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Augusta, Maine 04333

MaineDOT

Transportation Research Division

Technical Report 18-3

Live Load Testing and Load Rating of Five Reinforced Concrete T-Beam Bridges

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16. Abstract (Limit 200 words)

Five reinforced T-beam bridges were tested during the summer of 2017 by the University of Maine (UMaine) as part of this program for the Maine Department of Transportation (MaineDOT):

1. Canton No. 3356 School Street over Whitney Brook,

2. Peru No. 5432 Ridge Road over Spears Stream,

3. Jackson No. 3776 Village Road over Marsh Stream (North Branch),

4. Alna No. 2130 Route 218 over Carlton (Trout) Brook,

5. Franklin No. 3307 Route 200 over Card's Mill Stream.

Revised load ratings were computed using data collected during live load testing. Details of bridge instrumentation, load cases, and strain plots for each bridge are provided in Appendices A.1 to A.6 inclusive. The results of the tests and analyses are summarized below and are compared with the existing ratings. Use of these revised load ratings, live load test data, and extrapolation of these results to other structures is at the sole discretion of the bridge owner.

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Live Load Testing and Load Rating of Five Reinforced Concrete T-Beam Bridges

Prepared for:
Dale Peabody P.E.
Director Transportation Research
Maine Dept. of Transportation
16 State House Station
Augusta ME 04333-0016
University of Maine's Advanced Structures and Composites Center
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Prepared by:
Andrew Schanck EI
Graduate Researcher

William Davids PhD, PE
John C. Bridge Professor of Civil and Environmental Engineering

Reviewed by:

Scott Tomlinson PE
Research Engineer

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The University of Maine Advanced Structures and Composites Center is an ISO 17025 accredited testing laboratory, accredited by the International Accreditation Service.

Advanced Structures and Composites Center
35 Flagstaff Rd
University of Maine
Orono, ME 04469

Telephone: 207-581-2123
FAX: 207-581-2074
composites@maine.edu
www.composites.umaine.edu

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Executive Summary

Five reinforced T-beam bridges were tested during the summer of 2017 by the University of Maine (UMaine) as part of this program for the Maine Department of Transportation (MaineDOT):

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Revised load ratings were computed using data collected during live load testing. Details of bridge instrumentation, load cases, and strain plots for each bridge are provided in Appendices A.1 to A.6 inclusive. The results of the tests and analyses are summarized below and are compared with the existing ratings. Use of these revised load ratings, live load test data, and extrapolation of these results to other structures is at the sole discretion of the bridge owner.

1. Canton No. 3356: On July 11, 2017, maximum applied loading produced 75% of HL-93 flexural service load with impact. The rating factors per AASHTO were 0.98 for interior, 2.48 for the original exterior girder, and 0.28 for the extended exterior girder. The extended exterior girder made this span the lowest capacity bridge tested. Applied loads were greater than the predicted capacity. Not surprisingly, measured extended exterior girder strains were the highest for any of the bridges and had the smallest compressive strength window for uncracked behavior. Using the provisions of the AASHTO *Manual for Bridge Evaluation* (AASHTO 2012), the rating factor for HL-93 was increased to 1.39 for the interior girders, 2.48 for the original exterior girder, and 0.30 for the extended girder.
2. Peru No. 5432: On July 13, 2017 87% of HL-93 flexural service loading with impact was produced from maximum loading. This was the highest loading for all bridges. The live load rating factors per AASHTO were 0.75 for the interior girders and 2.04 for the exterior girders. Overall, the strains measured at this bridge were highest, although it still exhibited uncracked section behavior. Rating factors were increased for this structure to 1.10 for interior girders and 2.55 for exterior girders, bringing this bridge to an acceptable operating flexural rating.
3. Jackson No. 3776: On July 18, 2017 70% of HL-93 flexural service loading with impact was produced for this under maximum loading. The initial rating factors per AASHTO were 0.69 for interior girders and 2.08 for exterior girders. Fairly low strain was produced compared to other bridges in the group due to the relatively small percentage of HL-93 loading, but rating factor increases were still allowable and uncracked section behavior was observed. Live load testing results allowed the rating factors to be increased to 1.20 for interior girders and 2.40 for exterior girders, bringing this bridge's flexural rating factors to acceptable values.
4. Alna No. 2130: On July 20, 2017 75% of HL-93 service flexural loading with impact was achieved under maximum applied load. This structure had near-acceptable AASHTO calculated rating factors of 0.92 for the interior and 1.01 for the exterior girders. Uncracked

section behavior was observed, and rating factor increases to 1.28 and 1.98 for interior and exterior girders, respectively, were justified, bringing this bridge's flexural rating factors to acceptable values.

5. Franklin No. 3307: On July 25, 2017 maximum applied loading produced 84% of HL-93 service flexural loading with impact. This bridge exhibited very low strains as compared with those predicted, exhibiting uncracked behavior and justifying rating factor increases from 0.92 and 2.78 to 1.61 and 5.03 for interior and exterior girders respectively. These increases brought this bridge's flexural rating factors to acceptable values.

1 Bridge Testing Program

Five reinforced concrete T-beam bridges were tested during the summer of 2017 as part of this program:

1. Canton No. 3356 School Street over Whitney Brook,
2. Peru No. 5432 Ridge Road over Spears Stream,
3. Jackson No. 3776 Village Road over Marsh Stream (North Branch),
4. Alna No. 2130 Route 218 over Carlton (Trout) Brook,
5. Franklin No. 3307 Route 200 over Card's Mill Stream.

All bridges were instrumented with a strain measuring system, loaded with heavy trucks, and then analyzed to determine whether it was reasonable to change the bridge rating factors based on the test results. These bridges were all constructed between 1936 and 1950, and were originally designed as simply supported with nominal concrete compressive strength of 2.5 ksi. The primary objective of this study was to determine more appropriate live-load rating factors for these bridges and to determine actual live load distribution factors. Recommendations for rating factor modifications are made based on the observed and computed response of these structures. Characteristics of the bridges tested and analyzed in this study are summarized in Table 1.

Table 1: Bridge Characteristics

Bridge	**Canton**	**Peru**	**Jackson**	**Alna**	**Franklin**
Number	3356	5432	3776	2130	3307
Year Built	1936	1950	1941	1939	1941
Span - Center to Center of Bearings (feet)	27.50	40.50	31.00	27.00	43.08
Number of Girders	6	5	5	4	5
Girder Spacing (in)	67.5, 59.5	76	68.63	72,70	68.75
Total depth (in)	28.00	35.75	30.50	33.00	31.00
Girder web thickness (in)	18.5, 14	20.0	19.5	16,12	19
Slab Thickness (in)	6.50	5.75	5.50	8.00	5.75

1.1 Instrumentation

The strain measurement system utilized in this research was the Wireless Structural Testing System (STS-Wi-Fi) produced by Bridge Diagnostics Inc. (BDI). The system used a mobile base station to communicate with up to 6 nodes, with up to 4 strain transducers connected to each node. This system communicated with a dedicated laptop running BDI-specific WinSTS data acquisition software. A sample setup in the field is shown in Figure 1, with strain sensors mounted under the bridge at mid-span connected to battery operated wireless nodes. The sensors used in these tests were equipped with extensions which are also visible in Figure 1. These extensions increased the gauge length of the transducers so as to minimize the effect of local stress concentrations and concrete cracks. A schematic of the entire network is shown in Figure 2 including strain and displacement sensors, wireless nodes, the wireless base station, autoclicker, and the data recording laptop.

Figure 1: Typical strain sensor mounted under bridge, equipped with extension

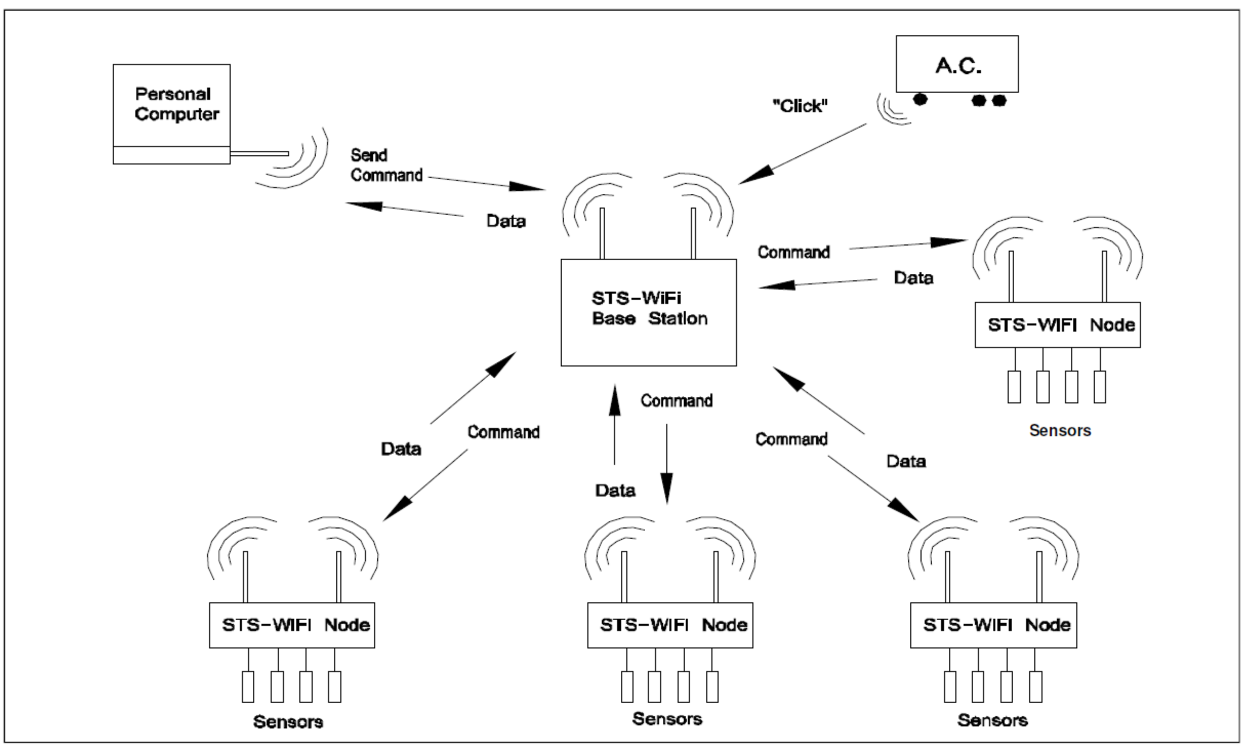

Figure 2: BDI STS-Wi-Fi network setup for bridge sensor setup.

Strain transducers were mounted under the bridges using a MaineDOT Under Bridge Inspection Truck (UBIT) as shown in Figure 3. The sensors were mounted to the girders by first grinding the concrete to be as flat as possible, then using LOCTITE 410 rubberized instant adhesive with LOCTITE SF7453 accelerant to attach the strain transducer mounting tabs to the cleaned concrete. All structures had three strain gages mounted to each girder at midspan - one to the bottom of the slab, one at mid-depth of the web, and one at the web bottom face at mid-span - to give a complete picture of load distribution and peak flexural strains in each girder type: center, interior non-center, and exterior. Strain transducers were also installed near the ends of selected girders (generally exterior and central girders as the number of remaining transducers allowed) to determine the extent of any rotational restraint at the supports. Strain sensor layout varied slightly for some bridges, with individual sensor layouts shown in the appendices A.2.2 for Canton, A.3.2 for Peru, A.4.2 for Jackson, A.5.2 for Alna, and A.6.2 for Franklin.

Figure 3: MaineDOT UBIT used to install sensors

1.2 Loading

The vehicles used for this testing were Maine DOT standard three-axle dump trucks, and/or three-axle trucks provided by contractors, as shown in Figure 4. Each truck wheel or pair of wheels was weighed using state patrol certified portable scales as shown in Figure 5. Various load cases were applied to each bridge, with each test given a specific identification code with the format: "Test Configuration_Test Position_Test Number". Test configurations included two trucks, one in each lane ("SBS"), two trucks in a single lane ("BTB"), four trucks, two in each lane arranged to produce maximum moment ("MAX"), and four trucks, two in each lane arranged to produce less than maximum moment ("ALT"). Test positions included load close to the first curb ("1"), load close to the bridge centerline ("2"), and load close to the opposite curb ("3"). Test number refers to the test index if a certain load case was repeated. In most cases at least three tests were completed for each configuration, one in each position, with the test in position "1" performed twice to verify repeatability. Not all bridges were subjected to all load cases.

Figure 4: Maine DOT (left) and contractor supplied (right) three axle trucks used for loading

Figure 5: State highway patrol certified portable truck scales used to verify vehicle weight for each test

1.3 Typical Results

Results from a representative test of one of the five bridges are presented in this section to overview the general trends seen. Jackson No. 3776 had typical geometry and results for all test configurations. Figure 6 shows a time history of the strains measured at midspan of the center girder during the MAX_2_1 test, and Figure 7 shows a time history of the strains recorded at the ends of the same girder during the same test. In this test, trucks were backed onto the bridge sequentially and came to rest such that two trucks were arranged back to back in each lane, with their rear tandem wheels spaced approximately symmetrically about the midspan, with the trucks in the two lanes spaced about symmetrically with respect to the striped centerline, thereby maximizing applied moment. When all position measurements had been taken, the trucks were then removed from the bridge in reverse order. This sequential loading is seen in the strain plateaus in Figure 6 which clearly demark a truck backing onto, or pulling off the bridge. This figure also demonstrates the typical linear response to flexure seen across all bridges. The sensor at the section bottom recorded modestly high positive (tensile) strain at the maximum strain plateau, while the sensor at the top of the section recorded very small compressive strains and the sensor at the mid-depth of the section roughly split the difference. This strain distribution across the section indicates that section's neutral axis lies in the web, close to the bottom of the slab. The location of the neutral axis within the section, as well as the relatively low strains recorded, indicate that the section behaves as uncracked under test loading and has not experienced significant flexural cracking due to prior loading. Figure 7 shows the typical behavior of girder ends. At both ends of the girder, the bottom of the section experiences small tensile strains while the top of the section experiences very small compressive strains, mimicking the strain distribution at midspan. This indicates that very little, if any, unintended end restraint is present.

