. NO. M-02-021

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MANCHESTER	8AL	M-02-021	M02021-8AL-MUN-BRI	OCT 1, 2024

REMARKS, PHOTOS & SKETCHES

BRIDGE ORIENTATION

The bridge M-02-021 (8AL) carries Atwater Avenue over Saw Mill Brook in the Town of Manchester, Massachusetts. The abutments are labeled east and west and Saw Mill Brook flows from north to south.

GENERAL REMARKS

Note: While the only plans on record are from 1952 and the corrugated arch has since been replaced, it appears as though the new corrugated arch is the same size and of the same gauge steel.

ITEM 62 - CULVERT

Item 62.6 - Pipe

The corrugated structural steel arch has an original thickness of 7/32" (5 Gauge).

The bottom of both sides of the arch have full length x up to 8" high minor laminar rust with minor section loss / pitting, up to 1/16" deep (5/32" remaining), heaviest at the interface with the abutment / footing (See Photos 1 & 2).

Item 62.8 - Embankment

The top of both embankments have large placed cut granite blocks as embankment protection.

The top east end of the south embankment has previous settlement of several blocks that has since been stabilized with gravel and crushed stone. There are a few random voids between the stones, up to 1' diameter with penetrations of up to 2' deep (See Photo 3).

CONDITION RATING GUIDE

	CODE	CONDITION	DEFECTS
	N	NOT APPLICABLE	Use if structure is not a culvert.
G	9	EXCELLENT	No deficiencies.
G	8	VERY GOOD	No noticeable or noteworthy differences which affect the condition of the culvert. Insignificant scrape marks caused by drift.
G	7	GOOD	Shrinkage cracks, light scaling, and insignificant spalling, which does not expose reinforcing steel. Insignificant damage caused by drift with not misalignment and not requiring corrective action. Some minor scouring has occurred near curtain walls, wingwalls, or pipes. Metal culverts have a smooth symmetrical curvature with superficial corrosion and no pitting.
F	6	SATISFACTORY	Deterioration or initial disintegration, minor chloride contamination, cracking with some leaching, or spalls on concrete or masonry walls and slabs. Local minor scouring at curtain walls, wingwalls, or pipes. Metal culverts have a smooth curvature, non-symmetrical shape, significant corrosion or moderate pitting.
F	5	FAIR	Moderate to major deterioration, or disintegration, extensive cracking and leaching, or spalls on concrete or masonry walls and slabs. Minor settlement or misalignment. Noticeable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection in one section, significant corrosion or deep pitting.
Р	4	POOR	Large spalls, heavy scaling, wide cracks, considerable efforescence, or opened construction joints permitting loss of backfill. Considerable settlement or misalignment. Considerable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection throughout, extensive corrosion or deep pitting.
Р	3	SERIOUS	Any condition described in Code 4 but which is excessive in scope. Severe movement or differential settlement of the segments, or loss of fill. Holes may exist in walls or slabs. Integral wingwalls, nearly severed from culvert. Severe scour or erosion at curtain walls, wingwalls, or pipes. Metal culverts have extreme distortion and deflection in one section, extensive corrosion, or deep pitting with scattered perforations.
С	2	CRITICAL	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
С	1	"IMMINENT" FAILURE	Bridge closed. Corrective action may put back in light service.
	0	FAILED	Bridge closed. Replacement necessary.

DEFICIENCY REPORTING GUIDE

DEFICIENCY: A defect in a structure that requires corrective action

CATEGORIES OF DEFICIENCIES:

M= Minor Deficiency (Examples include but are not limited to: Spalled concrete, minor to moderate corrosion to steel culverts, minor settlement or misalignment, minor scouring, minor damage to guardrail, etc.)

S = Severe/Major Deficiency(Examples include but are not limited to: Large spalls, wide cracks, moderate to major deterioration in concrete, considerable settlement, considerable scouring or undermining, etc.)

A deficiency in a structural component or element of a bridge that poses an extreme hazard or unsafe condition to the public. (Follow-up Critical Deficiency Report must be submitted C-S= Critical Deficiency separately)

URGENCY OF REPAIR:

[Inspector(s) stay at the bridge until the District Maintenance crew or the responsible Agency crew(if not a State bridge) show up and corrective action is taken.] I = Immediate-(Action will be taken by the District Maintenance Engineer or the Responsible Agency (if not a State owned bridge) upon receipt of the Inspection Report). A = ASAP-

P = Prioritize-[Shall be prioritized by District Maintenance Engineer or the Responsible Party (if not a State owned bridge) and repairs made when funds and/or manpower is available].

REMARKS

The bottom of the north embankment along the east side of the culvert has a 3' wide x 1' high x up to 1' deep area of undermining of the bottom stone.

Item 62.9 - Wearing Surface

The bituminous concrete wearing surface and both approaches have scattered minor longitudinal and transverse cracks up to 1/8" wide.

The eastbound lane has a full length x up to 2' wide area of minor longitudinal cracking (cracks up to 1/4" wide and some sealed) and settlement that continues into both approaches (See Photo 4).

Item 62.10 - Railing

There is single panel w-beam guardrail along both sides of the culvert with steel posts and no offset blocks.

The south rail has a 5' long area of minor damage to the top at the east end (See Photo 5).

Item 62.17 - Abutment / Footing

The bottom several inches of the concrete abutments / footings are hidden under the water surface.

The exposed portion has moderate scaling throughout.

The top of the west abutment / footing has a 20" long x up to 4" wide x up to 3" deep spall located 7' from the north end (See Photo 6).

ITEM 61 - CHANNEL AND CHANNEL PROTECTION

Item 61.2 - Embankment Erosion

All channel embankments have minor erosion.

Item 61.3 - Debris

There is a full channel width x up to 2' high pile of debris and tree branches within the culvert, near the north end (See Photo 7).

<u>Item 61.6 - Rip-Rap/Slope Protection</u>

See Item 62.8 - Embankment.

Item 61.7 - Aggradation

There is a 5' wide x up to 2' high area of aggradation at the southeast corner of the culvert (See Photo 8).

APPROACHES

Approaches a - Appr. Pavement Condition

See Item 62.9 - Wearing Surface.

TRAFFIC SAFETY

Item 36a - Bridge Railing

See Item 62.10 - Railing.

Item 36c - Approach Guardrail

The approach guardrail is w-beam panels on steel posts with no offset blocks.

REMARKS

The northwest, southwest and southeast approach rails have random minor dents throughout.

Item 36d - Approach Guardrail Ends

The southwest corner has a boxing glove end that is turned from traffic.

The northwest corner has a boxing glove end that is not turned from traffic and has minor dents and surface rust.

The southeast and northeast corners are continuous with the roadway.

Photo Log

Photo 1: Typical laminar rust and section loss to the bottom 8" of the corrugated arch (east side shown). Typical laminar rust and section loss to the bottom 8" of the corrugated arch (east side shown).

Photo 3: Settled stones and voids in the top east end of the south embankment.

Photo 4: Longitudinal cracking to the eastbound lane of the wearing surface.

Photo 5: Damage to the top of the south railing near the east end.

Photo 6: Spall to the top of the west abutment / footing near the north end.

Photo 7: Debris pile near the north end of the culvert.

Photo 8: Aggradation at the southeast corner of the culvert.



Photo 1: Typical laminar rust and section loss to the bottom 8" of the corrugated arch (east side shown).



Photo 2: Typical laminar rust and section loss to the bottom 8" of the corrugated arch (east side shown).



Photo 3: Settled stones and voids in the top east end of the south embankment.



Photo 4: Longitudinal cracking to the eastbound lane of the wearing surface.



Photo 5: Damage to the top of the south railing near the east end.



Photo 6: Spall to the top of the west abutment / footing near the north end.



Photo 7: Debris pile near the north end of the culvert.



Photo 8: Aggradation at the southeast corner of the culvert.

Report Date: October 7, 2024 State Information		ClassificationC
	Agency Br.No.	(112) NBIS Bridge Length
Town= Manchester	L.O.	(104) Highway System
B.I.N= 8AL	AASHTO= 026.0	(26) Functional Class - Rural Local
RANK= 0 H.I.= 80.2 %	FHWA Select List= N (6/21/2017)	• •
Identification	,	(101) Parallel Structure
(8) Structure Number	M020218ALMUNBRI	(102) Direction of Traffic - 2-way traffic
(5) Inventory Route	150000000	•
(2) State Highway Department District	04	(103) Temporary Structure
(3) County Code 009 (4) Place code	37945	(105) Federal Lands Highways
(6) Features Intersected	WATER SAW MILL BROOK	, , ,
(7) Facility Carried	HWY ATWATER AVE	
(9) Location	0.25 MI E OF SCHOOL ST	(21) Maintain - Town Agency
(11) Kilometerpoint	0000.000	(22) Owner - Town Agency
(12) Base Highway Network	N	. ,
(13) LRS Inventory Route & Subroute	00000000000	ConditionCo
(16) Latitude	42 DEG 35 MIN 37.17 SEC	(58) Deck
(17) Longitude	70 DEG 45 MIN 42.10 SEC	(59) Superstructure
(98) Border Bridge State Code	Share %	(60) Substructure
(99) Border Bridge Structure No. #		(61) Channel & Channel Protection
Structure Type and Ma	aterial	(62) Culverts
(43) Structure Type Main: Steel	Code 319	Load Rating and PostingCo
· /	s bridge type: Not applicable	(31) Design Load - H 20=M 18
(44) Structure Type Appr:	0)1	(63) Operating Rating Method - Allowable Stress (AS)
Other	Code 000	(64) Operating Rating
		(65) Inventory Rating Method - Allowable Stress (AS)
(45) Number of spans in main unit	001	(66) Inventory Rating
(46) Number of approach spans	0000	(70) Bridge Posting
(107) Deck Structure Type - Not applicable	Code N	(41) Structure - Open AppraisalCo
(108) Wearing Surface / Protective System:		
A) Type of wearing surface - Bituminous	Code 6	(67) Structural Evaluation
B) Type of membrane - Not applicable	e=no deck Code N	(68) Deck Geometry (69) Underclearances, vert. and horiz.
C) Type of deck protection - Not applicable	e=no deck Code N	(71) Waterway adequacy
Age and Service		(72) Approach Roadway Alignment
(27) Year Built	2006	(36) Traffic Safety Features 0 N 0
(106) Year Reconstructed	0000	(113) Scour Critical Bridges
(42) Type of Service: On - Highway		Inspections
Under - Waterway	Code 15	(90) Inspection Date 10/01/24 (91) Frequency 24
(28) Lanes: On Structure 02	Under structure 00	(92) Critical Feature Inspection: (93) CFI D
(29) Average Daily Traffic	005800	(A) Fracture Critical Detail N 00 MO A) 00/0
(30) Year of ADT 2024 (109) Truck	ADT 07 %	(B) Underwater Inspection N 00 MO B) 00/0
(19) Bypass, detour length	199 KM	(C) Other Special Inspection N 00 MO C) 06/
Geometric Data		(*) Other Inspection () N 00 MO *) 00/
(48) Length of maximum span	0004.4 M	(*) Closed Bridge N 00 MO *) 00//
(49) Structure Length	00004.4 M	(*) UW Special Inspection N 00 MO *) 00//
(50) Curb or sidewalk: Left 00.	0 M Right 00.0 M	(*) Damage Inspection MO *) 00/
(51) Bridge Roadway Width Curb to Curb	000.0 M	Rating Loads
(52) Deck Width Out to Out	000.0 M	Report Date 00/00/00 H20 Type 3 Type 3S2 Type 8
(32) Approach Roadway Width (w/shoulders)	005.5 M	Operating 0.0 0.0 0.0 0.0
		Inventory 0.0 0.0 0.0 0.0
(33) Bridge Median - No median	Code 0	Field Posting
(34) Skew 10 DEG (35) Structur	**	Status Posting Date 00/00/00
(10) Inventory Route MIN Vert Clear	99.99 M	2 Axle 3 Axle 5 Axle Single
(47) Inventory Route Total Horiz Clear	08.4 M	Actual
(53) Min Vert Clear Over Bridge Rdwy	99.99 M	Recommended Mississ Circs
(54) Min Vert Underclear ref	00.00 M	Missing Signs N Misc.
(55) Min Lat Underclear RT ref	00.0 M	
(56) Min Lat Underclear LT	00.0 M	Bridge Name N Anti-missile fence N Acrow Panel N Jointless Bridge
Navigation Data		Freeze/Thaw N : Not Applicable
(38) Navigation Control - No navigation control of	on waterway Code 0	# Stairs On/Adjacent 0 Stair Owner(s)
(111) Pier Protection	Code	# Stairs On/Adjacent
(39) Navigation Vertical Clearance	000.0 M	
(116) Vert-lift Bridge Nav Min Vert Clear	M	N / N Liftbucket N / N Rigging N / N Other
(40) Navigation Horizontal Clearance	0000.0 M	N / N Ladder N / N Staging
		N / N Boat N / N Traffic Control Inspection
		Y/Y Wader N/N RR Flagperson Hours:
		N / N Inspector 50 N / N Police