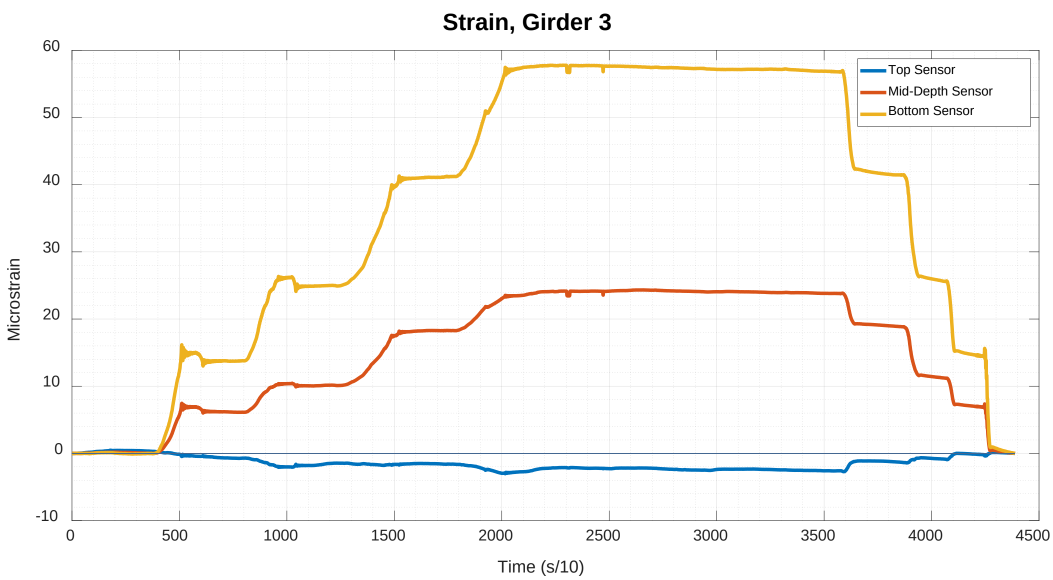

Figure 6: Jackson No. 3776 – MAX_2_1, center girder strains at midspan

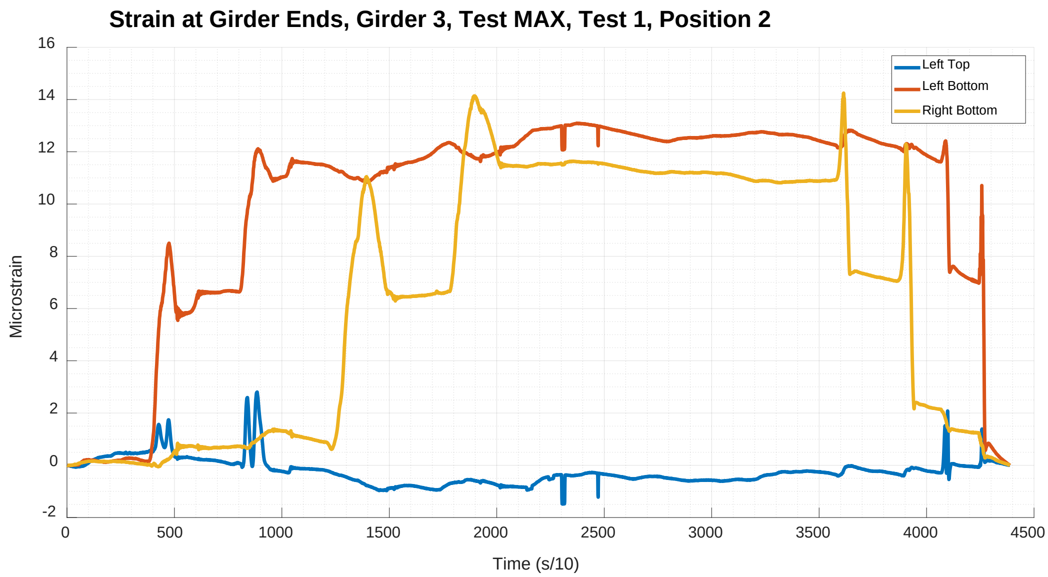

Figure 7: Jackson No. 3776 – MAX_2_1, center girder strains at ends

1.4 Analysis Methodology

1.4.1 Analysis Overview

Material properties, load and resistance factors, and design live loads were taken from or calculated as specified in the AASHTO *Manual for Bridge Evaluation* and used with field-measured geometry to determine original nominal rating factors for each of the bridges. Bridges were then tested using heavily loaded trucks and strains were measured and correlated with these applied loads. Resulting strains from live load testing were then used to verify cracked/uncracked behavior and compute distribution factors determined from live load testing and modified rating factors. These calculation sheets are included in the appendices of this report. Appendix A.2.5 contains calculations for Canton No. 3356, A.3.5 pertains to Peru No. 5432, A.4.5 corresponds to Jackson No. 3776, A.5.5 is for Alna No. 2130, and A.6.5 is for Franklin No. 3307.

1.4.2 Bridge Characteristics

First, necessary parameters were defined for use in calculations. These included material properties for each bridge, as well as general bridge geometry (i.e. span length, girder section properties, and reinforcement layout and geometry). These were taken from each bridge's most recent available rating report or were based on minimum material properties specified by the AASHTO *Manual for Bridge Evaluation*. Dead load moments were determined from the bridge geometry and typical unit weights as specified in AASHTO *Manual for Bridge Evaluation*.

1.4.3 AASHTO Distribution Factors

Distribution factors for moment for interior and exterior girders are calculated based on in-situ measured bridge characteristics along with nominal values for dimensions that were not possible to verify in the field in accordance with AASHTO *LRFD Bridge Design Specifications*. All live load distribution factors for moment are taken assuming cross-section "e" from Table 4.6.2.2.1-1 and "Cast-in-Place Concrete Tee Beam, Monolithic concrete." For moment on interior beams this is per Table 4.6.2.2.2b-1, with all ranges of applicability met. For the exterior girder moment distribution factors are per Table 4.6.2.2.2d-1, with all ranges of applicability met.

1.4.4 AASHTO Live Loads with Impact

1.4.5 AASHTO live loads with impact (LL + IM) per lane were determined as the maximum load effect with HL-93 per (6A.2.3) and AASHTO LRFD Design 3.6.1.2 and 3.6.2. This includes the worst case of truck or tandem loading with lane loads and impact as applicable. Girder moment was calculated based on this load and the AASHTO Distribution Factors calculated as described in section 1.4.3 of this report.AASHTO Rating Factor

Flexural rating factors are computed per AASHTO Manual for Bridge Evaluation (6A.4.2.1-1) with terms as defined in that section. Values specific to the bridges in this study are as shown in Equation 1. The live load per lane computed according to section 1.4.4 of this report with impact was multiplied by the AASHTO distribution factors as described in section 1.4.3 of this report. It should be noted that only flexural rating factors were computed as bridges were not instrumented to determine effects of shear. This implies that shear rating factors could not be improved based on measured strains.

$$RF = \frac{C - \gamma_{DC}DC - \gamma_{DW}DW \pm \gamma_P P}{\gamma_{LL}(LL + IM)} \qquad \text{Equation 1}$$

$C = \varphi_c \varphi_s \varphi R_n$ (6A.4.2.1-1)
$\varphi_c = 1.0$ per Table 6A.4.2.3-1
$\varphi_s = 0.85 - 1.0$ per Table 6A.4.2.4-1
$\varphi = 0.9$ per LRFD Design 5.7.2.1
$\gamma_{DC} = 1.25$ per Table 6A.4.2.2-1
$\gamma_{DW} = 1.25$ per Table 6A.4.2.2-1 (field-measured dimensions, no coring)
$\gamma_{LL} = 1.35$ per Table 6A.4.2.2-1 (operating rating)
$R_n = A_s f_y \left(d_s - \frac{a}{2}\right)$ per LRFD Design 5.7.3.2.2-1
$P = 0$ for all bridges in this study,
no permanent loads
other than dead loads

1.4.6 Live Loads Applied during Testing

Applied moment loadings were determined based on measured truck axle weights for all load configurations. The average of axle loads for side-by-side trucks was used to allow live load distribution factors to be calculated and applied. The trucks were positioned to produce the maximum moment effect on the bridge, with the exception of the "ALT" test series, which was designed to apply less than maximum moment. Continuous data recording was started, and then trucks started moving onto the bridge in a serial manner. For each load configuration and position, trucks were moved onto the bridge one after another and the strains were allowed to plateau at the pre-determined configurations with data recording continuing during truck movement.

Applied moments were calculated assuming the bridges were simply supported. The percentage of AASHTO HL-93 loading achieved is the ratio of the moment produced by the live loads applied during testing and calculated per section 1.4.8 of this report and the moment produced by the AASHTO HL-93 loading as described in section 1.4.4 of this report.

1.4.7 Verification of Uncracked Behavior

For each bridge, the theoretical strains under test loading were computed and compared with the measured strains to verify whether concrete sections behaved as though they had remained uncracked. Theoretical strains were calculated as shown in Equation 2.

$$\varepsilon_C = \frac{DF * M_{max}}{E_c S} \qquad \text{Equation 2}$$

$DF =$ Distribution Factor per AASHTO LRFD Spec 4.6.2.2.2b-1 or 4.6.2.2.2d-1
$M_{max} =$ Maximum applied moment per girder
$E_c =$ Concrete elastic modulus per AASHTO LRFD SpecC5.4.2.4 $- 1$
$S =$ Girder section modulus, uncracked or cracked

In all cases, E_c was calculated using the compressive strength specified by the AASHTO *Manual for Bridge Evaluation*, 2.5 ksi. In addition, strains were computed assuming a compressive strength of 5 ksi, which is more conservative and may more accurately reflect the in-service concrete compressive strength of these older structures. The maximum strains recorded were compared with these calculated values. Maximum strains equal to or less than the estimated uncracked strain indicated that the bridge remained uncracked with the assumed compressive strength, while strains greater than the theoretical uncracked strain indicated possible cracking. For all bridges, the strains measured in all girders under maximum loading were smaller than those predicted with uncracked sections and 5 ksi compressive strength. For this reason, the strains computed assuming 5 ksi concrete were used in calculating rating factor improvements.

As an additional comparison to help verify uncracked behavior, the measured neutral axis depth for all girders was determined under maximum loading using the recorded strains. These depths were taken relative to the top of the deck in the case of interior girders and the top of the integral curbs for exterior girders. Where present, integral concrete wearing surfaces were considered part of the sections. These neutral axis locations inferred from measured strains were compared to the sections' theoretical neutral axis locations based on conventional strength of materials assumptions. Neutral axis locations inferred from measured strains were determined using the strains recorded at girder bottoms and at mid-height. The strains measured at the bottom of the slab were generally not used per BDI's recommendation against relying on very small measured strains. In general, measured neutral axis locations tended to be consistent with either uncracked section behavior or fell between cracked and uncracked behavior ("partially cracked"). When behavior not consistent with either uncracked or partially cracked behavior was seen, it tended to be unreasonable and was assumed to be in error.

1.4.8 Distribution Factors Determined from Live Load Testing

The appropriate section modulus – uncracked or cracked – was determined based on results from 1.4.7. The moment carried by each girder was then calculated as per Equation 3.

$$M_i = E_c S_i \varepsilon_i \qquad \text{Equation 3}$$

$M_i = Moment\ carried\ by\ girder\ i$
$E = Modulus\ of\ elasticity\ of\ girder$
$S_i = Section\ modulus\ of\ girder\ i$
$\varepsilon_i = Strain\ measured\ in\ girder\ i$

The distribution factor for each girder was then calculated by Equation 4.

$$DF_i = \frac{M_i}{\sum_{i=1}^{n} M_i} \qquad \text{Equation 4}$$

$DF_i = Distribution\ factor\ for\ girder\ i$
$M_i = Moment\ carried\ by\ girder\ i$
$n = Total\ number\ of\ girders$

1.4.9 Modified Rating Factor

In accordance with the AASHTO *Manual for Bridge Evaluation*, the ratio of computed strain ε_C (based on the section behavior determined in 1.4.7) to measured strain ε_T was then used to compute a rating factor modifier as detailed below in Equation 5 to Equation 7. This analysis is based on the interior girder and exterior girder that experienced the largest measured strain.

$$RF_T = RF_c K \qquad \text{Equation 5}$$

In Equation 5, RF_T is the modified rating factor taking into account test results, RF_c is the rating factor based on standard calculations, and K is an adjustment factor specified by the AASHTO *Manual for Bridge Evaluation* that incorporates the test results. K is computed per Equation 6 below.

$$K = 1 + K_a K_b \qquad \text{Equation 6}$$

K_a accounts for the difference between measured response based on load testing and expected response as shown below in Equation 7. K_b accounts for the magnitude of the applied test load and confidence in extrapolating results; and is defined in Table 8.8.2.3.1-1 in the AASHTO *Manual for Bridge Evaluation*. For all structures K_b was taken as 0.5 per the AASHTO *Manual for Bridge Evaluation*, which reflects both the magnitude of the applied load and assumes results cannot be extrapolated to higher loads.

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1 \qquad \text{Equation 7}$$

2 Live Load Test Results

2.1 Canton No. 3356

The bridge in Canton, No. 3356 over Whitney Brook, is shown in Figure 8. Testing was conducted on July 11, 2017 with a maximum applied moment producing 75% of HL-93 loading with impact. The rating factors based on the AASHTO *LRFD Design Manual* and *Manual for Bridge Evaluation* are 0.98 and 2.48 for the interior and original exterior girders respectively. This bridge was unique in that its roadway was widened by utilizing an additional exterior girder. This girder was originally designed to support a sidewalk, as is shown from the original design drawings in Figure 8 but is now used to support traffic loading, as is shown in Figure 8. The rating factor computed per the AASHTO *MBE* for this extended girder was 0.28, making this bridge the lowest rating bridge of those tested. As expected, strains measured from this extended exterior girder were the largest measured from all exterior girders, while strains measured from the remaining girders were typical of other bridges. Table 2 shows the maximum measured strains for this bridge.

Assuming the nominal concrete compressive strength of 2.5 ksi specified, the strains recorded indicate the sections remain uncracked. Using a more conservative value of 5 ksi, the high strains recorded in the extended girder seem to indicate partially cracked behavior. This is supported by the observed neutral axis depths, which, with the exception of girders 3 and 5 which are likely in error, display uncracked behavior as can be seen in Table 3. Based only on the computed cracking strain, cracked behavior would be experienced in the exterior girder for compressive strengths greater than about 3.5 ksi. However, uncracked behavior was still assumed for the remaining girders as they experienced relatively low strains and their measured neutral axes indicated either uncracked or partially cracked behavior. These conditions allowed interior girder rating factors to be increased to 1.39 and the original exterior girder rating factor to be increased to 5.69. Conservatively using its uncracked section modulus to predict strain response, the extended exterior girder’s rating factor could be increased to 0.30.

The live load distribution factors determined from the measured strains and those calculated per AASHTO are shown in in Table 3, and generally indicate that the AASHTO distribution factors are conservative with the exception of girder 1 – the extended exterior girder. It should be noted as well that due to technical issues, tests could not be conducted at load position 2 (trucks centered about the roadway centerline) for this bridge, which may have led to a more even distribution of live load. As shown in Table 2, strain measured at the ends of the girders indicate very little, if any, end restraint.

Figure 8: Canton No. 3356 general condition

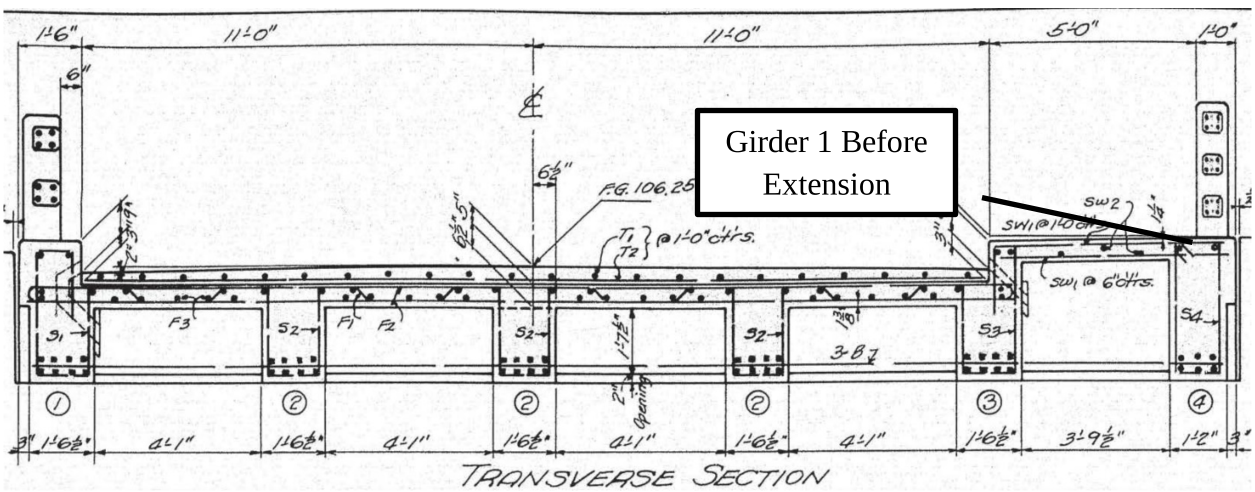

Figure 9: Canton No. 3356 original cross section

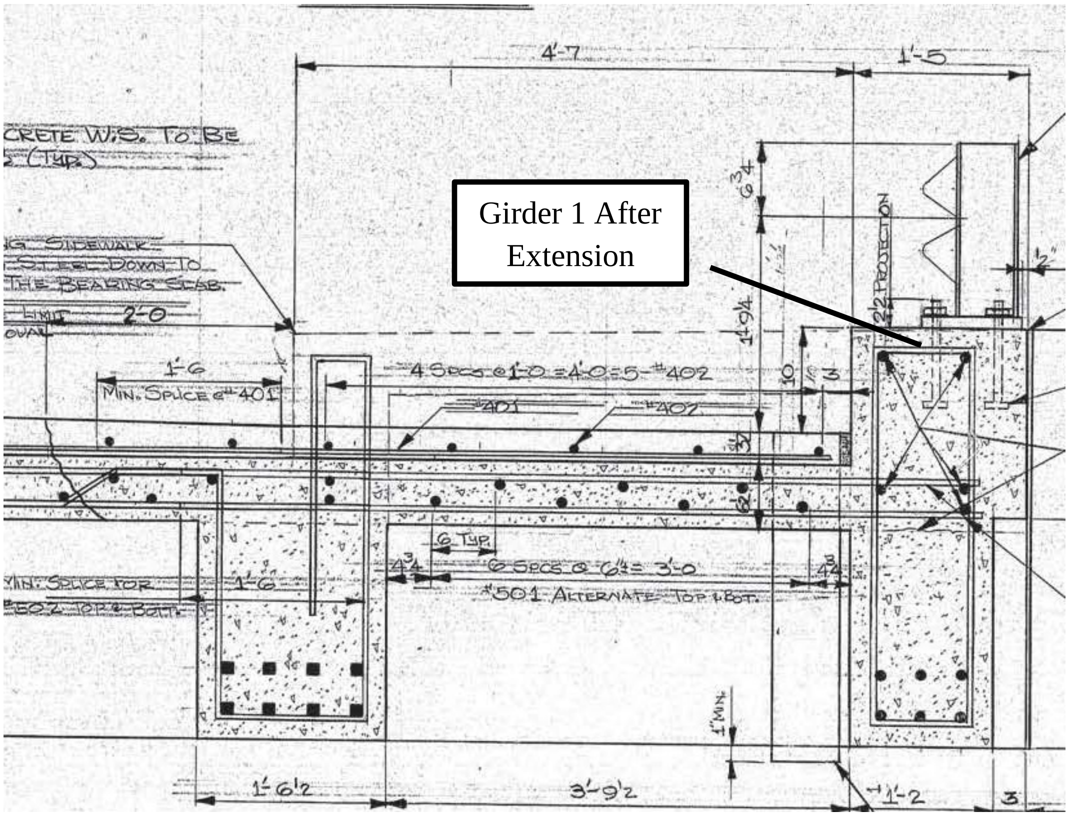

Figure 10: Canton No. 3356 redesigned cross section

Table 2: Canton No. 3356 strains recorded from tests SBS_1_2 and MAX_1_1

Canton		**SBS_1_2**			**MAX_1_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$
1	**Top**	3.47	-2.34	-	5.71	-3.20	-
	Center	9.87	-	-	13.0	-	-
	Bottom	50.6	-0.38	-3.84	78.5	5.70	-0.63
2	**Top**	6.98	-	-	4.93	-	-
	Center	21.1	-	-	29.3	-	-
	Bottom	49.8	-	-	66.9	-	-
3	**Top**	2.01	-0.57	-	2.67	-1.22	-
	Center	44.5	-	-	45.4	-	-
	Bottom	59.1	0.89	2.64	72.6	8.49	3.16
4	**Top**	0.13	-	-	-0.70	-	-
	Center	19.9	-	-	27.4	-	-
	Bottom	49.1	-	-	63.2	-	-
5	**Top**	1.26	-	-	1.79	-	-
	Center	15.0	-	-	20.1	-	-
	Bottom	22.0	-	-	31.2	-	-
6	**Top**	0.90	-	-	2.61	-	-
	Center	3.47	-	-	6.76	-	-
	Bottom	9.93	-	-	18.0	-	-

Table 3: Canton No. 3356 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	17.9	11.5	26.8
2	12.2	7.60	12.7
3	12.2	7.60	2.93
4	12.2	7.60	13.0
5	12.2	7.60	1.59
6	19.5	13.9	22.04

Table 4: Canton No. 3356 distribution factors

Canton		**SBS_1_2**		**MAX_1_1**	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.438	0.436	-0.5%	0.496	13%
2	0.611	0.398	-35%	0.393	-36%
3	0.611	0.472	-23%	0.424	-31%
4	0.611	0.392	-36%	0.360	-41%
5	0.611	0.175	-71%	0.171	-72%
6	0.438	0.127	-71%	0.155	-65%

2.2 Peru No. 5432

The bridge in Peru, No. 5432 over Spears Stream, is shown in Figure 11. Testing was conducted on July 13, 2017 with maximum applied moment producing 87% of HL-93 flexural service load with impact. This is the highest percentage of HL-93 load applied among all the bridges. This led to some of the highest strains recorded of all the bridges tested, as shown in Table 4. Despite these high strains, the girders behaved as uncracked when both the nominal 2.5 ksi compressive strength, and when the more conservative 5 ksi was used to predict strains. Based on a computed modulus of rupture, uncracked behavior would be experienced for compressive strengths up to around 5 ksi. Further evidence for uncracked behavior is provided by the measured neutral axis depths which tended to be consistent with uncracked or partially cracked behavior as seen in Table 5. The originally computed rating factors based on the AASHTO *LRFD Design Manual* and *Manual for Bridge Evaluation* are 0.75 and 2.04 for the interior and exterior girders respectively. Under the conditions experienced during testing, the interior and exterior rating factors were able to be increased to 1.10 and 2.55, bringing both flexural rating factors to acceptable operating levels.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 5. The AASHTO distribution factors are quite conservative. However, as can be seen in Table 4, the strain reading at the bottom sensor of girder 2 is likely in error as it reads a value of strain very close to that measured at the girder's mid-depth.

The distribution factors resulting from replacing the recorded strain in girder 2 with that recorded in girder 4 are presented in Table 5. The distribution factor for girder 2 calculated in this way is more consistent with the distribution factors calculated for the other girders and is still much smaller than the distribution factor predicted by AASHTO. It is evident from the girder end strains reported in Table 4 that any unintended end restraint was negligible.

Figure 11: Peru No. 5432 general condition

Table 5: Peru No. 5432 strains from tests SBS_2_1 and MAX_2_1

Peru		**SBS_2_1**			**MAX_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$
1	**Top**	-3.44	-3.35	-	-5.34	-2.43	-
	Center	2.39	-	-	3.41	-	-
	Bottom	30.00	7.39	-0.54	46.47	12.12	-
2	**Top**	-3.29	-	-	-6.05	-	-
	Center	18.59	-	-	27.46	-	-
	Bottom	18.81	-	-	26.56	-	-
3	**Top**	-5.35	-2.60	-	-9.71	-2.19	-
	Center	19.57	-	-	27.26	-	-
	Bottom	56.34	3.66	-2.29	77.96	-1.33	1.25
4	**Top**	-0.48	-	-	-5.91	-	-
	Center	22.10	-	-	33.22	-	-
	Bottom	60.52	-	-	87.34	-	-
5	**Top**	-3.97	0.83	-	-5.20	-4.23	-
	Center	15.46	-	-	28.38	-	-
	Bottom	34.77	11.71	-0.43	55.16	14.88	0.45

Table 6: Peru No. 5432 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	22.9	15.5	31.6
2	15.4	9.00	12.5
3	15.4	9.00	16.8
4	15.4	9.00	15.5
5	22.9	15.5	16.9

Table 7: Peru No. 5432 distribution factors from recorded strains

Peru		**Two Trucks**		**Four Trucks**	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.473	0.360	-24%	0.382	-19%
2	0.68	0.186	-73%	0.161	-76%
3	0.68	0.500	-26%	0.474	-30%
4	0.68	0.537	-21%	0.530	-22%
5	0.473	0.417	-12%	0.453	-4%

Table 8: Peru No. 5432 distribution factors assuming identical strains in girder 2 and 4

Peru		**Two Trucks**		**Four Trucks**	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.473	0.306	-35%	0.322	-32%
2	0.68	0.457	-33%	0.448	-34%
3	0.68	0.425	-38%	0.400	-41%
4	0.68	0.457	-33%	0.448	-34%
5	0.473	0.355	-25%	0.382	-19%

2.3 Jackson No. 3776

The bridge in Jackson, No. 3776 over the North Branch of Marsh Stream, is shown in Figure 12. Testing was conducted on July 18, 2017 with maximum applied moment producing 70% of HL-93 moment with impact. This was the smallest percentage of HL-93 moment with impact applied to any of the bridges tested, and just barely qualifies the load test to be useful in improving live load rating. As a result, the strains recorded were among the smallest on any bridges tested, as shown in Table 7. Rating factors determined per AASHTO equaled 0.69 and 2.08 for interior and exterior girders respectively. When compared with predicted strains using both the nominal and conservative compressive strengths of 2.5 ksi and 5 ksi respectively, the recorded strains indicated uncracked behavior. This uncracked behavior would theoretically continue with compressive strengths up to around 5.5 ksi. The measured neutral axis depths also provide evidence for uncracked behavior. For all girders, the measured neutral axis depths indicate uncracked or nearly uncracked behavior as seen in Table 8. These conditions allowed for the interior and exterior rating factors to be increased to 1.20 and 2.40 respectively.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 8. The results of experimental determination would suggest that AASHTO's distribution factors are somewhat conservative. However, Table 7 shows that the measured strain at the bottom of the midspan section of girder 2 is large in comparison to the other measured strains for the tests reported. This may be due to some cracking in this particular girder that was not visible. Regardless, the AASHTO distribution factors remain conservative, with the exception of girder 5. It is evident from the girder end strains reported in Table 7 that any unintended end restraint was negligible.

Figure 12: Jackson No. 3776 general condition

Table 9: Jackson No. 3776 strains from tests SBS_2_1 and MAX_2_1

Jackson		**SBS_2_1**			**MAX_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	***με***	***με***	***με***	***με***	***με***	***με***
1	**Top**	-1.68	0.72	-	-2.40	0.93	-
	Center	8.37	-	-	12.86	-	-
	Bottom	21.95	1.08	1.69	30.27	2.22	6.49
2	**Top**	-1.36	-	-	-2.08	-	-
	Center	15.58	-	-	20.27	-	-
	Bottom	54.6	-	-	67.69	-	-
3	**Top**	-2.43	-0.81	-	-2.22	-0.57	-
	Center	17.29	-	-	24.07	-	-
	Bottom	44.57	4.21	4.71	57.69	11.56	11.78
4	**Top**	-	-	-	-	-	-
	Center	12.8	-	-	19.01	-	-
	Bottom	38.95	-	-	53.94	-	-
5	**Top**	0.322	-1.49	-	-0.740	-3.17	-
	Center	3.09	-	-	5.75	-	-
	Bottom	27.41	1.34	0.42	44.79	4.93	2.55

Table 10: Jackson No. 3776 neutral axis depths

Girder	Uncracked NA Depth (in)	Cracked NA Depth (in)	Measured NA Depth (in)
1	20.9	14.1	23.4
2	13.4	7.70	16.7
3	13.4	7.70	13.1
4	13.4	7.70	15.4
5	20.9	14.1	28.2

Table 11: Jackson No. 3776 distribution factors

Jackson		SBS_2_1		MAX_2_1	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.396	0.299	-24%	0.300	-24%
2	0.635	0.525	-17%	0.474	-25%
3	0.635	0.428	-33%	0.404	-36%
4	0.635	0.374	-41%	0.378	-40%
5	0.396	0.373	-6%	0.444	12%

2.4 Alna No. 2130

The bridge in Alna, No. 2130 over Carlton (Trout) Brook, is shown in Figure 13. Testing occurred on July 20, 2017 with maximum applied moment producing 75% of HL-93 live load with impact. Rating factors determined per AASHTO equaled 0.92 and 1.01 for interior and exterior girders respectively. The recorded strains presented in Table 9 indicate uncracked behavior when compared with predicted strains using both the nominal and conservative compressive strengths of 2.5 ksi and 5 ksi respectively. Uncracked behavior would theoretically continue up to a compressive strength around 8 ksi based on a nominal computed modulus of rupture. Measured neutral axis height tends to confirm uncracked behavior for girders 2, 3, and 4. However, the measured neutral axis height for girder 1 is unreasonable (outside of the section) and so was determined to be in error. This can be seen in Table 9 These conditions allowed for interior and exterior rating factors to be increased to 1.28 and 1.98 respectively.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 10. For exterior girders, AASHTO is very conservative. However, for interior girders, the AASHTO distribution factors appear to be unconservative. Alna No 2130 is the only structure for which this is the case. One possible cause is that Alna has only four girders compared to five for all other structures tested. It is surprising that so little load is drawn to the exterior girders given that the bridge’s curb and railing were recently replaced and are quite massive. It would seem that such a large mass of concrete would add significantly to the stiffness of the girders. However, the girder, curb, and railing may not act fully compositely, decreasing the beneficial stiffening effect. From the relatively large negative girder end strains reported in Table 9, it would seem that a small amount of unintended end restraint was experienced during the SBS_2_1 test. However, this was not seen in the MAX_2_1 test and so it should not be regarded as a reliable beneficial effect.

Figure 13: Alna No. 2130 general condition

Table 12: Alna No. 2130 Strain and Neutral Axis Data

Alna		**SBS_2_1**			**MAX_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	***με***	***με***	***με***	***με***	***με***	***με***
1	**Top**	3.60	-0.07	-	2.46	2.74	-
	Center	12.05	-	-	10.81	-	-
	Bottom	14.43	0.69	-2.45	12.12	4.56	0.77
2	**Top**	-2.97	-0.81	-	-5.29	-1.50	-
	Center	17.25	-	-	17.55	-	-
	Bottom	47.97	-3.80	-5.71	47.06	3.28	5.74
3	**Top**	-4.84	-1.82	-	-7.55	-3.84	-
	Center	18.31	-	-	17.62	-	-
	Bottom	59.28	-4.09	-10.11	56.37	-0.42	0.24
4	**Top**	-	-0.17	-	-	-0.19	-
	Center	8.498	-	-	9.07	-	-
	Bottom	22.18	-0.18	-2.15	19.71	-0.07	1.04

Table 13: Alna No. 2130 neutral axis depths

Girder	Uncracked NA Depth (in)	Cracked NA Depth (in)	Measured NA Depth (in)
1	21.2	15.1	-17.1
2	12.2	7.70	12.6
3	12.2	7.70	14.8
4	21.2	15.1	25.4

Table 14: Alna No. 2130 distribution factors

Alna		SBS_2_1		MAX_2_1	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.485	0.201	-59%	0.185	-62%
2	0.707	0.663	-6%	0.667	-6%
3	0.707	0.827	17%	0.829	17%
4	0.485	0.309	-36%	0.319	-34%

2.5 Franklin No. 3307

The bridge in Franklin, No. 3307 over Card's Mill Stream, is shown in Figure 14. Testing occurred on July 25, 2017 with maximum applied moment producing 84% of HL-93 load with impact. Rating factors determined per AASHTO equaled 0.92 and 2.78 for interior and exterior girders respectively. Measured strains given in Table 11 indicate uncracked behavior when compared with strains predicted using the nominal 2.5 ksi compressive strength or the more conservative 5 ksi. Based on the models by AASHTO, the section would remain uncracked for compressive strengths up to 11 ksi. This behavior is supported by the measured neutral axis depths, which indicate uncracked or partially cracked behavior for all girders as seen in Table 12. Based on these conditions, the interior and exterior flexural rating factors could be increased to 1.61 and 5.03 respectively.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 12. It is clear that for all girders AASHTO is conservative and this conservatism is relatively consistent across both presented load cases. Notably, the measured distribution factors seem to increase from the exterior girders to the center girder and are reasonably symmetric about the center girder. This is somewhat unexpected as the deck had a significant side-slope to conform to road super-elevation and because this symmetric behavior was not seen in other bridges. It is evident from the girder end strains reported in Table 11 that any unintended end restraint was negligible.