TEST BORING LOG

GZA
GeoEnvironmental, Inc.
Engineers and Scientists

Cell Signaling Technology (CST)
Utility Bridge Crossing over Sawmill Brook
Atwater Ave, Manchester-by-the-Sea, MA

EXPLORATION NO.: B-1 SHEET: 1 of 2

PROJECT NO: 18.0175487.04 REVIEWED BY: MPS

Logged By: Sam Doyle **Drilling Co.:** New England Boring Contractors

Foreman: Mike Matarozzo

Type of Rig: Truck-Mounted Boring Location: See PlanSee Plan Rig Model: GtechDrill GT-8 Ground Surface Elev. (ft.): 48

Drilling Method: Final Boring Depth (ft.): 35

D&W Date Start - Finish: 9/3/2024 - 9/3/2024

V. Datum: NAVD88

H. Datum: NAD83

Hammer Type: Automatic Hammer Hammer Weight (lb.): 140

Hammer Fall (in.): 30

Auger or Casing O.D./I.D Dia (in.): 4.0/4.5

Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size: NX
 Groundwater Depth (ft.)

 Date
 Time
 Stab. Time
 Water
 Casing

 9/3/2024
 1210
 30 min
 10.3
 29

	Casing		Sa	mple	Э			0 1 5	ž	DID	- Stratum -
Depth (ft)	Blows/ (Core Rate)	No.	Depth (ft.)		Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	PID (PPM)	
-		S-1	0.5-2.5	24	14	19 50 39 18	89	S-1: Very dense, brown to tan, fine to coarse SAND, little Gravel, trace Silt.	1 2 3		_{-0.44} ASPHALT _{47.6}
5_		S-2	4.0-6.0	24	5	13 36 55 20	91	S-2: Very dense, brown, fine to coarse SAND, some Gravel, trace Silt, wet.	4		FILL
10 _		S-3	9.0-11.0	24	0	5 3 3 5	6	S-3: No Recovery	5		9 39.0 ORGANIC SILT AND SAND
- - -		S-4	11.0-13.0	24	12	6 8 6 9	14	S-4: (Top 6") Black to brown, ORGANIC SILT, some fine to medium Sand, trace Gravel, wet. (Bottom 9") Gray, fine to medium SAND, trace Silt, wet.			<u> 11.5 36.5</u>
15 _		S-5	14.0-16.0	24	7	4 6 8 6	14	S-5: Medium dense, gray, fine to medium SAND, trace Silt, wet.			
20 _		S-6	19.0-21.0	24	10	4 7 7 6	14	S-6: Medium dense, gray, fine SAND, some Silt, wet.			SAND
25 _		S-7	24.0-26.0	24	19	5 4 1 1	5	S-7: (Top 9") Gray, fine SAND, some Silt, wet. (Bottom 10") Gray, CLAYEY SILT, little fine Sand, wet.			25 23.0
-		S-8	26.0-28.0	24	24	WOH 1 WOH/12"	1	S-8: Very Soft, gray, CLAY & SILT, wet.			CLAY & SILT
-									6		28.519.5

- 1 Ground elevation estimated from Autocad file titled "25770ec.dwg" provided by Hancock Associates on 10/17/2022.
- 2 Rollerbit through existing pavement to approximately 6 inches below ground surface (bgs).
- 3 Drove 4-inch casing from 0 to 29 feet bgs.
- 4 Slow rollerbit advancement from 2.5 to 4 feet bgs.
- 5 No recovery in 2-inch or 3-inch split spoon samples from 9 to 11 feet bgs. Blow counts represent 2-inch split spoon drive.
- 6 Hit an obstruction at approximately 28.5 feet bgs. Rollerbit through obstruction to refual at 30 feet bgs. Possible bedrock indicated in wash water from approximately 29.5 to 30 feet bgs.

See Log Key for exploration of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-1

32A TEMPLATE TEST BORING - GZA GLX PLOG 2016_

TEST BORING LOG Cell Signaling Technology (CST) **EXPLORATION NO.: GZA** Utility Bridge Crossing over Sawmill Brook SHEET: 2 of 2 GeoEnvironmental, Inc. Atwater Ave, Manchester-by-the-Sea, MA PROJECT NO: 18.0175487.04 Engineers and Scientists **REVIEWED BY: MPS** Type of Rig: Truck-Mounted Boring Location: See PlanSee Plan H. Datum: NAD83 Logged By: Sam Doyle **Drilling Co.:** New England Boring Contractors Rig Model: GtechDrill GT-8 Ground Surface Elev. (ft.): 48 Mike Matarozzo **Drilling Method:** Final Boring Depth (ft.): V. Datum: NAVD88 Foreman: Date Start - Finish: 9/3/2024 - 9/3/2024 D&W Groundwater Depth (ft.) Sampler Type: SS Hammer Type: Automatic Hammer Date Time Stab. Time Water Casing Hammer Weight (lb.): 140 Sampler O.D. (in.): 2.0 9/3/2024 1210 10.3 Hammer Fall (in.): 30 Sampler Length (in.): 24 Auger or Casing O.D./I.D Dia (in.): 4.0/4.5 Rock Core Size: NX Casino Sample Stratum Depth Blows Sample Description and Identification PID Description $\stackrel{?}{=}$ Depth Pen Rec. Blows SPT (Core (Modified Burmister Procedure) (PPM) (ft) No. (RQD) (ft.) (in) (in) Value Rate) 30.0-35.0 60 59 RQD= C-1: Hard, fresh, medium grained, gray to pink, (5:29) 81% GRANITE. Slightly to moderately fractured, very close to (4:46)moderately close, vertical to sub-horizontal **BEDROCK** (4:53) fractures/joints. (4:26)7 (4:41)35 13.0 35 End of exploration at 35 feet 40 50

7 - Upon completion, borehole was backfilled with soil cuttings to approximately 6-inch bgs. Repaired ground surface with aspahlt cold patch.

See Log Key for exploration of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-1

32A TEMPLATE TEST BORING - GZA GLX PLOG 2016 09 22.GDT - 10/9/24 12:08 - Z\GINT PROJECT DATABASE\18\18.0175487.04 - ATWATER AVE.GR\

REMARKS

55

TEST BORING LOG

GZA GeoEnvironmental, Inc. Engineers and Scientists

Cell Signaling Technology (CST) Utility Bridge Crossing over Sawmill Brook Atwater Ave, Manchester-by-the-Sea, MA EXPLORATION NO.: SHEET: 1 of 2

PROJECT NO: 18.0175487.04

REVIEWED BY: MPS

Logged By: Sam Doyle **Drilling Co.:** New England Boring Contractors

Foreman: Mike Matarozzo

Type of Rig: Truck-Mounted Boring Location: See PlanSee Plan Rig Model: GtechDrill GT-8 Ground Surface Elev. (ft.): 48

Drilling Method: Final Boring Depth (ft.): 45 D&W Date Start - Finish: 9/4/2024 - 9/4/2024

V. Datum: NAVD88

H. Datum: NAD83

Hammer Type: Automatic Hammer Hammer Weight (lb.): 140

Hammer Fall (in.): 30

Auger or Casing O.D./I.D Dia (in.): 4.0/4.5

Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size:

Groundwater Depth (ft.) Date Time Stab. Time Water Casing 9/4/2024 1224 30 min 43

	Casing		Sa	mple	е			0 1 5 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	ırk	D.D.	€ Stratum >	П
Depti (ft)	Blows/ (Core Rate)	No.	Depth (ft.)		Rec. (in)	Blows (RQD)	SPT Value	Sample Description and Identification (Modified Burmister Procedure)	Remark	PID (PPM)	Description = #	\\
		S-1	0.5-2.5	24	12	32 30 25 23	55	S-1: Very dense, dark brown to tan, fine to coarse SAND, little Silt, trace Gravel.	1 2 3		0.625 ASPHALT 47	4
5_	- - - -	S-2	4.0-6.0	24	7	10 13 10 8	23	S-2: Medium dense, brown, fine to coarse SAND, some Gravel, trace Silt.	4		FILL	
- 10/9/24 12:08 - 2:10in Project DATABASE(18(18:01/948/.04 - ATWATER AVE.GPJ 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	- - - - -	S-3	9.0-11.0	24	5	3 1 1 6	2	S-3: (Top 2") Brown, fine to coarse SAND, some Gravel, trace Silt, wet. (Middle 1") Black, ORGANIC SILT, some fine to medium Sand, trace Root. (Bottom 2") Gray, fine to medium SAND, little Silt, wet.	5		11.5 12 ORGANIC SIL 136	<u>5</u> 0
15 _		S-4	14.0-16.0	24	0	5 7 7 9	14	S-4: No Recovery	6			
L DA LABASI		S-5	16.0-18.0	24	16	10 10 12 11	22	S-5: Medium dense, gray, fine to medium SAND, little Silt, wet.				
20 - 2:/elln PROJECT		S-6	19.0-21.0	24	7	5 6 8 8	14	S-6: Medium dense, gray, fine SAND, little Silt, wet.			SAND	
25 - 30 - 30 - 30 - 30 - 30 - 30 - 30 - 3	- - - -	S-7	24.0-26.0	24	6	6 6 8 8	14	S-7: Medium dense, gray, fine SAND, little Silt, wet.				
30	-	S-8	29.0-31.0	24	10	4 10	31	S-8: (Top 4") Gray, fine to medium SAND, little Silt, wet.			30 18	0

- 1 Ground elevation estimated from Autocad file titled "25770ec.dwg" provided by Hancock Associates on 10/17/2022.
- 2 Rollerbit from ground surface to approximately 6-inches below ground surface (bgs).
- 3 Drove 4-inch casing from 0 to 45 feet bgs.
- 4 Rollerbit chatter from approximately 3.5 to 4 feet bgs.
- 5 Rollerbit chatter and complete loss of water from approximately 8.5 to 9 feet bgs.
- 6 No recovery in 2-inch split spoon sample S-4 from 14 to 16 feet bgs. 3-inch split spoon would not advanced through boring to collect a sample from 14 to 16 feet bgs. Drilling indicated it was due to a possible shift in the casing. Samped 16 to 18 feet with 2 inch split spoon.