Figure 14: Franklin No. 3307 general condition

Table 15: Franklin No. 3307 Strain and Neutral Axis

Franklin		**SBS_2_1**			**MAX_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	***με***	***με***	***με***	***με***	***με***	***με***
1	**Top**	0.46	-0.83	-	1.33	-3.67	-
	Center	11.56	-	-	16.74	-	-
	Bottom	30.9	-0.17	-0.01	44.31	-5.04	1.54
2	**Top**	-8.43	-	-	-8.43	-	-
	Center	16.82	-	-	25.45	-	-
	Bottom	41.3	-	-	58.21	-	-
3	**Top**	-7.16	1.57	-	-8.97	-0.07	-
	Center	19.63	-	-	27.27	-	-
	Bottom	55.65	8.42	-0.12	76.16	7.09	2.82
4	**Top**	-	-	-	-	-	-
	Center	14.44	-	-	21.61	-	-
	Bottom	45.29	-	-	62.48	-	-
5	**Top**	-4.95	0.27	-	-	-0.64	-
	Center	6.44	-	-	11.06	-	-
	Bottom	25.24	1.28	-2.79	38.65	-1.34	-4.96

Table 16: Franklin No. 3307 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	22.0	16.1	23.3
2	14.2	9.20	12.7
3	14.2	9.20	15.4
4	14.2	9.20	15.8
5	22.0	16.1	26.0

Table 17: Franklin No. 3307 distribution factors

Franklin		**SBS_2_1**		**MAX_2_1**	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.432	0.319	-27%	0.324	-25%
2	0.6	0.413	-31%	0.412	-31%
3	0.6	0.556	-7%	0.539	-10%
4	0.6	0.453	-25%	0.442	-26%
5	0.432	0.260	-40%	0.282	-35%

3 Summary of Live Load Test Data Conclusions

Analyses of the tested bridges are described in detail in Section 2. In general, calculations were based on mechanics of materials principles and AASHTO code requirements including the *Manual for Bridge Evaluation*.

Overall, a high percentage of HL-93 loading with impact was applied to the structures. In all cases, the maximum applied moment was at least 70% of HL-93 service moment with impact, which is required to justify rating factor increases per the AASHTO *Manual for Bridge Evaluation*. Numerically, this translates to a test understanding factor, k_b equal to 0.5 for all bridges, which effectively reduces the measured benefit by 50% Because measured strains were invariably smaller than those predicted, all test benefit factors, k_a were greater than zero, and all rating factors could be increased based on measure strains.

Live load distribution factors inferred from the test data showed reasonable agreement with AASHTO-recommended values, although the AASHTO values are generally conservative. The maximum differences between values inferred from the tests and values computed per AASHTO were observed for the bridge in Canton. However, these differences may have been less pronounced for symmetric load cases.

Assuming the nominal specified concrete compressive strength of 2.5 ksi, all bridges exhibited uncracked behavior under maximum applied moment. With the exception of the bridge in Canton, the assumption of uncracked behavior could be extended to strains predicted for a compressive strength of 5 ksi or greater, continuing to 11 ksi for the bridge in Franklin. This observation is significant in that it is possible that the actual compressive strengths of the concrete in these bridges is significantly greater than the nominal value specified by AASHTO given their age and condition.

The test results and analyses presented here justify significant increases in the rating factors for four of the five bridges according to the AASHTO *Manual for Bridge Evaluation*. The average increase in HL-93 flexural operating rating factors for the critical interior girders of all bridges was 55.6%, with minimum and maximum increases of 41.8% and 74.9% respectively. All rating factor increases have been calculated based on the assumption that the observed results cannot be confidently extrapolated to loads of 30% beyond that produced by HL-93 load with impact, largely due to uncertainty of uncracked section behavior at higher loads.

With the exception of the bridge in Canton, each bridge's controlling operating flexural rating factor could be increased to 1.0 or greater for HL-93 loading with impact, indicating that they are sufficient for such loading. For the case of the bridge in Canton, the controlling rating factor of 0.30 is for the exterior girder originally supporting a sidewalk that was re-purposed to support traffic loads. The remaining girders of the Canton bridge had increased flexural rating factors above 1.0.

4 References

1. AASHTO (2010). "The Manual for Bridge Evaluation (2nd Ed)." American Association of State Highway and Transportation Officials, Washington DC. (with 2015 Interirm Revisions).

2. AASHTO (2012). "AASHTO LRFD Bridge Design Specifications (Customary U.S. Units)." American Association of State Highway and Transportation Officials, Washington DC. doi:978-1-56051-523-4.

A.1 Experimental Configuration and Data Collected

For each of the five bridges tested, a collection of data files is provided which contains input data, experimental configuration data, and data collected during tests. The files pertaining to each bridge are tabulated in the following appendices.

A.1.1 Input Data

Input data for each bridge include bridge geometries, material properties, and sensor layouts. Section properties for interior and exterior girders are also listed in separate Comma Separated Variable (.csv) files, labeled and in units of inches to the appropriate power.

A .csv is also provided which gives a list of the serial numbers of the sensors in the order as well as a MATLAB variable file (.mat) giving the layout of those sensors on each bridge. The sensor list .csv file provides sensors in the order that they are used and tabulated by STS-WiFi, and consequently in resulting test data. The sensor layout gives relative positions of sensors as they appeared for each bridge. Each girder is represented by three rows of data representing its top, middle and bottom respectively. Each collection of rows is placed in its relative position as it appears on the bridge. From left to right, columns represent the end receiving two sensors, mid-span, and the end receiving one sensor respectively. In this way, the relative position of each sensor can be determined. For example, a sensor in the second column of the second row would represent a sensor placed at mid-height of the first girder at midspan.

A.1.2 Experimental Configuration

Experimental configuration data includes data on the loading trucks. Each test includes a .mat file containing information on the trucks used to test it. The truck .mat file contains structured arrays for each truck, containing its plate number, truck number in relation to each test, individual wheel weights (in pounds), lengths (center to center of wheels; side, front and back in inches), wheel bearing surface widths (front to back in inches), and wheel bearing lengths (front to back in inches).

A.1.3 Collected Data

For each test configuration, a .mat file is provided which contains strain data recorded during the test. This data has been rectified by a linear correction function to correct for the sensors' tendency to drift its zero-point during a test.

A.2 Canton No. 3356

A.2.1 Input Data, Experimental Configuration, and Experimental Data Collected

Table 18: Canton No. 3356 input data, experimental configuration, and experimental data collected

File Contents	File Name	File Type
Exterior Section Data	Br3356_Geom _Ext.csv	CSV Format
Interior Section Data	Br3356_Geom _Int.csv	CSV Format
Sensors	Br3356 _Sensors.csv	CSV Format
Sensor Layout	Br3356 _SensorLayout.mat	MATLAB Data File
Truck Weight and Dimensions	Br3356_Trucks.mat	MATLAB Data File
Sensor Data	Br_3356_ALT_1_1_Strain.mat	MATLAB Data File
	Br_3356_BTB_1_1_Strain.mat	MATLAB Data File
	Br_3356_BTB_1_3_Strain.mat	MATLAB Data File
	Br_3356_BTB_2_1_Strain.mat	MATLAB Data File
	Br_3356_MAX_1_1_Strain.mat	MATLAB Data File
	Br_3356_MAX_1_2_Strain.mat	MATLAB Data File
	Br_3356_SBS_1_1_Strain.mat	MATLAB Data File

A.2.2 Instrumentation

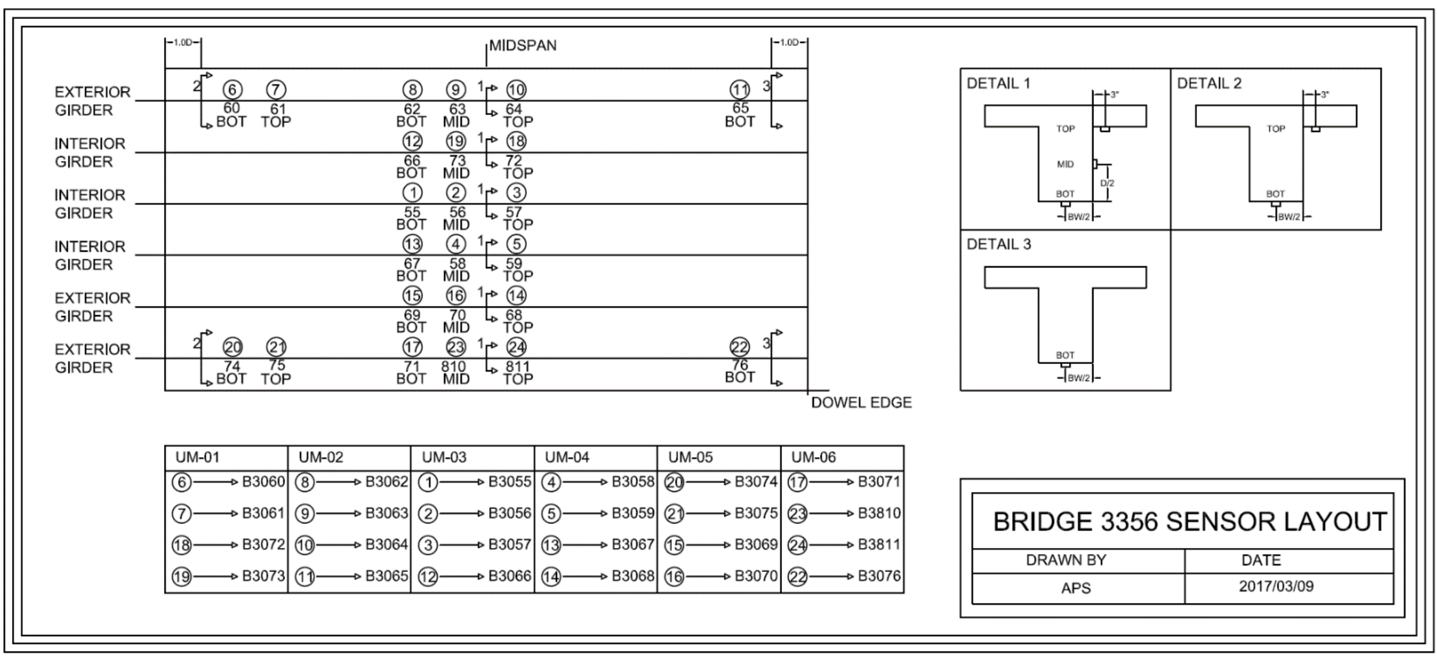

Figure 15: Canton No. 3356 sensor layout

A.2.3 Loading

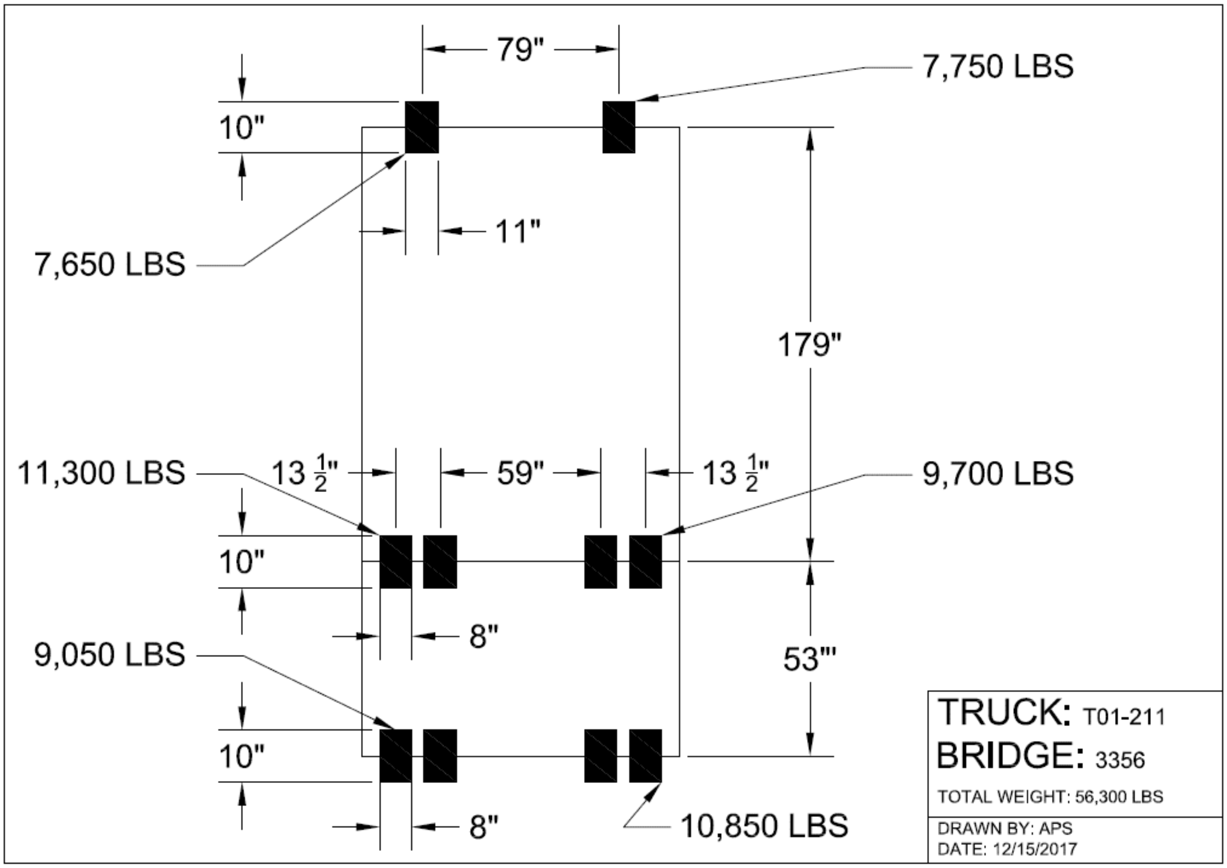

Figure 16: Canton No. 3356 Truck T01-211 loading

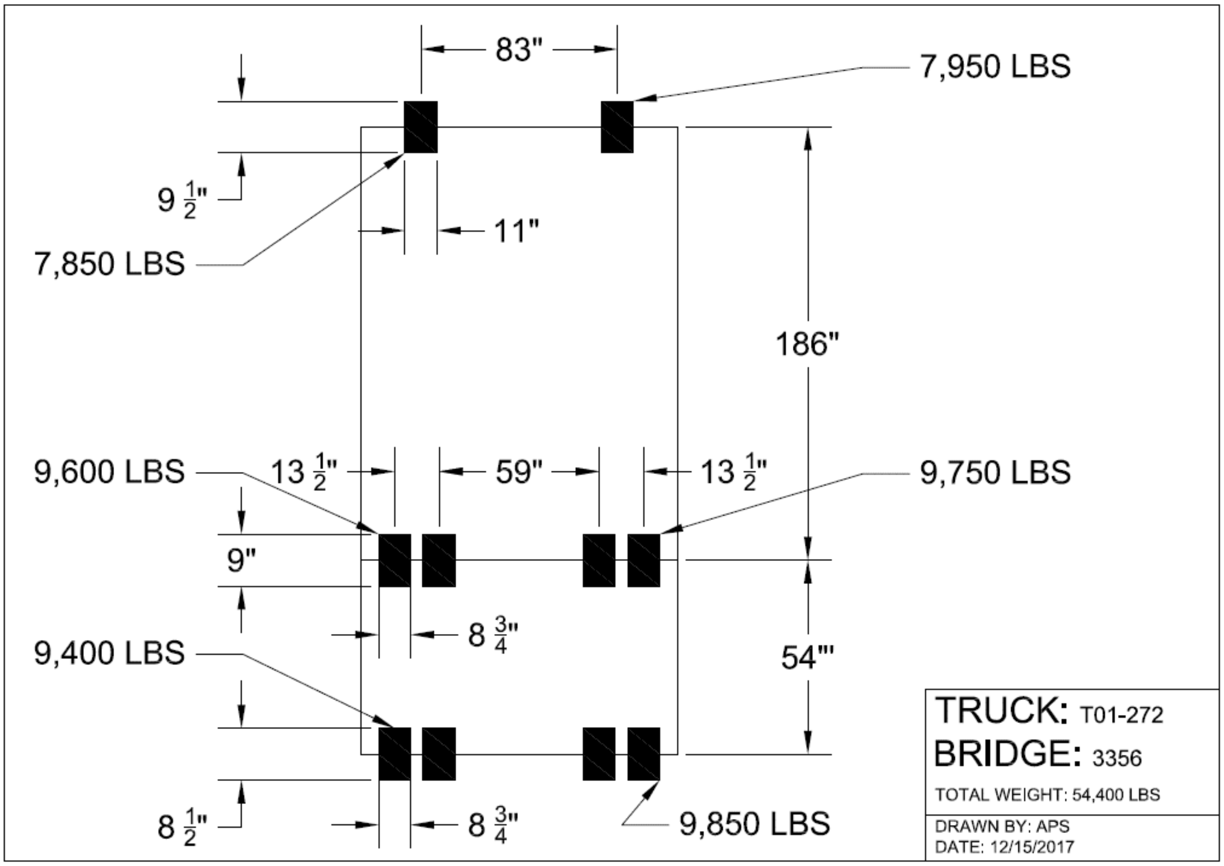

Figure 17: Canton No. 3356 Truck T01-272 loading

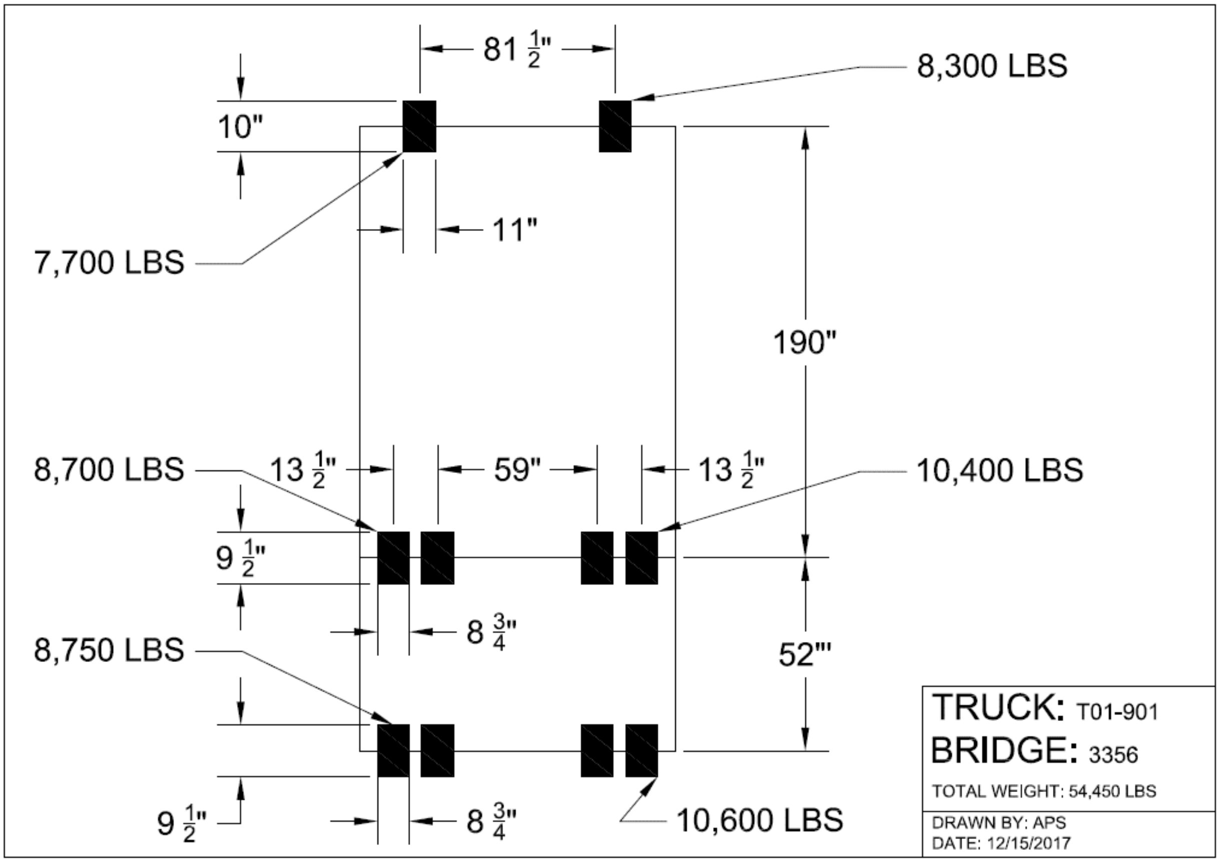

Figure 18: Canton No. 3356 Truck T01-901 loading

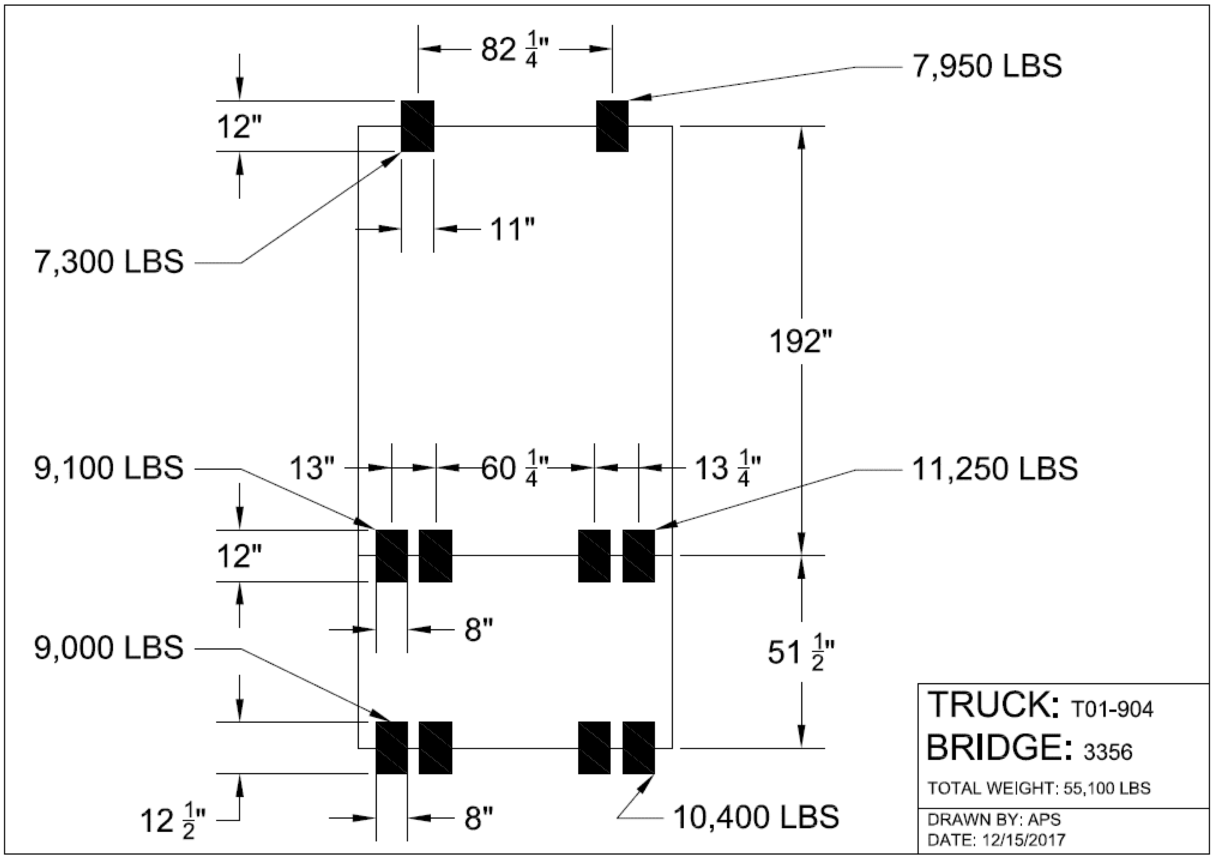

Figure 19: Canton No. 3356 Truck T01-904 loading

A.2.4 Representative Data Plots

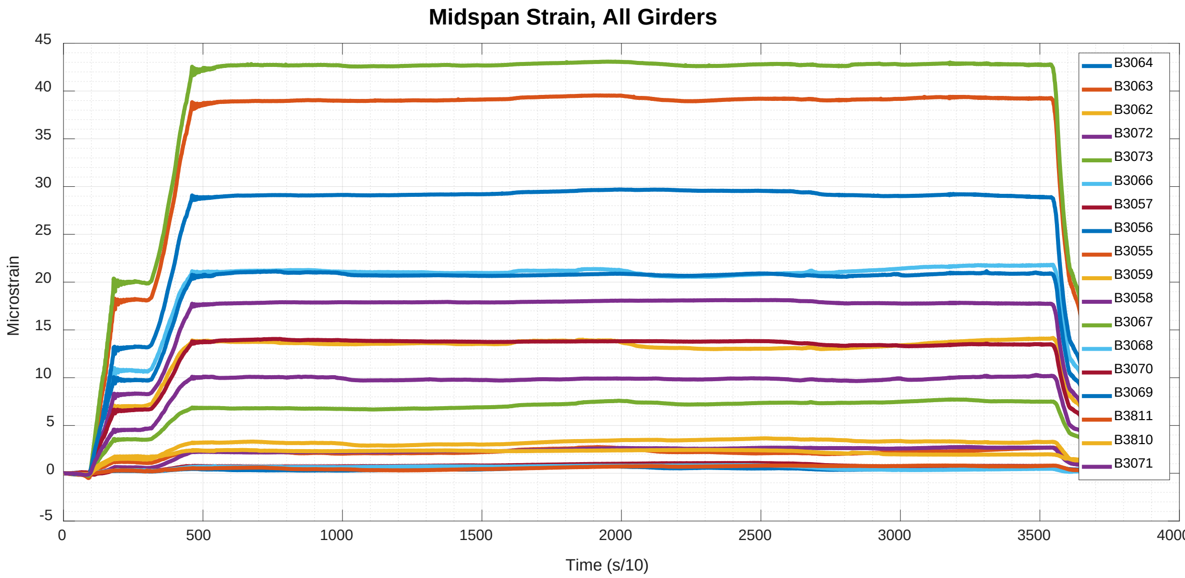

Figure 20: Canton No. 3356- BTB_2_1 strains - Midspan

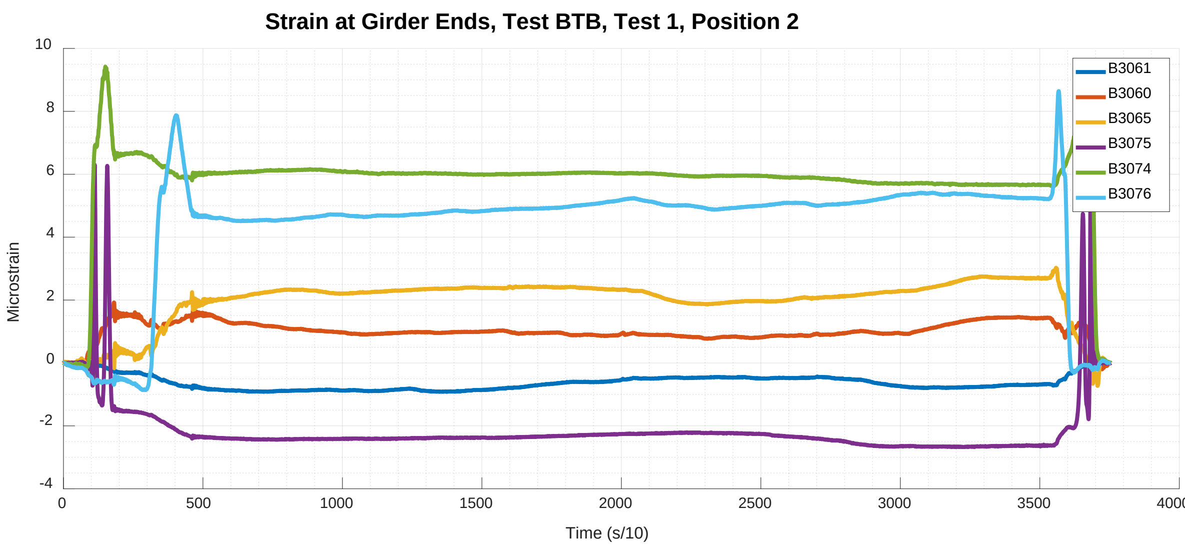

Figure 21: Canton No. 3356- BTB_2_1 strains - Ends

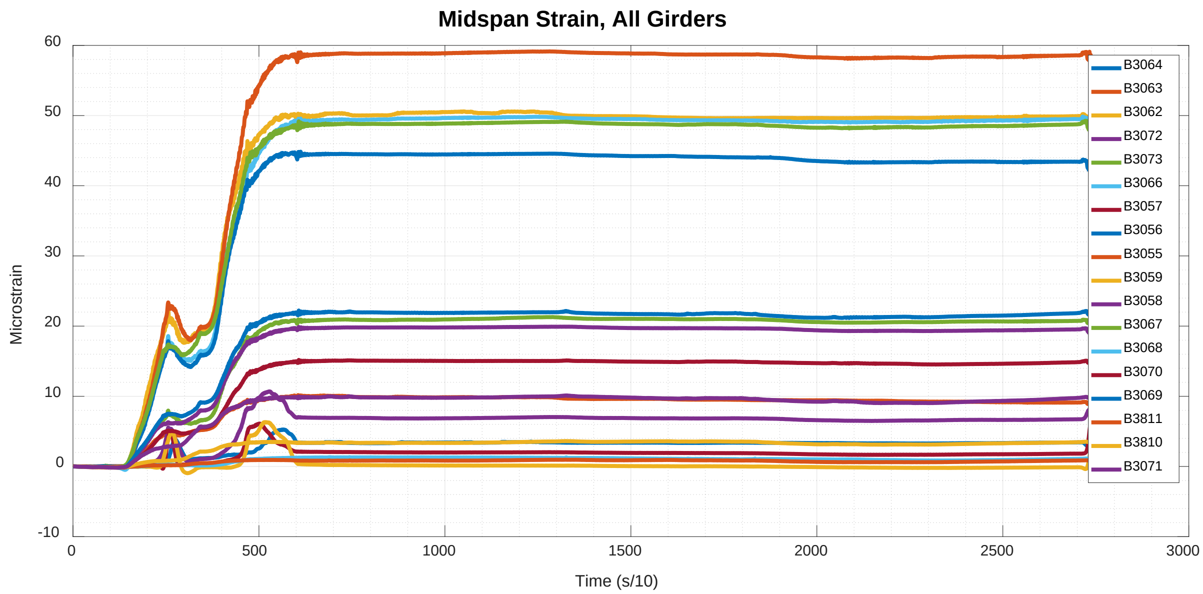

Figure 22: Canton No. 3356- SBS_1_2 strains - Midspan

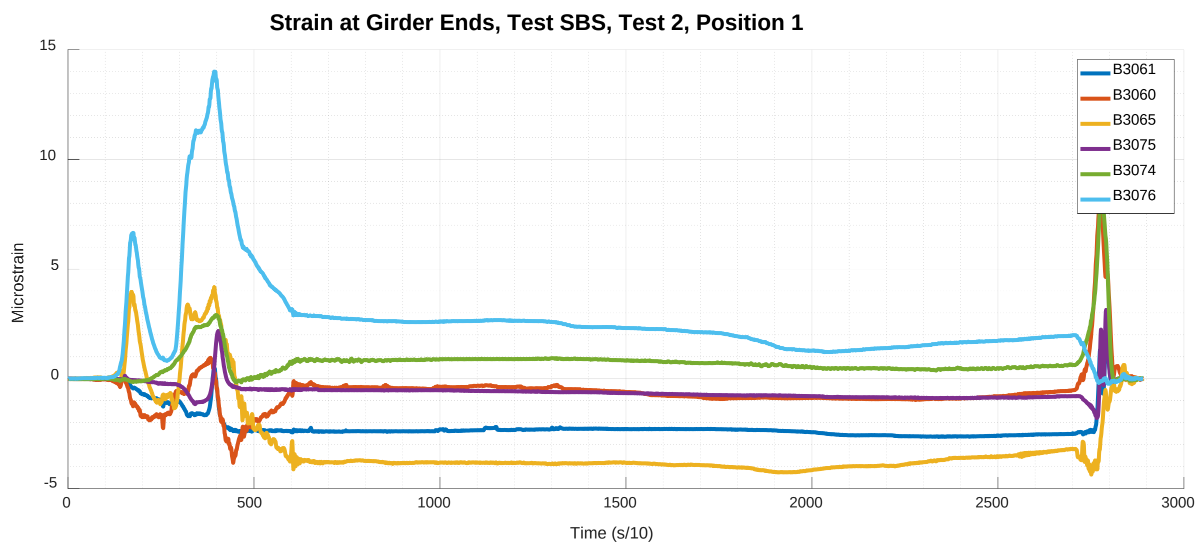

Figure 23: Canton No. 3356- 4 SBS_1_2 strains - Ends

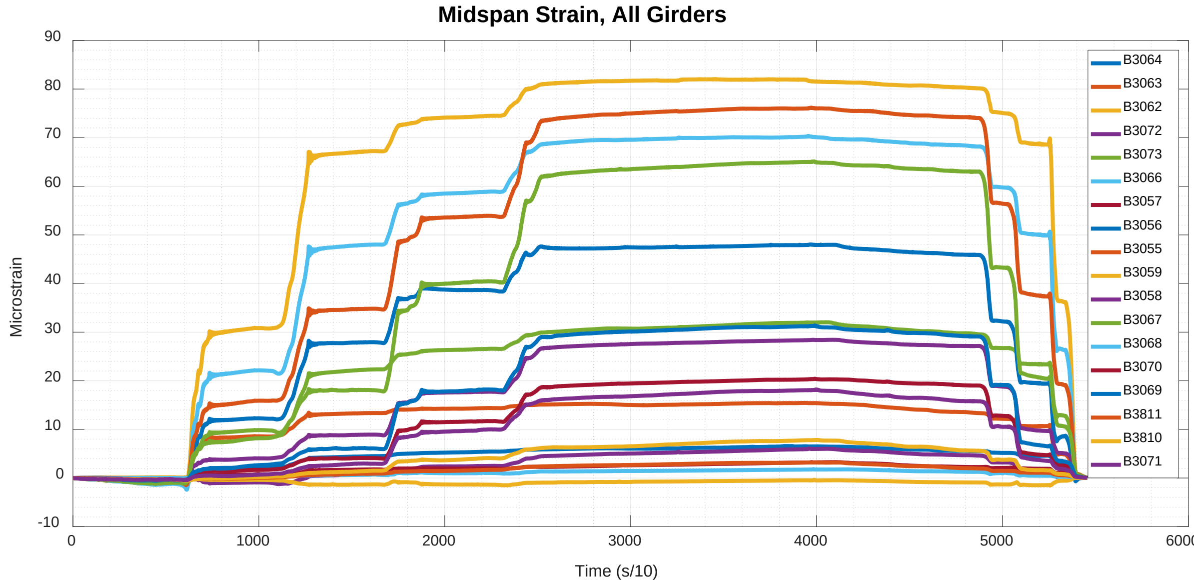

Figure 24: Canton No. 3356- MAX_1_2 strains - Midspan

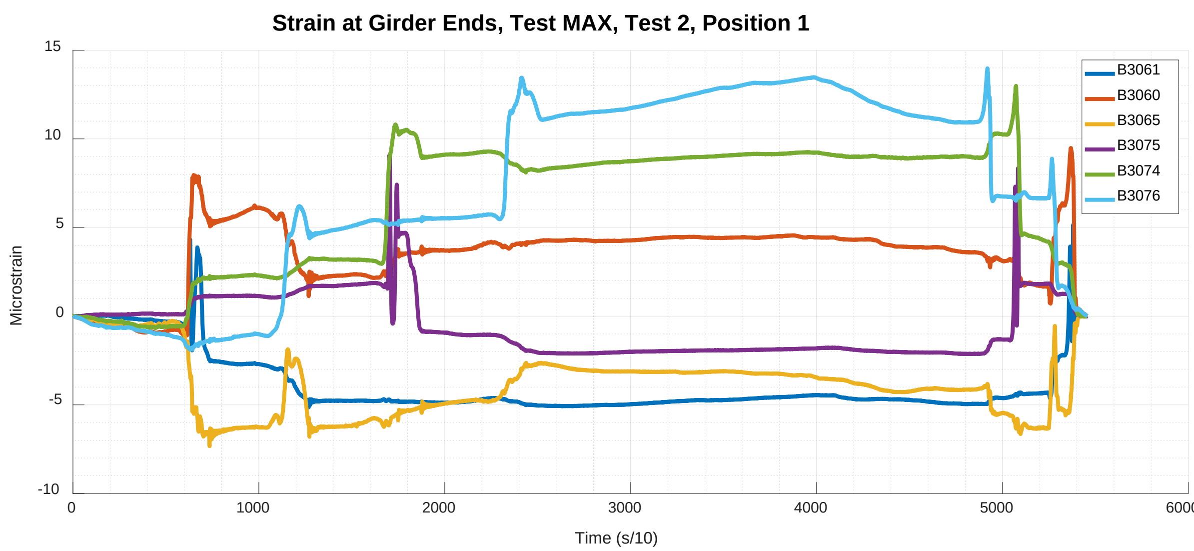

Figure 25: Canton No. 3356- MAX_1_2 strains – Ends

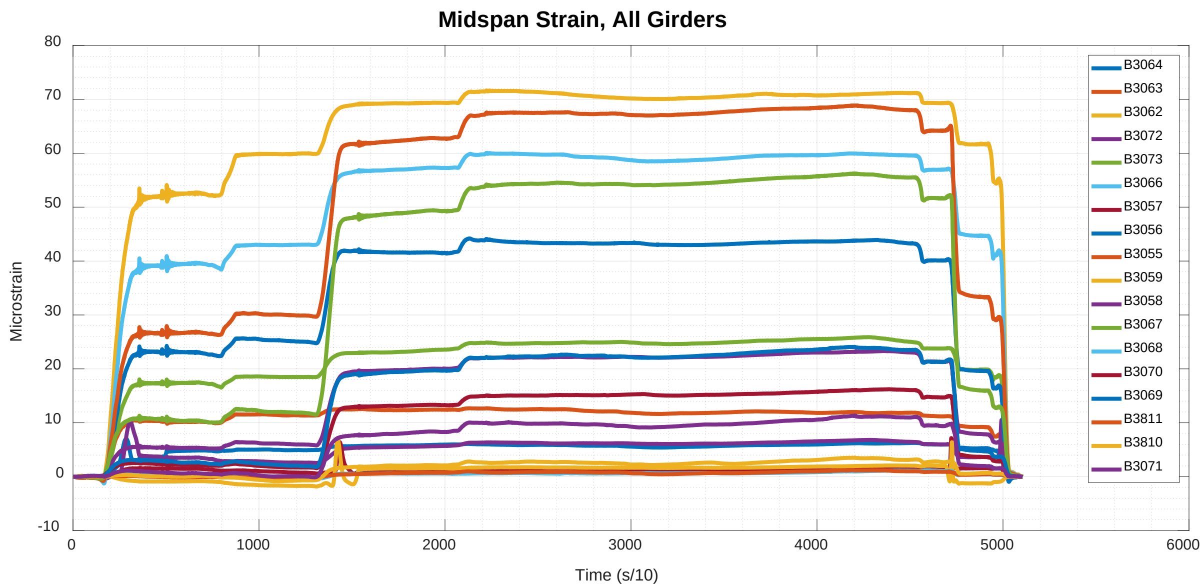

Figure 26: Canton No. 3356 – ALT_1_1 strains – Midspan

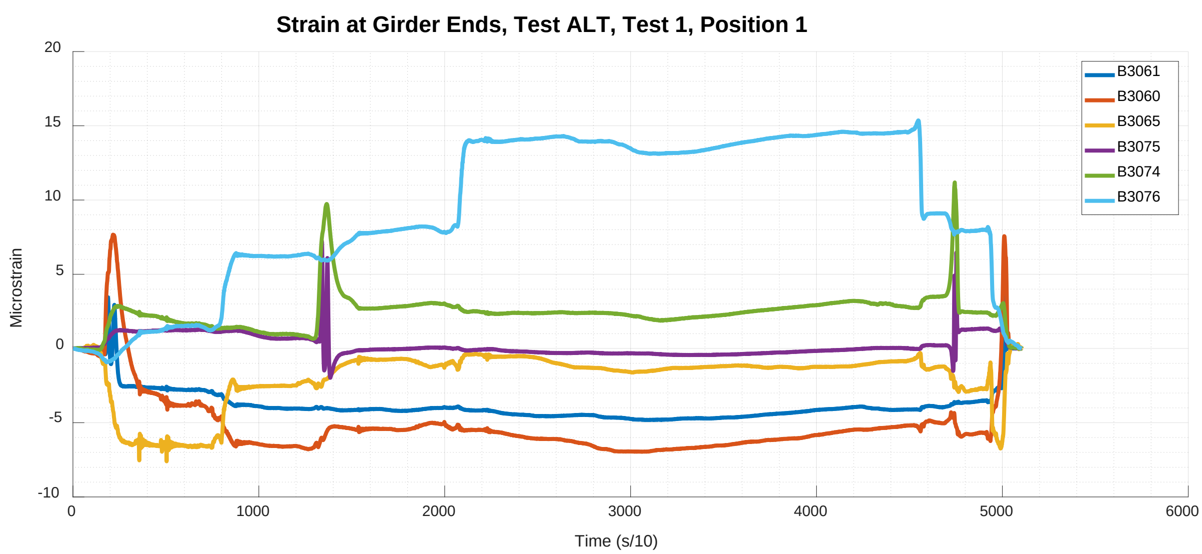

Figure 27: Canton No. 3356 – ALT_1_1 strains – Ends

A.2.5 Rating Factor Calculations

Figure 28: Canton No. 3356 Calculations

AASHTO Rating Calculations:

Bridge 3356 - Canton, Maine

Material Parameters:

Concrete Compressive Strength $f'_c := 5\ ksi$

Reinforcement Yield Strength $F_y := 33\ ksi$

Unit Weights: Reinforced Concrete, Wearing Surface $\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$ $\gamma_{ws} := 0.144\ \frac{kip}{ft^3}$

Concrte Elastic Modulus - LRFD Design C5.4.2.4-1

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Geometric Properties:

Span Length $L := 27\ ft$

Girder Spacing - Interior and Exterior $S := 67.5\ in$ $S_x := 46\ in$

Number of Girders $NG := 6$

Skew Angle $skew := 0\ °$

Width and Number of Lanes $lanewidth := 13.5\ ft$ $Nlane := 2$

Wearing Surface Thickness $ws := 4\ in$

Girder Height - Interior, Exterior $h := 28\ in$ $h_x := 28\ in$

Deck Thickness $d_s := 6.5\ in$

Web Height - Interior, Exterior $d_g := h - d_s$ $d_{gx} := h_x - d_s$

Web Thickness - Interior, Exterior $b_w := 18.5\ in$ $b_{wx} := 18.5\ in$

Distance from Centerline of Girder to Edge of Curb $d_e := -5.75\ in$

Height of Curb Above Deck $h_{curb} := 12\ in$

Width of Curb $b_{curb} := 18\ in$

Height to Centroid of Reinforcement - Interior, Exterior

$$y_{bar} := \begin{bmatrix} 3 \\ 4.67 \\ 5.5 \\ 4.67 \\ 3 \end{bmatrix} \textbf{in} \qquad y_{barx} := \begin{bmatrix} 3 \\ 4.67 \\ 5.5 \\ 4.67 \\ 3 \end{bmatrix} \textbf{in}$$

Dictance Between Centroids of Deck and Web

$$e_g := \frac{d_g + d_s}{2} = 14\ \textbf{in}$$

Depth of Reinforcement - Interior, Exterior

$$d := h - y_{bar} \qquad d_x := h_x - y_{barx} + h_{curb}$$

Area of Reinforcement at All Changes - Interior, Exterior