See Log Key for exploration of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-2

324 TEMPLATE TEST BORING - GZA GLX PLOG 2016 09 22:GDT - 10/9/24 12:08 - Z\GINT PROJECT DATABASE\18\18,0175487.04 - ATWATER AVE.GPJ

TEST BORING LOG

GZA GeoEnvironmental, Inc. Engineers and Scientists

Cell Signaling Technology (CST) Utility Bridge Crossing over Sawmill Brook Atwater Ave, Manchester-by-the-Sea, MA

EXPLORATION NO.: SHEET: 2 of 2

PROJECT NO: 18.0175487.04

REVIEWED BY: MPS

Logged By: Sam Doyle

Drilling Co.: New England Boring Contractors

Foreman: Mike Matarozzo

Type of Rig: Truck-Mounted Boring Location: See PlanSee Plan Rig Model: GtechDrill GT-8 Ground Surface Elev. (ft.): 48 **Drilling Method:**

Final Boring Depth (ft.): 45 Date Start - Finish: 9/4/2024 - 9/4/2024 H. Datum: NAD83 V. Datum: NAVD88

Hammer Type: Automatic Hammer Hammer Weight (lb.): 140

Hammer Fall (in.): 30

Auger or Casing O.D./I.D Dia (in.): 4.0/4.5

Sampler Type: SS Sampler O.D. (in.): 2.0 Sampler Length (in.): 24 Rock Core Size:

D&W

Groundwater Depth (ft.) Date Time Stab. Time Water

Casing 9/4/2024 1224 30 min 43

Casing		Sa	mpl	е.				논		C Stratum .
Blows/ (Core	No.	Depth (ft.)	Pen	Rec.	Blows (RQD)	SPT Value	(Modified Burmister Procedure)	Rema	PID (PPM)	Stratum . (±) Description (±) (±)
					21 14		(Bottom 6") Olive-gray, fine to coarse SAND, some Gravel, little Silt, wet.			
	S-9	34.0-36.0	24	8	15 11 9 9	20	S-9: Medium dense, olive-gray, fine to coarse SAND, some Gravel, little Silt, wet.	7		
	S-10	39.0-41.0	24	12	19 29 48 47	77	S-10: Very dense, olive-gray, fine to coarse SAND and GRAVEL little Silt wet			GLACIAL DEPOSIT
	S-11	43.0-45.0	24	8	55 35	66	S-11: Very dense, olive-gray, fine to coarse SAND and	8		
					31 31		GRAVEL, little Silt, wet. End of exploration at 45 feet	9		45 3.0
							·			
	Blows/	Blows/ (Core Rate) No.	S-9 34.0-36.0 S-10 39.0-41.0	S-9 34.0-36.0 24 S-10 39.0-41.0 24	No. Depth (ft.) Pen Rec. (in) (in)	S-10 39.0-41.0 24 8 55 35 S-11 43.0-45.0 24 8 55 35	S-10 39.0-41.0 24 8 55 35 66 SPT Value S-11 43.0-45.0 24 8 55 35 66 SPT Value Value	S-9 34.0-36.0 24 8 15 11 9 9 9 9 9 9 9 9	S-10 39.0-41.0 24 8 55 35 31 31 Sample Description and Identification (Modified Burmister Procedure) Sample Description and Identification (Modified Burmister Procedure) \$\frac{1}{2}\$\frac	S-9 34.0-36.0 24 8 15 11 9 9 20 S-9: Medium dense, olive-gray, fine to coarse SAND, some Gravel, little Silt, wet. S-10 39.0-41.0 24 12 19 29 48 47 77 S-10: Very dense, olive-gray, fine to coarse SAND and GRAVEL, little Silt, wet. S-11 43.0-45.0 24 8 55 35 31 31 66 S-11: Very dense, olive-gray, fine to coarse SAND and GRAVEL, little Silt, wet.

7 - Rollerbit chatter from 33.5 to 34 feet bgs.

8 - Casing and rollerbit refusal at 45 feet bgs on probable bedrock.
9 - Upon completion, borehole was backfilled with soil cuttings to approximately 6-inch bgs. Repaired ground surface with aspahlt cold patch.

See Log Key for exploration of sample description and identification procedures. Stratification lines represent approximate boundaries between soil and bedrock types. Actual transitions may be gradual. Water level readings have been made at the times and under the conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the times the measurements were made.

Exploration No.: B-2

32A TEMPLATE TEST BORING - GZA GLX PLOG 2016

REMARKS

Article 3.12.6, differential movements between and within substructure units shall be considered when determining the most critical combinations of force effects.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS$$

(3.4.1-2)

Table 3.4.1-1—Load Combinations and Load Factors

1	DC	3			- 55	26			38	U	se One	of These	e at a Tir	ne
Load Combination Limit State	DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	ws	WL	FR	TU	TG	SE	EO	BL	IC	CT	CV
Strength I (unless noted)	γ_P	1.75	1.00	S-00	3 - Z	1.00	0.50/1.20	γ_{TG}	γsε			10	30=30	200
Strength II	γ_P	1.35	1.00		_	1.00	0.50/1.20	YTG	YSE	_	7—	92_9	3-3	
Strength III	γ_P	_	1.00	1.00	_	1.00	0.50/1.20	77G	YSE	_	_	1-0	-	-
Strength IV	Yp	-	1.00		Ī	1.00	0.50/1.20			l	-	7-7		
Strength V	70	1.35	1.00	1.00	1.00	1.00	0.50/1.20	770	YSE		-	_		
Extreme Event I	1.00	γεQ	1.00	100		1.00	· ·	:::		1.00		12-10 °	- S	
Extreme Event II	1.00	0.50	1.00		_	1.00	·		_		1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	YTG	YSE	_	-	-	-	-
Service II	1.00	1.30	1.00			1.00	1.00/1.20	-		-		2-1	27-22	
Service III	1.00	YLL	1.00	(1.00	1.00/1.20	γ_{TG}	ΥSE	%—)		-3	25-20%	
Service IV	1.00	-	1.00	1.00	-	1.00	1.00/1.20	-	1.00	- T	-			-
Fatigue I— LL, IM & CE only		1.75			_	12		_						9200
Fatigue II— LL, IM & CE only		0.80			-		85-8	s ' − s					2 5 - 1 20	

Note: For Service I, the load factor for EV equals 1.2 for Stiffness Method Soil Failure as shown in Table 3.4.1-2.

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

	Type of Load, Foundation Type, and	Load l	Factor
	Method Used to Calculate Downdrag	Maximum	Minimum
DC: Component	and Attachments	1.25	0.90
DC: Strength IV	only	1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Sur	faces and Utilities	1.50	0.65
EH: Horizontal E	arth Pressure		10.0005.171.00
 Active 		1.50	0.90
 At-Rest 		1.35	0.90
• AEP for anch	nored walls	1.35	N/A
EL: Locked-in Co	onstruction Stresses	1.00	1.00
EV: Vertical Eart	h Pressure		
 Overall and 0 	Compound Stability	1.00	N/A
 Retainin 	g Walls and Abutments	1.35	1.00
 MSE wa 	ll internal stability soil reinforcement loads		
 Stiffnes 	s Method		
 Re 	inforcement and connection rupture	1.35	N/A
	l failure – geosynthetics (Service I)	1.20	N/A
 Cohere 	nt Gravity Method	1.35	N/A
 Rigid Buried 	Structure	1.30	0.90
 Rigid Frames 	3	1.35	0.90
 Flexible Buri 	ed Structures	107.07.00	304,545,345,3
 Metal I 	Box Culverts, Structural Plate Culverts with Deep Corrugations, and		
	ass Culverts	1.50	0.90
 Thermo 	plastic Culverts	1.30	0.90
 All other 		1.95	0.90
• Internal and	Compound Stability for Soil Failure in Soil Nail Walls	1.00	N/A
ES: Earth Surchar	·ge	1.50	0.75

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	PS	CR, SH
Superstructures—Segmental	1.0	See γ_P for <i>DC</i> , Table 3.4.1-2
Concrete Substructures supporting Segmental		
Superstructures (see 3.12.4, 3.12.5)		
Concrete Superstructures—nonsegmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
 using I_g 	0.5	0.5
using I _{effective}	1.0	1.0
Steel Substructures	1.0	1.0

Table 3.4.1-4—Load Factors for Live Load for Service III Load Combination, 7LL

Component	γ_{LL}
Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in <u>Article 5.9.5.4</u> in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

indicate that the ratio of maximum stress range to effective stress range is increased as compared to standard bridge girders. This is due to a number of factors such as occasional heavy wheels and reduced local load distribution that occurs in deck elements. These Specifications produce a ratio that is consistent with the original findings of NCHRP Report 299 (Moses et al., 1987).

Earlier editions of these specifications used an additional factor of 1.5 that was applied to the then-current 1.5 load factor for Fatigue I resulting in an effective load factor of 2.25. The current additional factor of 1.3 results in essentially the same combined load factor when applied to the current load factor of 1.75 for Fatigue I.

3.5—PERMANENT LOADS

3.5.1-Dead Loads: DC, DW, and EV

Dead loads shall include the weight of all components of the structure, appurtenances and utilities attached thereto, earth cover, wearing surface, future overlays, and planned widenings.

In the absence of more precise information, the unit weights, specified in Table 3.5.1-1, may be used for dead loads.

C3.5.1

Table 3.5.1-1 provides traditional unit weights. The unit weight of granular materials depends upon the degree of compaction and water content. The unit weight of concrete is primarily affected by the unit weight of the aggregate, which varies by geographical location and increases with concrete compressive strength. The unit weight of reinforced concrete is generally taken as 0.005 kcf greater than the unit weight of plain concrete. The values provided for wood include the weight of mandatory preservatives. The weight of transit rails, etc., is to be used only for preliminary design.

Table 3.5.1-1—Unit Weights

		Unit Weight				
	Material	(kcf)				
Aluminum Al	loys	0.175				
Bituminous W	/earing Surfaces	0.140				
Cast Iron		0.450				
Cinder Filling		0.060				
Compacted Sa	and, Silt, or Clay	0.120				
Concrete	Lightweight	0.110 to 0.135				
	Normal Weight with $f'_c \le 5.0$ ksi	0.145				
	Normal Weight with $5.0 < f'_c \le 15.0 \text{ ksi}$	$0.140 + 0.001 f'_c$				
Loose Sand, S		0.100				
Soft Clay		0.100				
Rolled Gravel	, Macadam, or Ballast	0.140				
Steel		0.490				
Stone Masonr	у	0.170				
Wood	Hard	0.060				
	Soft	0.050				
Water	Fresh	0.0624				
	Salt	0.0640				
Item		Weight per Unit Length (klf)				
Transit Rails,	Ties, and Fastening per Track	0.200				

3.5.2-Earth Loads: EH, ES, and DD

Earth pressure, earth surcharge, and downdrag loads shall be as specified in Article 3.11.

3.6-LIVE LOADS

3.6.1—Gravity Loads: LL and PL

3.6.1.1—Vehicular Live Load

3.6.1.1.1—Number of Design Lanes

Unless specified otherwise, the width of the design lanes should be taken as 12.0 ft. The number of design lanes should be determined by taking the integer part of the ratio w/12.0, where w is the clear roadway width in feet between curbs, barriers, or both. Possible future changes in the physical or functional clear roadway width of the bridge should be considered.

In cases where the traffic lanes are less than 12.0 ft wide, the number of design lanes shall be equal to the number of traffic lanes, and the width of the design lane shall be taken as the width of the traffic lane.

Roadway widths from 20.0 to 24.0 ft shall have two design lanes, each equal to one-half the roadway width.

3.6.1.1.2-Multiple Presence of Live Load

The provisions of this Article shall not be applied to the fatigue limit state for which one design truck is used, regardless of the number of design lanes. Where the single-lane approximate distribution factors in Articles 4.6.2.2 and 4.6.2.3 are used, other than the lever rule and statical method, the force effects shall be divided by 1.20.

Unless specified otherwise herein, the extreme live load force effect shall be determined by considering each possible combination of number of loaded lanes multiplied by a corresponding multiple presence factor to account for the probability of simultaneous lane occupation by the full HL93 design live load. In lieu of site-specific data, the values in Table 3.6.1.1.2-1:

- shall be used when investigating the effect of one lane loaded, and
- may be used when investigating the effect of three or more lanes loaded.

For the purpose of determining the number of lanes when the loading condition includes the pedestrian loads specified in Article 3.6.1.6 combined with one or more lanes of the vehicular live load, the pedestrian loads may be taken to be one loaded lane.

C3.6.1.1.1

It is not the intention of this Article to promote bridges with narrow traffic lanes. Wherever possible, bridges should be built to accommodate the standard design lane and appropriate shoulders.

C3.6.1.1.2

The multiple presence factors have been included in the approximate equations for distribution factors in Articles 4.6.2.2 and 4.6.2.3, both for single and multiple lanes loaded. The equations are based on evaluation of several combinations of loaded lanes with their appropriate multiple presence factors and are intended to account for the worst-case scenario. Where use of the lever rule is specified in Article 4.6.2.2 and 4.6.2.3, the Engineer must determine the number and location of vehicles and lanes, and, therefore, must include the multiple presence. Stated another way, if a sketch is required to determine load distribution, the Engineer is responsible for including multiple presence factors and selecting the worst design case. The factor 1.20 from Table 3.6.1.1.2-1 has already been included in the approximate equations and should be removed for the purpose of fatigue investigations.

The entry greater than 1.0 in Table 3.6.1.1.2-1 results from statistical calibration of these Specifications on the basis of pairs of vehicles instead of a single vehicle. Therefore, when a single vehicle is on the bridge, it can be heavier than each one of a pair of vehicles and still have the same probability of occurrence.

The consideration of pedestrian loads counting as a "loaded lane" for the purpose of determining a multiple presence factor (m) is based on the assumption that

The factors specified in Table 3.6.1.1.2-1 shall not be applied in conjunction with approximate load distribution factors specified in Articles 4.6.2.2 and 4.6.2.3, except where the lever rule is used or where special requirements for exterior beams in beam-slab bridges, specified in Article 4.6.2.2.2d, are used.

Table 3.6.1.1.2-1 — Multiple Presence Factors, m

Number of Loaded Lanes	Multiple Presence Factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

3.6.1.2—Design Vehicular Live Load

3.6.1.2.1—General

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

simultaneous occupancy by a dense loading of people combined with a 75-year design live load is remote. For the purpose of this provision, it has been assumed that if a bridge is used as a viewing stand for eight hours each year for a total time of about one month, the appropriate live load to combine with it would have a one-month recurrence interval. This is reasonably approximated by use of the multiple presence factors, even though they are originally developed for vehicular live load.

Thus, if a component supported a sidewalk and one lane, it would be investigated for the vehicular live load alone with m = 1.20, and for the pedestrian loads combined with the vehicular live load with m = 1.0. If a component supported a sidewalk and two lanes of vehicular live load, it would be investigated for:

- one lane of vehicular live load, m = 1.20;
- the greater of the more significant lanes of vehicular live load and the pedestrian loads or two lanes of vehicular live load, m = 1.0, applied to the governing case; and
- two lanes of vehicular live load and the pedestrian loads, m = 0.85.

The multiple presence factor of 1.20 for a single lane does not apply to the pedestrian loads. Therefore, the case of the pedestrian loads without the vehicular live load is a subset of the second bulleted item.

The multiple presence factors in Table 3.6.1.1.2-1 were developed on the basis of an ADTT of 5,000 trucks in one direction. The force effect resulting from the appropriate number of lanes may be reduced for sites with lower ADTT as follows:

- If 100 ≤ ADTT ≤ 1,000, 95 percent of the specified force effect may be used; and
- If ADTT < 100, 90 percent of the specified force effect may be used.

This adjustment is based on the reduced probability of attaining the design event during a 75-year design life with reduced truck volume.

C3.6.1.2.1

Consideration should be given to site-specific modifications to the design truck, design tandem, and/or the design lane load under the following conditions:

- The legal load of a given jurisdiction is significantly greater than typical;
- The roadway is expected to carry unusually high percentages of truck traffic;
- Flow control, such as a stop sign, traffic signal, or toll booth, causes trucks to collect on certain areas of a bridge or to not be interrupted by light traffic; or

Except as modified in Article 3.6.1.3.1, each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load,

where applicable. The loads shall be assumed to occupy

10.0 ft transversely within a design lane.

3.6.1.2.2—Design Truck

The weights and spacings of axles and wheels for the design truck shall be as specified in Figure 3.6.1.2.2-1. A dynamic load allowance shall be considered as specified in Article 3.6.2.

Except as specified in Articles 3.6.1.3.1 and 3.6.1.4.1, the spacing between the two 32.0-kip axles shall be varied between 14.0 ft and 30.0 ft to produce extreme force effects.

 Special industrial loads are common due to the location of the bridge.

See also discussion in Article C3.6.1.3.1.

The live load model, consisting of either a truck or tandem coincident with a uniformly distributed load, was developed as a notional representation of shear and moment produced by a group of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The vehicles considered to be representative of these exclusions were based on a study conducted by the Transportation Research Board (Cohen, 1990). The load model is called "notional" because it is not intended to represent any particular truck.

In the initial development of the notional live load model, no attempt was made to relate to escorted permit loads, illegal overloads, or short duration special permits. The moment and shear effects were subsequently compared to the results of truck weight studies (Csagoly and Knobel, 1981; Nowak, 1992; Kulicki, 2006), selected Weigh-in-Motion (WIM) data, and the 1991 Ontario Highway Bridge Design Code (OHBDC) live load model. These subsequent comparisons showed that the notional load could be scaled by appropriate load factors to be representative of these other load spectra.

Earlier editions of the commentary included information about the background of the HL-93. This information can be found in Kulicki (2006).

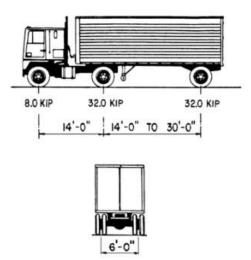


Figure 3.6.1.2.2-1—Characteristics of the Design Truck

3.6.1.2.3—Design Tandem

The design tandem shall consist of a pair of 25.0-kip axles spaced 4.0 ft apart. The transverse spacing of wheels shall be taken as 6.0 ft. A dynamic load allowance shall be considered as specified in Article 3.6.2.

3.6.1.2.4—Design Lane Load

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 10.0-ft width. The force effects from the design lane load shall not be subject to a dynamic load allowance.

3.6.1.2.5—Tire Contact Area

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 20.0 in. and whose length is 10.0 in.

The tire pressure shall be assumed to be uniformly distributed over the contact area. The tire pressure shall be assumed to be distributed as follows:

- On continuous surfaces, uniformly over the specified contact area, and
- On interrupted surfaces, uniformly over the actual contact area within the footprint with the pressure increased in the ratio of the specified to actual contact areas.

For the design of orthotropic decks and wearing surfaces on orthotropic decks, the front wheels shall be assumed to be a single rectangle whose width and length are both 10.0 in. as specified in Article 3.6.1.4.1.

C3.6.1.2.5

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the Engineer.

As a guideline for other truck loads, the tire area in in.² may be calculated from the following dimensions:

Tire width = P/0.8

Tire length = $6.4\gamma(1 + IM/100)$

where:

γ = load factor

IM = dynamic load allowance percent

P = design wheel load (kip)

Table 3.6.1.2.6a-1—Live Load Distribution Factor (LLDF) for Buried Structures

Structure Type	LLDF Transverse or Parallel to Span
Concrete Pipe with fill depth 2.0 ft or greater	1.15 for diameter 2.0 ft or less 1.75 for diameters 8.0 ft or greater Linearly interpolate for LLDF between these limits
All other culverts and buried structures	1.15

The rectangular area, A_{LL} , shall be determined as:

$$A_{LL} = l_w w_w ag{3.6.1.2.6a-1}$$

The term l_w and w_w shall be determined as specified in Articles 3.6.1.2.6b and 3.6.1.2.6c.