$$A_s := \begin{bmatrix} 5.0625 \\ 7.0625 \\ 9.0625 \\ 7.0625 \\ 5.0625 \end{bmatrix} \textbf{in}^2 \qquad A_{sx} := \begin{bmatrix} 5.0625 \\ 7.0625 \\ 9.0625 \\ 7.0625 \\ 5.0625 \end{bmatrix} \textbf{in}^2$$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder

$$P_{dint} := 0\ \textbf{kip}$$

Resistance Factors - Structural Dead load, Weading Surface, Live Load

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.25 \quad \gamma_{LL} := 1.35$$

Impact Factor

$$IM := 0.33$$

Resistance Factor

$$\phi := .9$$

System Factor

$$\phi_s := 1.0$$

Condition Factor

$$\phi_c := 1.0$$

Distributed Load Due to Rail

$$w_{rail} := 0.017\ \frac{\textbf{kip}}{\textbf{ft}}$$

Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway

$$exc := 2.52\ \textbf{in}$$

$$P_{dext} := \frac{P_{dint}}{2}$$

Distribution Factors

Area and Moment of Inertia of Web $A := d_g \cdot b_w = 397.75\ in^2$ $I := \dfrac{b_w \cdot d_g^{\ 3}}{12} = \left(1.532 \cdot 10^4\right)\ in^4$

$n := 1$

Modular Ratio - Deck and Web

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(9.328 \cdot 10^4\right)\ in^4$

Interior Moment Distribution Factor

$$g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.496$$

$$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.611$$

$$g_m := \max\left(g_{m1}, g_{m2}\right)$$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From Center of Gravity of Girders $e_1 := \dfrac{W_r}{2} - 5\ ft + exc = 8.71\ ft$

Eccentricity of Exterior Girder From Center of Gravity of Girders $X_{ext} := (NG - 1) \cdot \dfrac{S}{2} = 14.063\ ft$

Eccentricity of Each Girder

$$x_1 := X_{ext}$$
$$x_2 := X_{ext} - S$$
$$x_3 := X_{ext} - 2 \cdot S$$

$$x_4 := \text{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ ft\right)$$
$$x_5 := \text{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ ft\right)$$
$$x_6 := \text{if}\left(NG > 5, X_{ext} - 5 \cdot S, 0\ ft\right)$$
$$x_7 := \text{if}\left(NG > 6, X_{ext} - 6 \cdot S, 0\ ft\right)$$

Lever Rule - One Design Lane

$$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2} + x_6^{\ 2} + x_7^{\ 2}} = 0.388$$

$$g_{mR1} := \text{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0$$

Lever Rule - Two Design Lanes

$$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot (e_1 - 5\ ft)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2 + x_7^2} = 0.428$$

$$g_{mR2} := \text{if}\ (P_{dint} > 0, R_2, 0) = 0$$

Exterior Moment Distribution Factor

$$g_{mx1} := \frac{1.2\ (S + d_e - 2\ ft)}{2 \cdot S} = 0.336$$

$$ee := 0.77 + \frac{d_e}{9\ ft} = 0.717$$

$$g_{mx2} := g_{m2} \cdot ee = 0.438$$

$$g_{mx} := \max\ (g_{mx1}, g_{mx2})$$

<u>Interior</u>	<u>Exterior</u>
$g_m = 0.611$	$g_{mx} = 0.438$

Loading

Interior Girder Dead Load

$$w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.414\ \frac{kip}{ft}$$

Deck Dead Load

$$w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.457\ \frac{kip}{ft}$$

Curb Dead Load

$$w_{curb} := \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.225\ \frac{kip}{ft}$$