C3.6.1.2.6b

The case where traffic is parallel to the culvert span

applies to the vast majority of highway culverts.

For live load distribution transverse to culvert spans, the wheel/axle load interaction depth H_{int-t} shall be determined as:

determined as:

$$H_{int} = \frac{s_w - \frac{w_t}{12} - \frac{0.06D_j}{12}}{LLDF}$$
 (3.6.1.2.6b-1)

in which:

where H < H_{int-t}:

$$w_{w} = \frac{w_{t}}{12} + LLDF(H) + 0.06 \frac{D_{t}}{12}$$
 (3.6.1.2.6b-2)

• where $H \ge H_{int-t}$:

$$W_{w} = \frac{W_{t}}{12} + S_{w} + LLDF(H) + 0.06 \frac{D_{t}}{12}$$
 (3.6.1.2.6b-3)

For live load distribution parallel to culvert span, the wheel/axle load interaction depth H_{int-p} shall be determined as:

$$H_{int-p} = \frac{s_a - \frac{l_t}{12}}{LLDF}$$
 (3.6.1.2.6b-4)

in which:

where H < H_{int-p}:

$$l_{w} = \frac{l_{t}}{12} + LLDF(H)$$
 (3.6.1.2.6b-5)

where H ≥ H_{int}:

$$l_{w} = \frac{l_{i}}{12} + s_{a} + LLDF(H)$$
 (3.6.1.2.6b-6)

where:

 A_{LL} = rectangular area at depth H (ft²) l_w = live load patch length at depth H (ft) w_w = live load patch width at depth H (ft)

 H_{int-t} = wheel interaction depth transverse to culvert

span (ft)

 s_w = wheel spacing, 6.0 ft w_t = tire patch width, 20 (in.)

 D_i = inside diameter or clear span of the culvert

(in.)

LLDF = live load distribution factor as specified in Table 3.6.1.2.6a-1

H = depth of fill over culvert (ft)

 H_{int-p} = axle interaction depth parallel to culvert span

(ft)

 s_a = axle spacing (ft) l_t = tire patch length, 10 (in.)

The live load vertical crown pressure shall be determined as:

$$P_{L} = \frac{P\left(1 + \frac{IM}{100}\right)(m)}{A_{LL}}$$
(3.6.1.2.6b-7)

where:

 P_L = live load vertical crown pressure (ksf)

P = live load applied at surface on all interacting wheels (kip)

IM = dynamic load allowance as specified in Article 3.6.2.2

m = multiple presence factor specified in Article
3.6.1.1.2

 A_{LL} = rectangular area at depth H (ft²)

3.6.1.2.6c—Traffic Perpendicular to the Culvert Span

The provisions of Article 3.6.1.2.6b shall apply with the terms w_t and s_w in Eqs. 3.6.1.2.6b-1 through 3.6.1.2.6b-3 replaced by l_t and s_a respectively, and the terms l_t and s_a in Eqs. 3.6.1.2.6b-4 through 3.6.1.2.6b-6 replaced by w_t and s_w respectively.

3.6.1.3—Application of Design Vehicular Live Loads

3.6.1.3.1-General

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle spacing specified in Article 3.6.1.2.2, combined with the effect of the design lane load, and
- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

Axles that do not contribute to the extreme force effect under consideration shall be neglected.

Both the design lanes and the 10.0-ft loaded width in each lane shall be positioned to produce extreme force effects. The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

- For the design of the deck overhang—1.0 ft from the face of the curb or railing, and
- For the design of all other components—2.0 ft from the edge of the design lane.

Unless otherwise specified, the lengths of design lanes, or parts thereof, that contribute to the extreme force effect under consideration shall be loaded with the design lane load.

3.6.1.3.2—Loading for Optional Live Load Deflection Evaluation

If the Owner invokes the optional live load deflection criteria specified in Article 2.5.2.6.2, the deflection should be taken as the larger of:

C3.6.1.3.1

The effects of an axle sequence and the lane load are superposed in order to obtain extreme values. This is a deviation from the traditional AASHTO approach, in which either the truck or the lane load, with an additional concentrated load, provided for extreme effects.

The lane load is not interrupted to provide space for the axle sequences of the design tandem or the design truck; interruption is needed only for patch loading patterns to produce extreme force effects.

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft to 40.0 ft apart, combined with the design lane load specified in Article 3.6.1.2.4. The design tandems should be placed in adjacent spans to produce maximum force effect. One hundred percent of the combined effect of the design tandems and the design lane load should be used. This is consistent with Article 3.6.1.2.1 and should not be considered a replacement for the Strength II Load Combination.

Only those areas or parts of areas that contribute to the same extreme being sought should be loaded. The loaded length should be determined by the points where the influence surface meets the centerline of the design lane.

The HL-93 live load model was found to be appropriate for global analysis of long-span bridges (Nowak, 2010). In general, the design lane load portion of the HL-93 design load, which is the major contributor to live load force effects for long loaded lengths, is conservative. The conservatism is generally acceptable since members with long loaded lengths typically have much larger dead load than the live load. The conservatism could be somewhat less where the dead load has been mitigated, such as with cambered stiffening trusses on suspension bridges.

Where a sidewalk is not separated from the roadway by a crashworthy traffic barrier, consideration should be given to the possibility that vehicles can mount the sidewalk.

C3.6.1.3.2

As indicated in C2.5.2.6.1, live load deflection is a service issue, not a strength issue. Experience with bridges designed under previous editions of the AASHTO Standard Specifications for Highway Bridges indicated no adverse effects of live load deflection per se. Therefore, there appears to be little reason to require that the past criteria be

Table 3.6.2.1-1—Dynamic Load Allowance, IM

Component	IM
Deck Joints—All Limit States	75%
All Other Components:	
Fatigue and Fracture Limit State	15%
All Other Limit States	33%

The application of dynamic load allowance for buried components, covered in Section 12, shall be as specified in Article 3.6.2.2.

Dynamic load allowance need not be applied to:

- retaining walls not subject to vertical reactions from the superstructure, and
- foundation components that are entirely below ground level.

The dynamic load allowance may be reduced for components, other than joints, if justified by sufficient evidence, in accordance with the provisions of Article 4.7.2.1.

- such as deck joints, cracks, potholes, and delaminations,
- dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 3.6.2.1-1 is the product of 4/3 and the basic 25 percent.

Generally speaking, the dynamic amplification of trucks follows the following general trends:

- As the weight of the vehicle goes up, the apparent amplification goes down.
- Multiple vehicles produce a lower dynamic amplification than a single vehicle.
- More axles result in a lower dynamic amplification.

For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted. A study of dynamic effects presented in a report by the Calibration Task Group (Nowak 1992) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

This Article recognizes the damping effect of soil when in contact with some buried structural components, such as footings. To qualify for relief from impact, the entire component must be buried. For the purpose of this Article, a retaining type component is considered to be buried to the top of the fill.

3.6.2.2—Buried Components

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

$$IM = 33(1.0 - 0.125D_E) \ge 0\%$$
 (3.6.2.2-1)

where:

 D_E = the minimum depth of earth cover above the structure (ft)

Table 12.5.5-1—Resistance Factors for Buried Structures

Structure Type	Resistance Factor
Metal Pipe, Arch, and Pipe Arch Structures	60 P
Helical pipe with lock seam or fully welded seam:	
Minimum wall area and buckling	1.00
Annular pipe with spot-welded, riveted, or bolted seam:	
Minimum wall area and buckling	1.00
Minimum longitudinal seam strength	0.67
Bearing resistance to pipe arch foundations	Refer to Section 10
Structural plate pipe:	99-90
Minimum wall area and buckling	1.00
Minimum longitudinal seam strength	0.67
Bearing resistance to pipe arch foundations	Refer to Section 10
Long-Span Structural Plate and Tunnel Liner Plate Structures	
Minimum wall area	0.67
Minimum seam strength	0.67
 Bearing resistance of pipe arch foundations 	Refer to Section 10
Structural Plate Box Structures	
Plastic moment strength	1.00
Bearing resistance of pipe arch foundations	Refer to Section 10
Reinforced Concrete Pipe	
Direct design method:	
Type 1 installation:	0.90
Flexure	0.82
• Shear	0.82
Radial tension	525 V 52.5
Other type installations:	
Flexure	1.00 0.90
Shear	0.90
Radial tension Reinforced Concrete Cast-in-Place Box Structures	0.50
Flexure	0.90
• Shear	0.85
Reinforced Concrete Precast Box Structures	- A 12 C - C
Flexure	1.00
• Shear	0.90
Reinforced Concrete Precast Three-Sided Structures	
• Flexure	0.95
• Shear	0.90
Thermoplastic Pipe	
PE and PVC pipe:	
• Thrust, ϕ_T	1.00
 Soil stiffness, φ_s 	0.90
 Global buckling, φ_{bck} 	0.70
• Flexure, φ _f	1.00
Fiberglass Pipe	0.9
• Flexure, dy	0.63
Global Buckling, φ _{bck} Proc. Compared al Structural Plate Structures	
Deep Corrugated Structural Plate Structures	
 Minimum wall area and general buckling, φ_b 	0.70
 Plastic hinge, φh Soil, φs 	0.90 0.90
• Soil, φ _s Steel-Reinforced Thermoplastic Culverts	0.90
0.0000	
Minimum wall area and buckling	1.00

- · wall area of pipe,
- · buckling strength, and
- seam resistance for structures with longitudinal seams.

12.7.2.1—Section Properties

Dimensions and properties of pipe cross-sections; minimum seam strength; mechanical and chemical requirements for aluminum corrugated and steel corrugated pipe and pipe-arch sections; and aluminum and steel corrugated structural plate pipe, pipe-arch, and arch sections, may be taken as given in Appendix A12.

Dimensions, properties of pipe cross-sections, and material properties for steel-reinforced thermoplastic culverts shall be provided by the pipe manufacturer.

12.7.2.2-Thrust

The factored thrust, T_L , per unit length of wall shall be taken as:

$$T_L = \frac{P_{FD}(S)}{2} + \frac{P_{FL}(C_L)F_1}{2}$$
 (12.7.2.2-1)

in which:

$$C_{t} = \ell_{w} \le S \tag{12.7.2.2-2}$$

· for corrugated metal pipe:

$$F_1 = \frac{0.75S}{\ell_w} \ge F_{\min} \tag{12.7.2.2-3}$$

$$F_{\min} = \frac{15}{12(S)} \ge 1 \tag{12.7.2.2-4}$$

· for long-span corrugated metal structures:

$$F_1 = \frac{0.54(S)}{\frac{w_t}{12} + LLDF(H) + 0.03(S)}$$
(12.7.2.2-5)

where:

C_L = width of culvert on which live load is applied parallel to span (ft)

LLDF = live load distribution factor as specified in Article 3.6.1.2.6

 ℓ_w = live load patch length at depth H as specified in Article 3.6.1.2.6

 P_{FD} = factored dead load vertical crown pressure as specified in Article 12.12.3.4 with VAFtaken as 1.0 and D_o taken as S (ksf)

C12.7.2.1

Steel-reinforced thermoplastic culverts are pipe sections in which the main load-carrying members are steel ribs or corrugations encapsulated by thermoplastic material that may brace the ribs or corrugations from distortion and buckling. This composite system should be evaluated independently for each manufacturer's pipe system. Designers should obtain the required mechanical properties directly from the pipe manufacturer to determine fill heights.

C12.7.2.2

Factored vertical crown pressure is calculated as the factored free-field soil pressure at the elevation of the top of the structure, plus the factored live load pressure distributed through the soil cover to the top of the structure.

 P_{FL} = factored live <u>load vertical crown</u> pressure as specified in Article 12.12.3.4 (ksf)

S = culvert span (ft)

 T_L = factored thrust per unit length (kip/ft)

 w_t = tire patch width as specified in Article 3.6.1.2.5 (in.)

12.7.2.3—Wall Resistance

The factored axial resistance, R_n , per unit length of wall, without consideration of buckling, shall be taken as:

$$R_n = \phi F_v A \tag{12.7.2.3-1}$$

where:

 $A = \text{wall area (in.}^2/\text{ft)}$

 F_y = yield strength of metal (ksi)

φ = resistance factor as specified in Article 12.5.5

12.7.2.4—Resistance to Buckling

The wall area, calculated using Eq. 12.7.2.3-1, shall be investigated for buckling. If $f_{cr} < F_y$, A shall be recalculated using f_{cr} in lieu of F_y .

If
$$S < \left(\frac{r}{k}\right) \sqrt{\frac{24E_m}{F_u}}$$
, then $f_{cr} = F_u - \frac{\left(\frac{F_u k S}{r}\right)^2}{48E_m}$ (12.7.2.4-1)

If
$$S > \left(\frac{r}{k}\right)\sqrt{\frac{24E_m}{F_u}}$$
, then $f_{cr} = \frac{12E_m}{\left(\frac{kS}{r}\right)^2}$ (12.7.2.4-2)

where:

S = diameter of pipe or span of plate structure (in.)

 $E_m = \text{modulus of elasticity of metal (ksi)}$

 F_u = tensile strength of metal (ksi)

 f_{cr} = critical buckling stress (ksi)

r = radius of gyration of corrugation (in.)

k = soil stiffness factor taken as 0.22

12.7.2.4.1—Critical Compressive Stress

For steel-reinforced thermoplastic culvert, stub compression test $(AASHTO\ T\ 341)$ data shall be provided to establish the f_{cr} of the wall profile being evaluated. If f_{cr} value is less than the value established by Article 12.7.2.4, it shall be used as the limiting critical compressive stress for the pipe wall.

C12.7.2.4

The use of 0.22 for the soil stiffness is thought to be conservative for the types of backfill material allowed for pipe and arch structures. This lower bound on soil stiffness has a long history of use in previous editions of the Standard Specifications.

12.7.2.5—Seam Resistance

For pipe fabricated with longitudinal seams, the factored resistance of the seam shall be sufficient to develop the factored thrust in the pipe wall, T_{ℓ} .

12.7.2.6—Handling and Installation Requirements

Handling flexibility shall be indicated by a flexibility factor determined as:

$$FF = \frac{S^2}{E_m I}$$
 (12.7.2.6-1)

Values of the flexibility factors for handling and installation shall not exceed the values for steel and aluminum pipe and plate pipe structures as specified in Article 12.5.6.

12.7.2.7—Profile Evaluation for Steel-Reinforced Thermoplastic Culverts

To assure the adequacy of the thermoplastic liner, the culvert manufacturer shall provide the results of a three-dimensional finite element analysis of the profile that has been calibrated against results of full-scale tests. A minimum of two full-scale tests are required to properly calibrate the results of the finite element analysis. In particular, the measurement and prediction of the maximum strains within the profile shall be identified. The predicted long-term tensile strains within the profile shall be within the allowable limits for the HDPE material being used in the profile. The strain limits for polyethylene materials shall be taken as specified in Table 12.12.3.3-1.

Additionally, in order to establish the relative long-term interaction of the steel reinforcement with the HDPE profile, the stub compression test (AASHTO T 341) shall be performed utilizing stroke rates of 0.05 in./minute, 0.005 in./minute, and 0.0005 in./minute and the results compiled to determine if a reduction in the HDPE modulus results in a reduced result from the stub compression tests. If a reduction factor is deemed to be appropriate, the reduced value shall be used as described in Article 12.7.2.4.1.

12.7.3—Smooth Lined Pipe

Corrugated metal pipe composed of a smooth liner and corrugated shell attached integrally at helical seams, spaced not more than 30.0 in. apart, may be designed on the same basis as a standard corrugated metal pipe having the same corrugations as the shell and a weight per ft not less than the sum of the weights per ft of liner and helically corrugated shell.

The pitch of corrugations shall not exceed 3.0 in., and the thickness of the shell shall not be less than 60 percent of the total thickness of the equivalent standard pipe.

C12.7.2.6

Transverse stiffeners may be used to assist corrugated structural plate structures to meet flexibility factor requirements.

C12.7.2.7

The full-scale tests should follow the loading conditions, measurements, and test methods consistent with those utilized in the research report *DuroMaxx Pipe Assessment* by I. D. Moore (24 in. – February 2009, 60 in. – August 2009) or similar approved method.

While the steel ribs or corrugations are the main load-carrying member of the culvert, the thermoplastic profile braces the steel ribs or corrugations from distortion or buckling under load and is critical to the pipe performance. The liner also serves to distribute the load between ribs or corrugations. A structural evaluation of the profile alone is not required. However, an evaluation of the composite system of thermoplastic liner and steel rib or corrugation is necessary. It is important to assure that the tensile strains within the profile do not exceed the long-term strain capacity for the thermoplastic material used in the construction of the pipe.

Table A12-2—Spiral Rib Steel Pipe—Cross-Section Properties

3/4 >	$\frac{3}{4} \times \frac{3}{4} \times 7 \frac{1}{2}$ in. Corrugation						
Thickness (in.)	A (in. ² /ft)	r (in.)	$I \times 10^{-3}$ (in.4/in.)				
0.064	0.509	0.258	2.821				
0.079	0.712	0.250	3.701				
0.109	1.184	0.237	5.537				
0.138	1.717	0.228	7.433				

3/4	$\frac{3}{4} \times 1 \times 11 \frac{1}{2}$ in. Corrugation					
Thickness (in.)	A (in. ² /ft)	r (in.)	$I \times 10^{-3}$ (in.4/in.)			
0.064	0.374	0.383	4.58			
0.079	0.524	0.373	6.08			
0.109	0.883	0.355	9.26			

Note: Effective section properties are taken at full yield stress.

Table A12-3—Steel Structural Plate—Cross-Section Properties

	6×2 in. Corrugations					
Thickness (in.)	A (in. ²)	r (in.)	I (in. ⁴ /in. × 10 ⁻³)			
0.110	1.556	0.682	60.4			
0.140	2.003	0.684	78.2			
0.170	2.449	0.686	96.2			
0.188	2.739	0.688	108.0			
0.218	3.199	0.690	126.9			
0.249	3.650	0.692	146.2			
0.280	4.119	0.695	165.8			
0.318	4.671	0.698	190.0			
0.380	5.613	0.704	232.0			

Table A12-4—Corrugated Aluminum Pipe—Cross-Section Properties

	1 ½ × ¼ in. Corrugation					
Thickness (in.)	A (in. ² /ft)	r (in.)	$I \times 10^{-3}$ (in.4/in.)			
0.048	0.608	0.0824	0.344			
0.060	0.761	0.0832	0.349			

2	$2^{2}/_{3} \times \frac{1}{2}$ in. Corrugation						
Thickness (in.)	A (in. ² /ft)	r (in.)	$I \times 10^{-3}$ (in.4/in.)				
0.060	0.775	0.1712	1.892				
0.075	0.968	0.1721	2.392				
0.105	1.356	0.1741	3.425				
0.135	1.745	0.1766	4.533				
0.164	2.130	0.1795	5.725				

Table A12-8—Minimum Longitudinal Seam Strengths Steel and Aluminum Structural Plate Pipe—Bolted

	6 × 2 in. Steel Structural Plate Pipe						
Bolt Thickness (in.)	Bolt Diameter (in.)	4 Bolts/ft (kip/ft)	6 Bolts/ft (kip/ft)	8 Bolts/ft (kip/ft)			
0.109	3/4	43.0	-				
0.138	3/4	62.0	_	_			
0.168	3/4	81.0	<u> </u>				
0.188	3/4	93.0	_				
0.218	3/4	112.0	1—1	_			
0.249	3/4	132.0	_	S			
0.280	3/4	144.0	180.0	194.0			
0.318	7/8	<u> </u>	7 1	235.0			
0.380	7/8	_	_	285.0			

	9 ^ 2 1/2 III. A	luminum Structural Plate Pipe	71.77 10 2000 300
Thickness (in.)	Bolt Diameter (in.)	Steel Bolts 5.5 Bolts per ft (kip/ft)	Aluminum Bolts 5.5 Bolts per ft (kip/ft)
0.100	3/4	28.0	26.4
0.125	3/4	41.0	34.8
0.150	3/4	54.1	44.4
0.175	3/4	63.7	52.8
0.200	3/4	73.4	52.8
0.225	3/4	83.2	52.8
0.250	3/4	93.1	52.8

Table A12-9—Mechanical Properties for Spiral Rib and Corrugated Metal Pipe and Pipe Arch

Material	Minimum Tensile Strength, F_u (ksi)	Minimum Yield Stress, F_y (ksi)	Modulus of Elasticity, E_m (ksi)
Aluminum H34 ^{(1)&(4)}	31.0	24.0	10,000
Aluminum H32 ^{(2)&(4)}	27.0	20.0	10,000
Steel ⁽³⁾	45.0	33.0	29,000

- Shall meet the requirements of AASHTO M 197 (ASTM B744), for Alclad Alloy 3004-H34
 Shall meet the requirements of AASHTO M 197 (ASTM B744), for Alclad Alloy 3004-H32
 Shall meet the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M), M 218, and M 246 (ASTM A742)
- 4. H34 temper material shall be used with riveted pipe to achieve seam strength. Both H32 and H34 temper material may be used with helical pipe

Table A12-10-Mechanical Properties-Corrugated Aluminum and Steel Plate

Material	Minimum Tensile Strength (ksi)	Minimum Yield Stress (ksi)	Modulus of Elasticity (ksi)
Aluminum(1) Plate Thickness (in.)			
0.100-0.175	35.0	24.0	10,000
0.176-0.250	34.0	24.0	10,000
Steel ⁽²⁾ Plate Thickness (in.)	20	2	
All	45.0	33.0	29,000
Steel Deep Corrugated Plate	55.0	44.0	29,000

- Shall meet the requirements of AASHTO M 219 (ASTM B746), Alloy 5052 Shall meet the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M)

Table A12-11—PE Corrugated Pipes (AASHTO M 294)

Nominal Size (in.)	Min. ID (in.)	Max. OD (in.)	Min. A (in. ² /ft)	Min. c (in.)	Min. <i>I</i> (in. ⁴ /in.)
12	11.8	14.7	1.5	0.35	0.024
15	14.8	18.0	1.9	0.45	0.053
18	17.7	21.5	2.3	0.50	0.062
24	23.6	28.7	3.1	0.65	0.116
30	29.5	36.4	3.9	0.75	0.163
36	35.5	42.5	4.5	0.90	0.222
42*	41.5	48.0	4.69	1.11	0.543
48*	47.5	55.0	5.15	1.15	0.543

^{*} For the 42.0-in. and 48.0-in. pipe, the wall thickness should be designed using the long-term tensile strength provision, i.e., 900 psi, until new design criteria are established in the AASHTO bridge and structures specifications.