Dead Load from Nonstructural Components

$$w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.055\ \frac{kip}{ft}$$

Total Dead Load

$$DC := w_{girder} + w_{deck} + w_{ns} = 0.926\ \frac{kip}{ft}$$

Exterior Girder Dead Load

$$w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.414\ \frac{kip}{ft}$$

Exterior Deck Dead Load

$$w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.311\ \frac{kip}{ft}$$

Total Exterior Dead Load $DC_x := w_{girderx} + w_{deckx} + w_{ns} = 0.78\ \frac{kip}{ft}$

Wearing Surface Dead Load $DW := \gamma_{ws} \cdot ws \cdot S = 0.27\ \frac{kip}{ft}$

Exterior Wearing Surface Dead Load $DW_x := \gamma_{ws} \cdot ws \cdot S_x = 0.184\ \frac{kip}{ft}$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} + P_{dint} \cdot \frac{L}{4} = 84.368\ ft \cdot kip \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} + P_{dext} \cdot \frac{L}{4} = 71.103\ ft \cdot kip$$

$$M_{DW} := \frac{DW \cdot L^2}{8} = 24.604\ ft \cdot kip \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8} = 16.767\ ft \cdot kip$$

Live Load Moment - Truck Load

$$M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 206\ ft \cdot kip$$

Live Load Moment - Tandem

$$M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 287.5\ ft \cdot kip$$

Live Load Moment - Lane

$$M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 58.32\ ft \cdot kip$$

Total HL-93 Live Load

$$M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right) = 440.695\ ft \cdot kip$$

Nominal Resistance

Depth of Interior Whitney Stress Block

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 0.582 \\ 0.812 \\ 1.042 \\ 0.812 \\ 0.582 \end{bmatrix} \boldsymbol{in}$$

Depth of Exterior Whitney Stress Block

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 0.855 \\ 1.192 \\ 1.53 \\ 1.192 \\ 0.855 \end{bmatrix} \boldsymbol{in}$$

Nominal Interior Moment Resistance

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 343.993 \\ 445.223 \\ 547.752 \\ 445.223 \\ 343.993 \end{bmatrix} \boldsymbol{ft \cdot kip}$$

Midspan Moment Capacity

$$M_{capacity} := \max\left(M_n\right) = 547.752 \ \boldsymbol{ft \cdot kip}$$

Nominal Exterior Moment Resistance

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 509.161 \\ 674.598 \\ 840.743 \\ 674.598 \\ 509.161 \end{bmatrix} (\boldsymbol{ft \cdot kip})$$

Midspan Moment Capacity

$$M_{capacityx} := \max\left(M_{nx}\right) = 840.743 \ \boldsymbol{ft \cdot kip}$$

Rating Factors

$$RF_{Interior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacity} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m}$$

$$RF_{Exterior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx} - \gamma_{DC} \cdot M_{DCx} - \gamma_{DW} \cdot M_{DWx}}{\gamma_{LL} \cdot M_{LL} \cdot g_{mx}}$$

Interior	Exterior
$RF_{Interior} = 0.981$	$RF_{Exterior} = 2.482$

Parameters for Extended Girder

Geometry

$$S_{x2} := 39.75\ \mathbf{in}$$

$$h_{x2} := 28\ \mathbf{in}$$

$$d_{gx2} := h_{x2} - d_s$$

$$b_{wx2} := 14\ \mathbf{in}$$

$$d_{e2} := -7\ \mathbf{in}$$

$$h_{curb2} := 13.5\ \mathbf{in}$$

$$b_{curb2} := 17\ \mathbf{in}$$

$$y_{barx2} := \begin{bmatrix} 3 \\ 4.6 \\ 5 \\ 4.6 \\ 3 \end{bmatrix} \mathbf{in}$$

$$d_{x2} := h_{x2} - y_{barx2} + h_{curb2}$$

$$A_{sx2} := \begin{bmatrix} 1.325 \\ 2.209 \\ 2.65 \\ 2.209 \\ 1.325 \end{bmatrix} \mathbf{in}^2$$

Load

$$w_{girderx2} := \gamma_{RC} \cdot b_{wx2} \cdot d_{gx2} = 0.314\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$w_{deckx2} := \gamma_{RC} \cdot S_{x2} \cdot d_s = 0.269\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$DC_{x2} := w_{girderx2} + w_{deckx2} + w_{ns} = 0.637\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$DW_{x2} := \gamma_{ws} \cdot ws \cdot S_{x2} = 0.159\ \frac{\mathbf{kip}}{\mathbf{ft}}$$

$$M_{DCx2} := \frac{DC_{x2} \cdot L^2}{8} = 58.063\ \mathbf{ft} \cdot \mathbf{kip} \qquad M_{DWx2} := \frac{DW_{x2} \cdot L^2}{8} = 14.489\ \mathbf{ft} \cdot \mathbf{kip}$$

Capacity

$$a_{x2} := A_{sx2} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_{x2}} = \begin{bmatrix} 0.259 \\ 0.432 \\ 0.518 \\ 0.432 \\ 0.259 \end{bmatrix} \; \boldsymbol{in}$$

$$M_{nx2} := \overrightarrow{F_y \cdot A_{sx2} \cdot \left(d_{x2} - \frac{a_{x2}}{2} \right)} = \begin{bmatrix} 139.813 \\ 222.848 \\ 264.108 \\ 222.848 \\ 139.813 \end{bmatrix} \; \boldsymbol{ft \cdot kip}$$

$$M_{capacityx2} := \max\left(M_{nx2}\right) = 264.108 \; \boldsymbol{ft \cdot kip}$$

Rating Factor

$$RF_{Exterior2} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx2} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m} = 0.279$$

Rating Factor Improvements

Interior

Maximum Recorded Strain $\varepsilon_T := 75.38 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 329.16\ ft \cdot kip$

Uncracked Section Modulus $S_{unc} := 4304\ in^3$

Cracked Section Modulus $S_{cr} := 1478\ in^3$

Section Behavior $Behavior := \text{"Uncracked"}$

Section Modulus Effective for Behavior

$$S_e := \text{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(4.304 \cdot 10^3\right)\ in^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.378 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.828$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.747$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.414$

Improved Rating Factor $RF_{Improved} := RF_{Interior} \cdot k = 1.388$

Exterior

Maximum Recorded Strain $\varepsilon_T := 17.18 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 329.16\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

Uncracked Section Modulus $S_{unc} := 6902\ \boldsymbol{in}^3$

Cracked Section Modulus $S_{cr} := 2522\ \boldsymbol{in}^3$

Section Behavior $Behavior := \text{"Uncracked"}$

Section Modulus Effective for Behavior

$$S_e := \mathbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(6.902 \cdot 10^3\right)\ \boldsymbol{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 6.159 \cdot 10^{-5}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 2.585$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.747$

Test Understanding Factor $k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 2.293$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 5.691$

Exterior - Extended Girder

Maximum Recorded Strain $\varepsilon_T := 81.73 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 329.16\ ft \cdot kip$

Uncracked Section Modulus $S_{unc} := 4641\ in^3$

Cracked Section Modulus $S_{cr} := 1035\ in^3$

Section Behavior $Behavior := \text{“Uncracked”}$

Section Modulus Effective for Behavior

$$S_e := \text{if}\left(Behavior = \text{“Uncracked”}, S_{unc}, S_{cr}\right) = \left(4.641 \cdot 10^3\right)\ in^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 9.168 \cdot 10^{-5}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.122$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.747$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.061$

Improved Rating Factor $RF_{ImprovedExt2} := RF_{Exterior2} \cdot k = 0.296$

Summary

Interior	Exterior	Exterior- Extended
$RF_{Improved} = 1.388$	$RF_{ImprovedExt} = 5.69$	$RF_{ImprovedExt2} = 0.296$

Bridge 3356

Interior Section Properties

$y_{unc} := 16.8\ in$ $I_{unc} := 72270\ in^4$ $S_{unc} := 4304\ in^3$ $N_i := 4$

$y_{cr} := 21.3\ in$ $I_{cr} := 31529\ in^4$ $S_{cr} := 1478\ in^3$

Exterior Section Properties (Averaged Between Original and Extended Girder)

$y_{uncx} := 21\ in$ $I_{uncx} := 119966\ in^4$ $S_{uncx} := 5771.5\ in^3$ $N_x := 2$

$y_{crx} := 27.6\ in$ $I_{crx} := 48118.5\ in^4$ $S_{crx} := 1778.5\ in^3$

Test Parameters

Maximum Applied Moment
$M := 658.32\ ft \cdot kip$

Maximum Measured Strain
$\varepsilon_m := 81.73 \cdot 10^{-6}$

Using Nominal Compressive Strength - 2.5 ksi

$f'_c := 2.5\ ksi$ $E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(2.878 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 9.546 \cdot 10^{-5}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.899 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Using More Realistic Compressive Strength - 5 ksi

$f'_c := 5\ ksi$ $E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 6.75 \cdot 10^{-5}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.05 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Cracked"
  "Uncracked"
else
  "Cracked"

Distributiuon Factors from Testing

Bridge 3356

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder $S := \begin{bmatrix} 4641 \\ 4304 \\ 4304 \\ 4304 \\ 4304 \\ 6902 \end{bmatrix} in^3$

Strains Recorded During SBS & MAX Tests

$$\varepsilon_2 := \begin{bmatrix} 50.55 \\ 49.78 \\ 59.09 \\ 49.05 \\ 21.96 \\ 9.925 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 81.73 \\ 69.79 \\ 75.38 \\ 63.91 \\ 30.41 \\ 17.18 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity $E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

Moment Produced in Each Girder

$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 79.562 \\ 72.661 \\ 86.25 \\ 71.596 \\ 32.054 \\ 23.232 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 128.638 \\ 101.869 \\ 110.028 \\ 93.286 \\ 44.388 \\ 40.214 \end{bmatrix} ft \cdot kip$$

Total Moment

$$M_{sum2} := \sum M_2 = 365.355\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 518.422\ ft \cdot kip$$

Distribution Factors

$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.436 \\ 0.398 \\ 0.472 \\ 0.392 \\ 0.175 \\ 0.127 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.496 \\ 0.393 \\ 0.424 \\ 0.36 \\ 0.171 \\ 0.155 \end{bmatrix}$$

Distributiuon Factors from Testing
Bridge 2130

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder
$$S := \begin{bmatrix} 4641 \\ 4304 \\ 4304 \\ 4304 \\ 4304 \\ 6902 \end{bmatrix} in^3$$

Strains Recorded During SBS & MAX Tests
$$\varepsilon_2 := \begin{bmatrix} 50.55 \\ 49.78 \\ 59.09 \\ 49.05 \\ 21.96 \\ 9.925 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 81.73 \\ 69.79 \\ 75.38 \\ 63.91 \\ 30.41 \\ 17.18 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity
$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Moment Produced in Each Girder
$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 79.562 \\ 72.661 \\ 86.25 \\ 71.596 \\ 32.054 \\ 23.232 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 128.638 \\ 101.869 \\ 110.028 \\ 93.286 \\ 44.388 \\ 40.214 \end{bmatrix} ft \cdot kip$$

Total Moment
$$M_{sum2} := \sum M_2 = 365.355\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 518.422\ ft \cdot kip$$

Distribution Factors
$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.436 \\ 0.398 \\ 0.472 \\ 0.392 \\ 0.175 \\ 0.127 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.496 \\ 0.393 \\ 0.424 \\ 0.36 \\ 0.171 \\ 0.155 \end{bmatrix}$$

A.3 Peru No. 5432

A.3.1 Input Data, Experimental Configuration, and Experimental Data Collected

Table 19: Peru No. 5432 input data, experimental configuration, and experimental data collected

File Contents	File Name	File Type
Exterior Section Data	Br5432_Geom_Ext.csv	CSV Format
Interior Section Data	Br5432_Geom_Int.csv	CSV Format
Sensors	Br5432_Sensors.csv	CSV Format
Sensor Layout	Br5432_SensorLayout.mat	MATLAB Data File
Truck Weight and Dimensions	Br5432_Trucks.mat	MATLAB Data File
Sensor Data	Br_5432_ALT_1_1_Strain.mat	MATLAB Data File
	Br_5432_ALT_2_1_Strain.mat	MATLAB Data File
	Br_5432_ALT_3_1_Strain.mat	MATLAB Data File
	Br_5432_BTB_1_1_Strain.mat	MATLAB Data File
	Br_5432_BTB_1_2_Strain.mat	MATLAB Data File
	Br_5432_BTB_2_1_Strain.mat	MATLAB Data File
	Br_5432_MAX_1_1_Strain.mat	MATLAB Data File
	Br_5432_MAX_1_2_Strain.mat	MATLAB Data File
	Br_5432_MAX_2_1_Strain.mat	MATLAB Data File
	Br_5432_MAX_3_1_Strain.mat	MATLAB Data File
	Br_5432_SBS_1_1_Strain.mat	MATLAB Data File
	Br_5432_SBS_1_2_Strain.mat	MATLAB Data File
	Br_5432_SBS_2_1_Strain.mat	MATLAB Data File