Table A12-12—PE Ribbed Pipes (ASTM F894)

					23550	in. <i>I</i> ⁴ /in.)
Nominal Size (in.)	Min. ID (in.)	Max. OD (in.)	Min. A (in.²/ft)	Min. c (in.)	Cell Class 334433C	Cell Class 335434C
18	17.8	21.0	2.96	0.344	0.052	0.038
21	20.8	24.2	4.15	0.409	0.070	0.051
24	23.8	27.2	4.66	0.429	0.081	0.059
27	26.75	30.3	5.91	0.520	0.125	0.091
30	29.75	33.5	5.91	0.520	0.125	0.091
33	32.75	37.2	6.99	0.594	0.161	0.132
36	35.75	40.3	8.08	0.640	0.202	0.165
42	41.75	47.1	7.81	0.714	0.277	0.227
48	47.75	53.1	8.82	0.786	0.338	0.277



7.2.4 Live Loads

7.2.4.1A HL-93 Design Load is the LRFD Design Live Load as per Appendix C6A of the AASHTO Manual for Bridge Evaluation and shall be analyzed to determine a rating factor.

Rating Vehicles shall be as follows:

H20 truck	Two Axle	20 Tons
Type 3 truck	Three Axle	25 Tons
Type 3S2 truck	Five Axle	36 Tons
SU4 ¹	Four Axle	27 Tons
SU51	Five Axle	31 Tons
SU61	Six Axle	34.75 Tons
SU7 ¹	Seven Axle	38.75 Tons
Type EV2 ²	Two Axle	28.75 Tons
Type EV3 ²	Three Axle	43 Tons

Please note that MassDOT defines Posting Vehicles as trucks whose load ratings are used when a bridge is posted. MassDOT currently uses the following posting trucks for posting purposes at Inventory Level:

H20 truck	Two Axle	20 Tons
Type 3 truck	Three Axle	25 Tons
Type 3S2 truck	Five Axle	36 Tons

Note 1: NCHRP Report 575 investigated the current truck configurations operating nationwide and determined that the AASHTO Legal Loads underestimate the load effects of the actual Specialized Hauling Vehicles (SHVs) currently operating in most states. In 2005, AASHTO adopted the SU4, SU5, SU6, and SU7 truck models which are intended to capture the effects of these SHVs.

Note 2: Type EV2 and Type EV3 have been added to the Rating Vehicles as a result of the implementation of Fixing America's Surface Transportation Act (FAST Act) signed into law by the President on December 4, 2015. This act provided an exemption for emergency vehicles from the nationwide Interstate truck weight limits set forth in 23 U.S.C. 127(a). This requirement applies to all bridges within reasonable access to the Interstate System.

MassDOT has chosen to rate the interior beams of all bridges for the effects of Fast Act Emergency Vehicle loadings. The Rating Engineer may need to consider the first interior roadway beam and exterior safety curb beams depending upon the actual roadway lane striping. Additionally, superstructure members supporting these beams (e.g. floorbeams, trusses, etc.) will need to be rated for these vehicles.



- 7.2.4.8 Curbs with height less than 12 inches shall be considered mountable. The beams supporting a mountable sidewalk, mountable median, or mountable safety walk with a width greater than 2 feet measured from the face of the bridge rail to the curb line shall be rated by placing a wheel line 2 feet from the face of the bridge rail. If the above referenced width is 2 feet or less, the wheel line shall be placed 2 feet from the face of the curb. This rating shall be performed at the Operating Level. The Inventory Rating shall always be calculated with the wheel line located in the travelway 2 feet from the face of the curb. Refer to Paragraph 7.2.4.5 for Alternative Load Rating using Actual Lane Location procedures. Refer to Chapter 3, Paragraph 3.5.3.11, Case II for guidance regarding the application of the HL-93 loading for this situation.
- 7.2.4.9 Pedestrian Load will generally not be included in ratings, unless, based on engineering judgment, its application will produce the maximum anticipated loading. For structural members supporting both sidewalk loads and vehicular traffic, the probability is low for full loading on both the sidewalk and bridge; therefore, only Operating Ratings, including Pedestrian Load, need to be performed. This rating shall be reported in the Breakdown of Bridge Rating and omitted from the Summary of Bridge Rating.

7.2.5 Special Instructions for Load Ratings

- 7.2.5.1 Any request for clarification of, or deviation from, these guidelines must be submitted in writing (email is acceptable) to the State Bridge Engineer. Written responses will be provided.
- 7.2.5.2A Condition Factors of the AASHTO Manual for Bridge Evaluation, Article 6A.4.2.3 shall not be used in the calculations of the structural capacity. The structural capacity of the section being investigated shall be based on the field conditions.
- 7.2.5.3A System Factors of the AASHTO Manual for Bridge Evaluation, Article 6A.4.2.4 shall be included in the capacity calculations of the non-redundant structure for the section being investigated. Redundant secondary members within a non-redundant structure shall not have their capacities reduced by the same system factor. For example, a bridge comprised of two girders, floorbeams, and stringers shall use a system factor of 0.85 for the girders, 1.0 for the floorbeams, if they are spaced less than or equal to 12 feet, and 1.0 for stringers (refer to Chapter 3, Paragraph 3.6.1.6).
- 7.2.5.4 Pile bent structures constructed of steel piles, timber piles, or concrete piles, including their pile caps, shall be rated. Other non-reinforced concrete substructures, such as steel frames or substructures that include steel cross girder members, shall also be rated.

Typically, reinforced concrete substructures such as multi-column piers, single column hammerhead piers, solid wall piers and concrete abutments, do not need to be rated because they have sufficient capacity. However, in cases where these types of substructures have undergone deterioration in critical areas that has, in the opinion of the Rating Engineer, reduced their load carrying capacity significantly enough to influence the overall rating of the bridge, then the Rating Engineer shall consult with the MassDOT Ratings and Overload Engineer regarding the need for rating these substructures. This deterioration shall include deterioration of bridge seats and pedestals which has undermined the bridge bearings.

In either case, the report shall contain a statement noting the Rating Engineer's judgment with regards to the substructure.

Engineering judgment alone shall not be accepted as a valid method for rating superstructure elements. For structures with unknown structural detail and lack of plans, detailed field measurements, non-destructive testing, and a material testing program shall be performed.

For such situations, a program of material sampling and testing shall be developed and submitted to the State Bridge Engineer for approval prior to performing the testing. All material sampling and testing shall be performed in accordance with the latest ASTM and AASHTO Standards.

- For structures without the necessary details, such as concrete slabs with unknown reinforcing size and spacing, and with difficult access for the taking of samples as required by Paragraph 7.2.5.5 above, the Rating Engineer shall contact the Bridge Section for guidance.
- 7.2.5.7 If a beam supporting a raised median rates below statutory levels, the Rating Engineer shall apply the provisions of Paragraph 7.2.4.5 above.
- 7.2.5.8B All timber structures shall be rated using the Allowable Stress Design methodology. Where the actual species and grade of lumber are unknown, the Rating Engineer shall determine the species and grade by field observation and/or testing.

The Allowable Inventory Stresses for various timber species and grades and the appropriate adjustment factors shall be taken from Section 10 of the AASHTO LRFD Bridge Design Specifications: The values used for Allowable Operating Unit Stresses shall be equal to 1.33 times the values determined for the Allowable Inventory Unit Stresses.

- 7.2.5.9 Tire Contact Area Dimensions. The Tire Contact Area for a given rating vehicle wheel shall be calculated by dividing the reaction of the wheel by an assumed tire pressure of 80 psi. The length of this Tire Contact Area shall be taken as 10" for all vehicle wheels and the width shall be calculated by dividing the calculated Tire Contact Area by this width.
- 7.2.5.10 BrR can only model parabolic and linear varying web depths for reinforced concrete Tbeam superstructures. If a beam's web depth varies along a circular curve, the concrete T-beams can only be modeled in BrR using cross sections and cross-sectional ranges with linear varying web depths.
- 7.2.5.11 Unless there is a mix formula or design strength given on the plans, concrete for superstructures shall be assumed to have an f'c equal to 2000 psi for structures built prior to 1931; 3000 psi for structures built between 1931 and 1984; and 4000 psi for structures built after 1984. If a mix proportion is given on the plans, the compressive strengths shall be taken from the 1916 Joint Committee Report as shown in the following Table.

Mix	1:1:2	1:11/2:3	1:2:4	1:21/2:5	1:3:6
f'c	3000 psi	2500 psi	2000 psi	1600 psi	1300 psi



LOADINGS USED FOR BRIDGE RATING

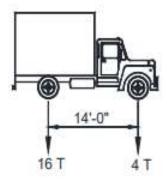
DANA-PRESCOTT

MAIN STREET / SWIFT RIVER

BRIDGE NO. D-02-033=P-15-015

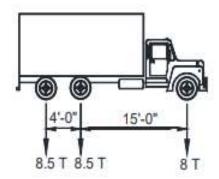
H20 VEHICLE

TOTAL WEIGHT 20 TONS



TYPE 3 VEHICLE

TOTAL WEIGHT 25 TONS



TYPE 3S2 VEHICLE

TOTAL WEIGHT 36 TONS

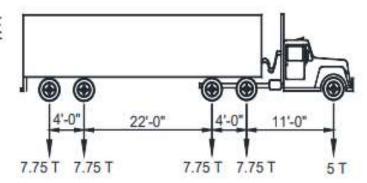


Figure 7.10A: Vehicle Diagrams (LRFR)



LOADINGS USED FOR BRIDGE RATING

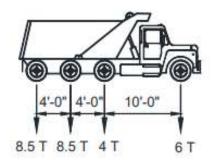
DANA-PRESCOTT

MAIN STREET / SWIFT RIVER

BRIDGE NO. D-02-033=P-15-015

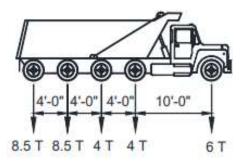
SU4 TRUCK

TOTAL WEIGHT 27 TONS



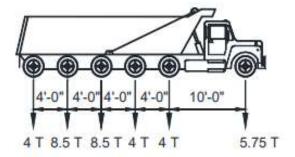
SU5 TRUCK

TOTAL WEIGHT 31 TONS



SU6 TRUCK

TOTAL WEIGHT 34.75 TONS



SU7 TRUCK

TOTAL WEIGHT 38.75 TONS

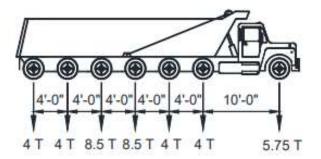


Figure 7.11: Vehicle Diagrams - Specialized Hauling Vehicles

LOADINGS USED FOR BRIDGE RATING

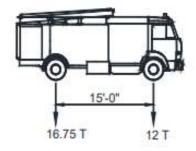
DANA-PRESCOTT

MAIN STREET / SWIFT RIVER

BRIDGE NO. D-02-033=P-15-015



TOTAL WEIGHT 28.75 TONS



EV3 VEHICLE

TOTAL WEIGHT 43 TONS

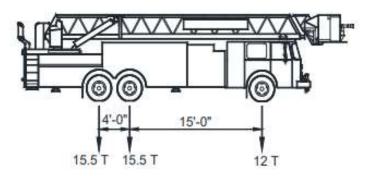


Figure 7.12: Vehicle Diagrams - Emergency Vehicles

listed within LRFD Design is applicable for design loadings (HL-93) and legal trucks only, not for permit trucks. As a result, the MPF for permit trucks was not considered during the LRFR calibration of permit loads. The MPF should not be considered when establishing the load demand of either routine or special permit trucks. It is important to note that when comparing the simplified live load distribution factors to establish the demand of routine permit trucks, the (MPF, m) of 1.2 included within the simplified single-lane distribution factor equation should be removed.

Article 6A.4.5.4.2c addresses the use of refined analysis for permits and provides an adjustment to the load factors when using refined analysis methods (such as 3-D analysis). Again, it is not necessary to incorporate the (MPF, m) for permit trucks during evaluation.

6A.4.2—General Load-Rating Equation

6A.4.2.1—General

The following general expression shall be used in determining the load rating of each component and connection subjected to a single force effect (i.e., axial force, flexure, or shear):

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_{P})(P)}{(\gamma_{LL})(LL + lM)}$$
(6A.4.2.1-1)

For the strength limit states:

$$C = \varphi_c \varphi_c \varphi R_u \qquad (6A.4.2.1-2)$$

Where the following lower limit shall apply:

$$\varphi_c \varphi \ge 0.85$$
 (6A.4.2.1-3)

For the service limit states:

$$C = f_{\mu}$$
 (6A.4.2.1-4)

where:

RF = Rating factor

C = Capacity

 $f_R = \text{Allowable stress specified in the LRFD code}$

 R_n = Nominal member resistance (as inspected)

DC = Dead load effect due to structural components and attachments

C6A.4.2.1

It should be noted that load modifiers, η , relating to ductility, redundancy, and operational importance contained in Article 1.3.2.1 of the AASHTO LRFD Bridge Design Specifications are not included in the general load-rating equation. In load rating, ductility is considered in conjunction with redundancy and incorporated in the system factor, φ_s . Operational importance is not included as a factor in the LRFR load rating provisions.

The load rating of a deteriorated bridge should be based on a recent thorough field inspection. Only sound material should be considered in determining the nominal resistance of the deteriorated section. Load ratings may also be calculated using as-built member properties to serve as a baseline for comparative purposes.

Resistance factor, ϕ , has the same value for new design and for load rating. Also, $\phi = 1.0$ for all nonstrength limit states. For condition factors, see Article 6A.4.2.3. For system factors, see Article 6A.4.2.4.

Table 6A.4.2.2-1—Limit States and Load Factors for Load Rating

				Design	n Load		
		Dead Load	Dead Load	Inventory	Operating	Legal Load	Permit Load
Bridge Type	Limit State*	YDC	YDW.	YLL	YLL	YLL	YLL
	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	-
Steel	Strength II	1.25	1.50		====		Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.80	220		
Reinforced	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	F-3
Concrete	Strength II	1.25	1.50	3-3	====		Table 6A.4.5.4.2a-1
	Service I	1.00	1.00		-		1.00
	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	
Prestressed	Strength II	1.25	1.50		-		Table 6A.4.5.4.2a-1
Concrete	Service III	1.00	1.00	0.80Table 6A.4.2.2-2	-	1.00	1 2=3
	Service I	1.00	1.00				1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	-
	Strength II	1.25	1.50	19-3			Table 6A.4.5.4.2a-1

^{*} Defined in the AASHTO LRFD Bridge Design Specifications

Notes:

- Shaded cells of the table indicate optional checks.
- Service I is used to check the 0.9 F_y stress limit in reinforcing steel.
- Load factor for DW at the strength limit state may be taken as 1.25 where thickness has been field measured.
- · Fatigue limit state is checked using the LRFD fatigue truck (see Article 6A.6.4.1).

Table 6A.4.2.2-2—Load Factors for Live Load for the Service III Load Combination, γιι, at the Design-Load Inventory Level

Component	7//
Prestressed concrete components rated using the refined estimates of time- dependent losses as specified in LRFD Design Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

6A.4.2.3—Condition Factor: φ.

Use of Condition Factors as presented below may be considered optional based on an agency's load-rating practice.

The condition factor provides a reduction to account for the increased uncertainty in the resistance of deteriorated members and the likely increased future deterioration of these members during the period between inspection cycles.

Table 6A.4.2.3-1—Condition Factor: 9c

Structural Condition of Member	φ _c
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

C6A.4.2.3

The uncertainties associated with the resistance of an existing intact member are at least equal to that of a new member in the design stage. Once the member experiences deterioration and begins to degrade, the uncertainties and resistance variabilities are greatly increased (scatter is larger).

Additionally, it has been observed that deteriorated members are generally prone to an increased rate of future deterioration when compared to intact members. Part of ϕ_c relates to possible further section losses prior to the next inspection and evaluation.

Improved inspections will reduce, but not totally eliminate, the increased scatter or resistance variability in deteriorated members. Improved inspection and field measurements will reduce the uncertainties inherent in

Table 6A.4.2.4-1—System Factor: φ, for Flexural and Axial Effects

Superstructure Type	φέ
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing>12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

If the simplified system factors presented in Table 6A.4.2.4-1 are used, they should be applied only when checking flexural and axial effects at the strength limit state of typical spans and geometries.

A constant value of $\varphi_0 = 1.0$ is to be applied when checking shear at the strength limit state.

For evaluating timber bridges, a constant value of $\varphi_0 = 1.0$ is assigned for flexure and shear.

If Table 6A.4.2.4-1 is used, the system factors are used to maintain an adequate level of system safety. Nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. The aim of φ_z is to add a reserve capacity such that the overall system reliability is increased from approximately an operating level (for redundant systems) to a more realistic target for nonredundant systems corresponding to inventory levels.

If the Engineer can demonstrate the presence of adequate redundancy in a superstructure system (Reference: NCHRP Report 406), then φ_s may be taken as 1.0. In some instances, the level of redundancy may be sufficient to utilize a value of φ_s greater than 1.0, but in no instance should φ_s be taken as greater than 1.2.

A more liberal system factor for nonredundant riveted sections and truss members with multiple eyebars has been provided. The internal redundancy in these members makes a sudden failure far less likely. An increased system factor of 0.90 is appropriate for such members.

Some agencies may consider all three-girder systems, irrespective of girder spacing, to be nonredundant. In such cases, φ, may be taken as 0.85 for welded construction and 0.90 for riveted construction.

Subsystems that have redundant members should not be penalized if the overall system is nonredundant. Thus, closely spaced parallel stringers would be redundant even in a two-girder-floorbeam main system.

For narrow bridges (such as one-lane bridges) with closely spaced three-and four-girder systems, all the girders are almost equally loaded and there is no reserve strength available. Therefore, φ_a is decreased to 0.85.

For the purposes of determining system factors, each web of a box girder may be considered as an I-girder.

System factors are generally not appropriate for shear, as shear failures tend to be brittle, so system reserve is not possible. The design resistance, factored for shear, should be calibrated to reflect the brittle characteristics. Thus, in the evaluation, all the ϕ_a should be equal. A constant value of $\phi_a = 1.0$ is assigned for evaluation.

More accurate quantification of redundancy is provided in NCHRP Report 406, Redundancy in Highway Bridge Superstructures. Tables of system factors are given in the referenced report for common simple-span and continuous bridges with varying number of beams and beam spacings. For bridges with configurations that are not covered by the tables, a direct redundancy analysis approach may be used, as described in NCHRP Report 406.

6A.4.3—Design-Load Rating

6A.4.3.1-Purpose

The design-load rating assesses the performance of existing bridges utilizing the LRFD-design loading (HL-93) and design standards. The design-load rating of bridges may be performed at the same design level

C6A.4.3.1

The design-load rating is performed using dimensions and properties for the bridge in its present condition, obtained from a recent field inspection.

No further evaluation is necessary for bridges that have

Table 6A.5.12.5-1-Limit States and Load Factors for Culvert Load Rating

		DC	V	MO		Design	Design Load			577	-	EFF	*	EV		M	ESI
		Max	Min	Max	Min	Inv.	Opr.	Legal Load*	Permit	Max	Min	Max	Min	Max	Min	Max	Min
Bridge Type	Limit State	Jul.	YOU	700	Your	TIL.	,ta	17.7	т	20.5	Yes	7EH	YEAT.	Yer	YEF	765	728
Reinforced	Strength I	123	06.0	1.50	0.65	1.75	1.35	2.00		Same as LF for Design/ Legal Loads	000	133	06'0	1.30 0.90		1.50	0.75
Concrete Box Culvert	Strength II	1.25	06.0	1.50	0.65	(3)		1)	Same a Table 6A. LF for 4.5.4.2a-1 Permit Loads	Same as LF for Permit Loads	0.00	1.35	0.00	130	0.90	1.50	0.75

Notes:

In addition to the load factor, use the 1.2 multiple presence factor for single-lane loading

Multiple presence factor is not included and is not required for single-lane loading

Use a 50 percent reduction to EH for rating positive moment in top slabs; need not be combined with the minimum load factor

Use a 50 percent reduction to ES for rating positive moment in top slabs; need not be combined with the minimum load factor. Water loads on interior walls are neglected.

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