A.3.2 Instrumentation

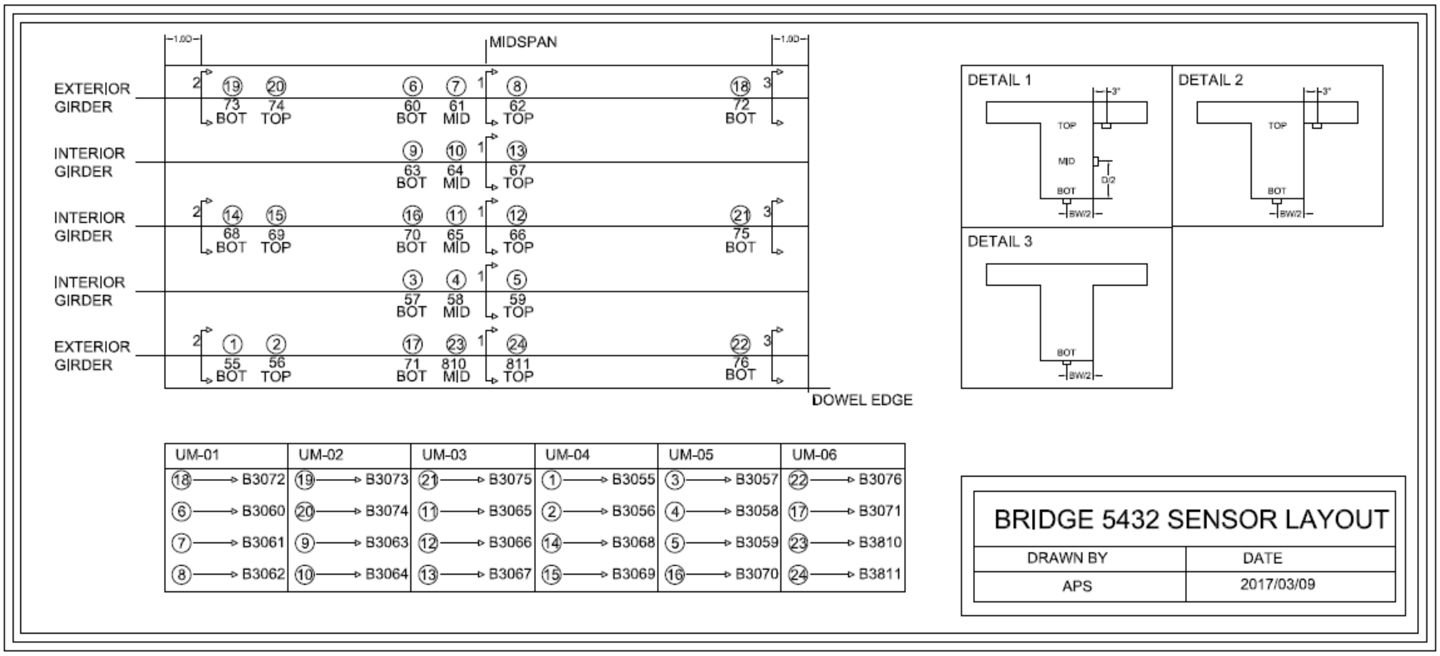

Figure 29: Peru No. 5432 sensor layout

A.3.3 Loading

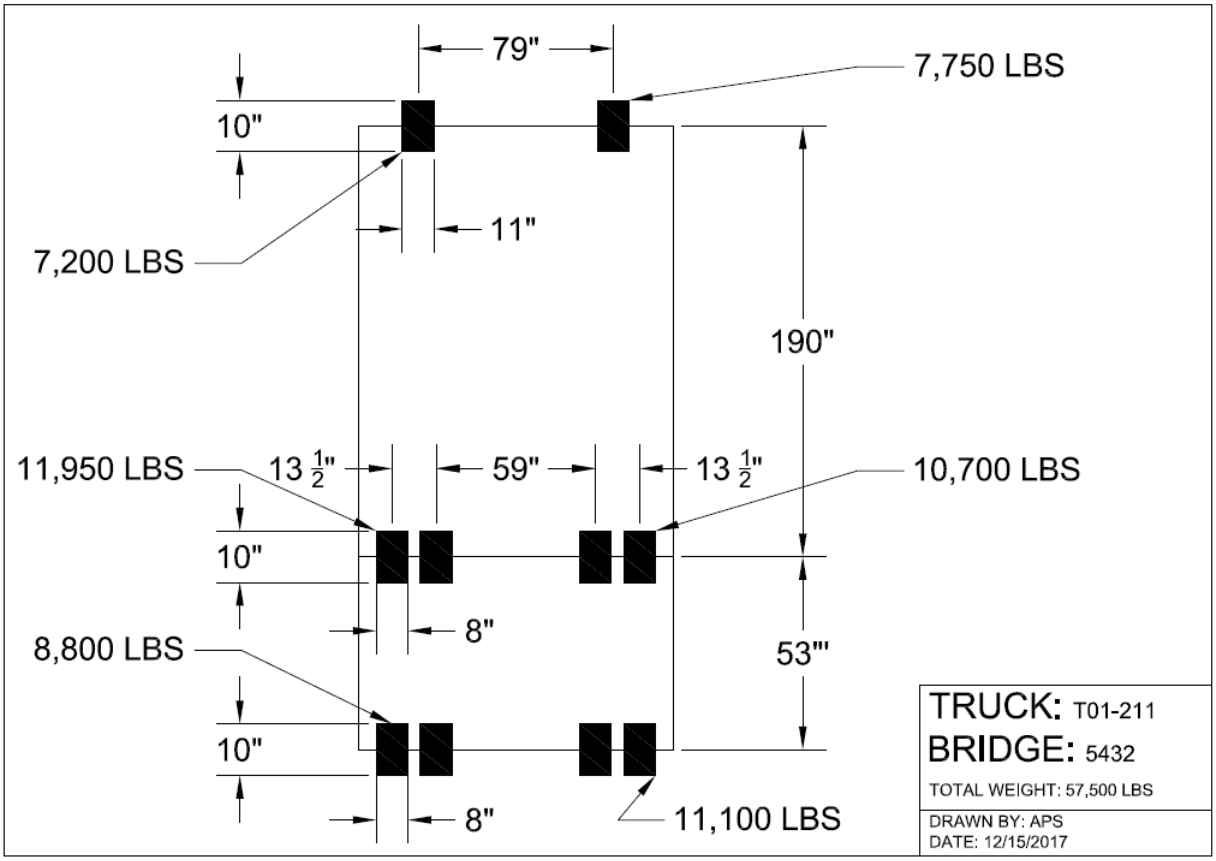

Figure 30: Peru No. 5432 Truck T01-211 loading

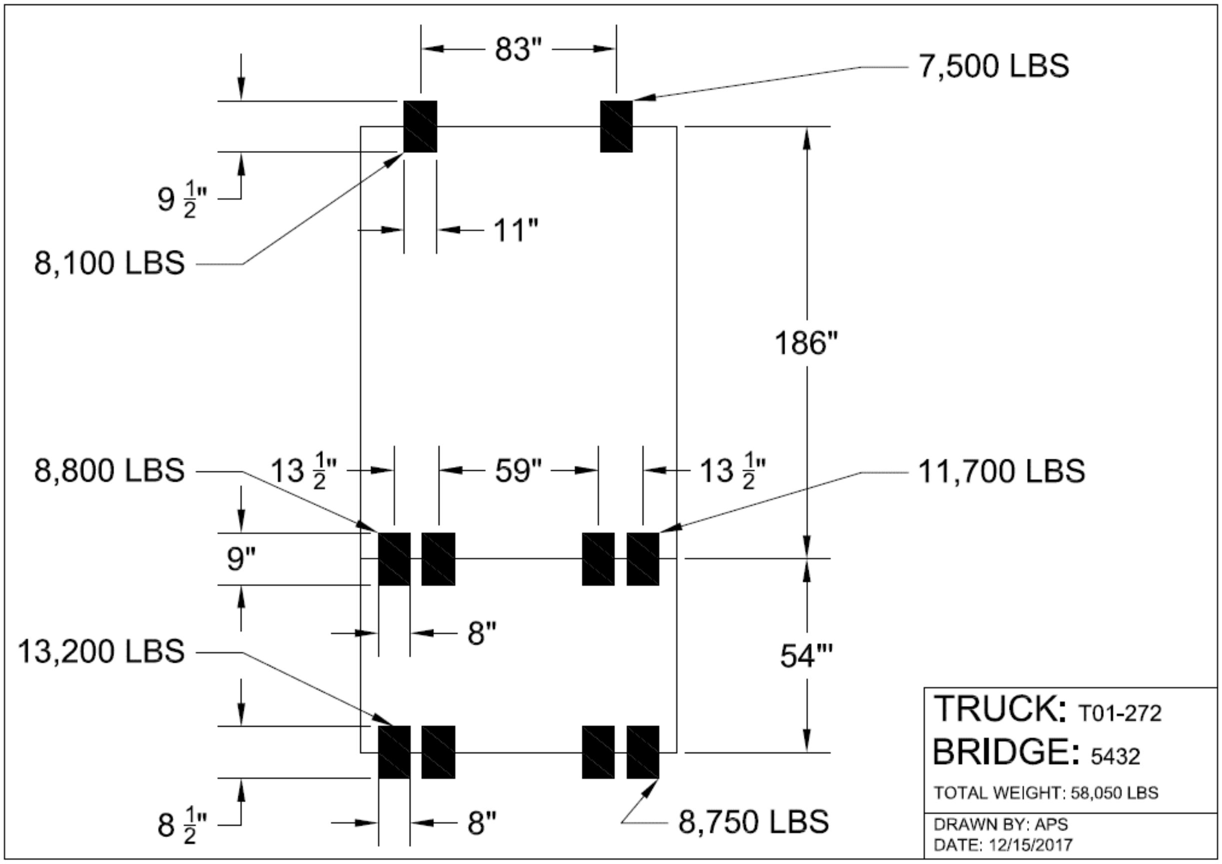

Figure 31: Peru No. 5432 Truck T01-272 loading

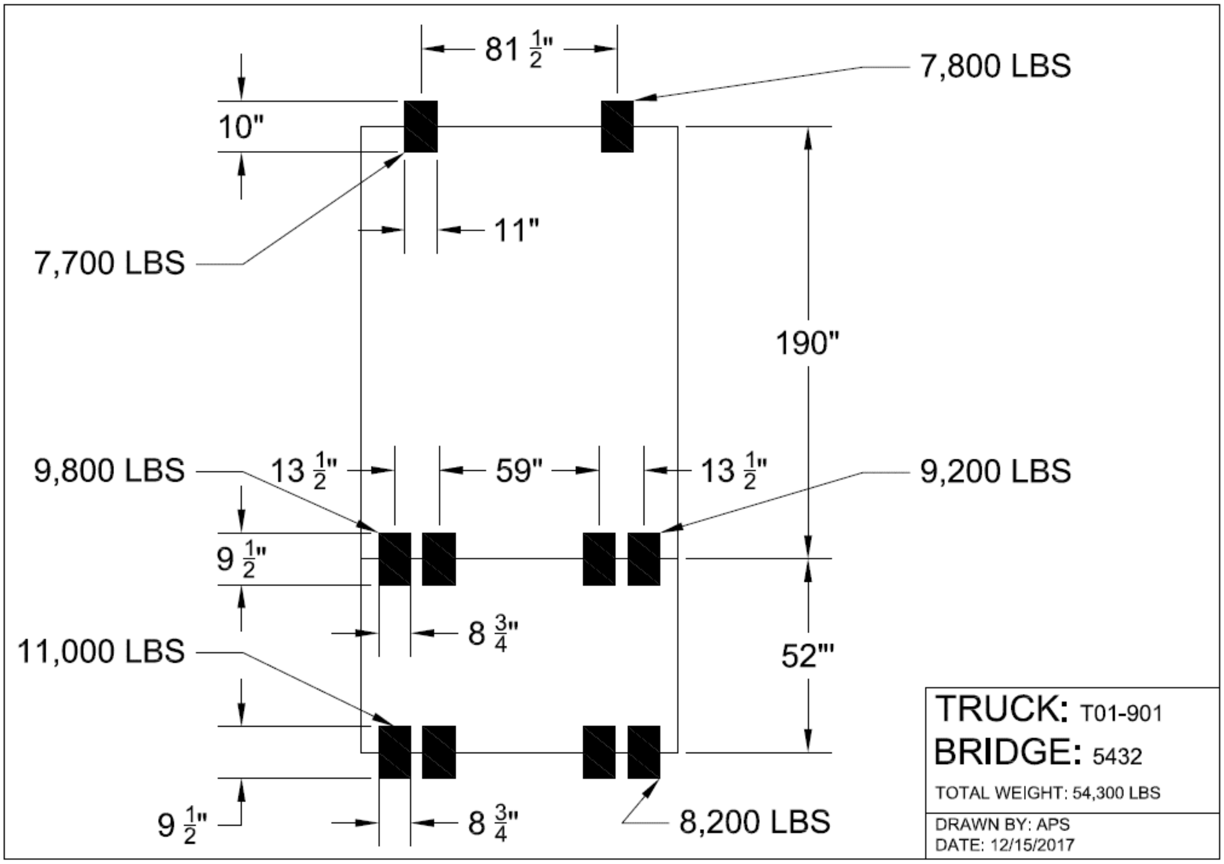

Figure 32: Peru No. 5432 Truck T01-901 loading

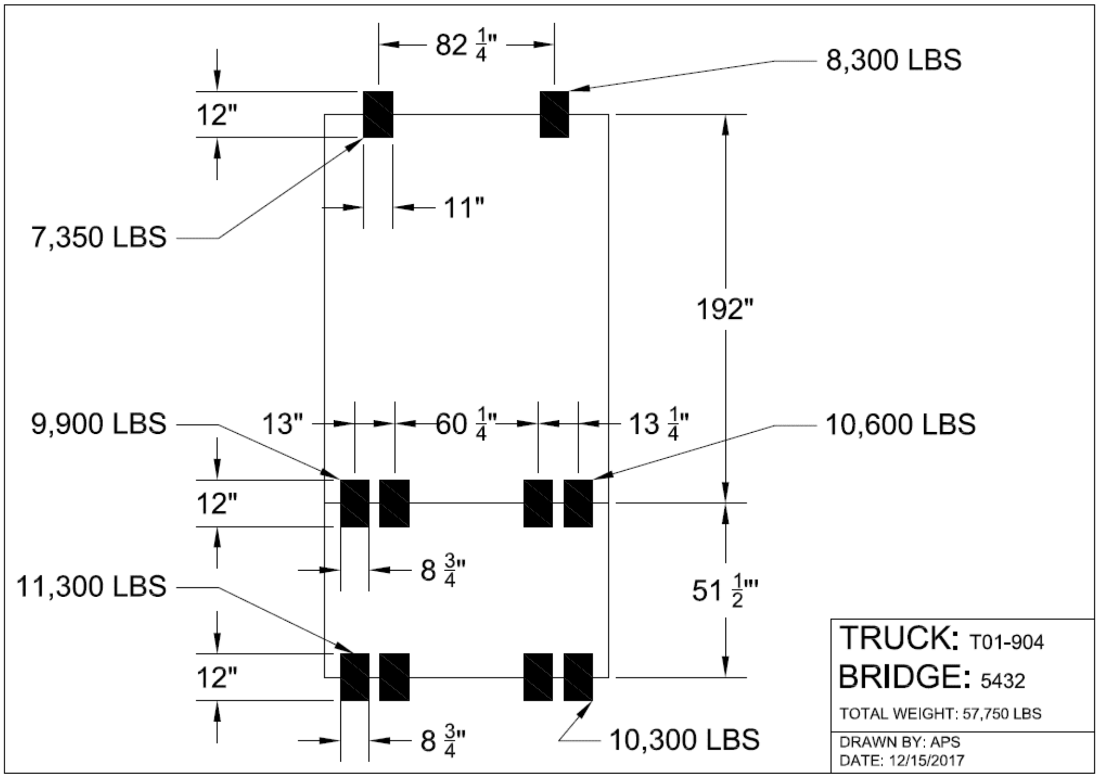

Figure 33: Peru No. 5432 Truck T01-904 loading

A.3.4 Representative Data Plots

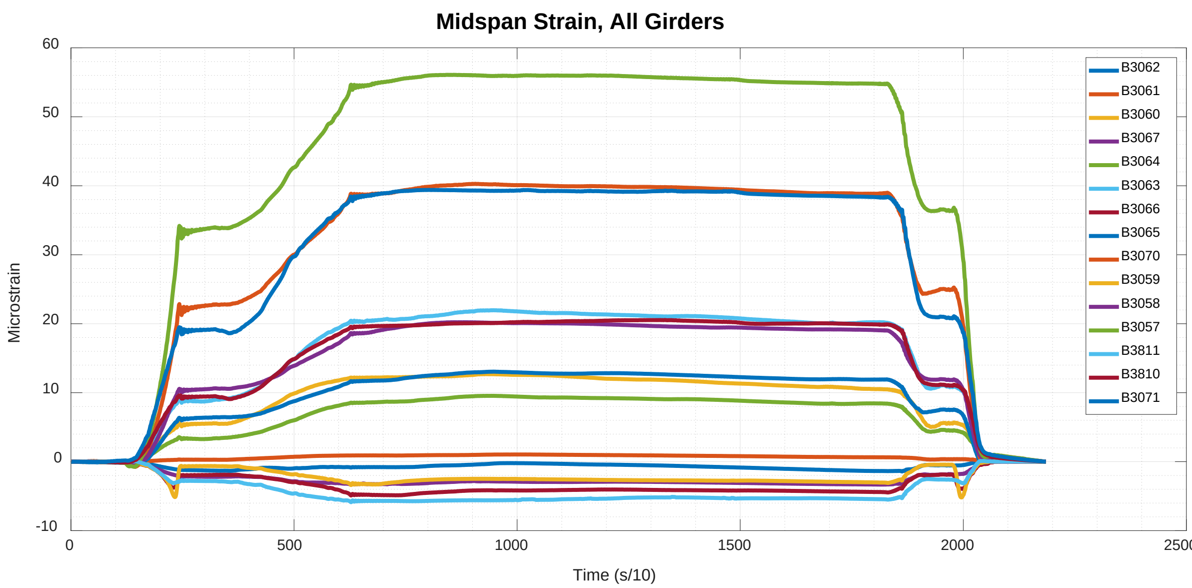

Figure 34: Peru No. 5432 – BTB_2_1 strains - Midspan

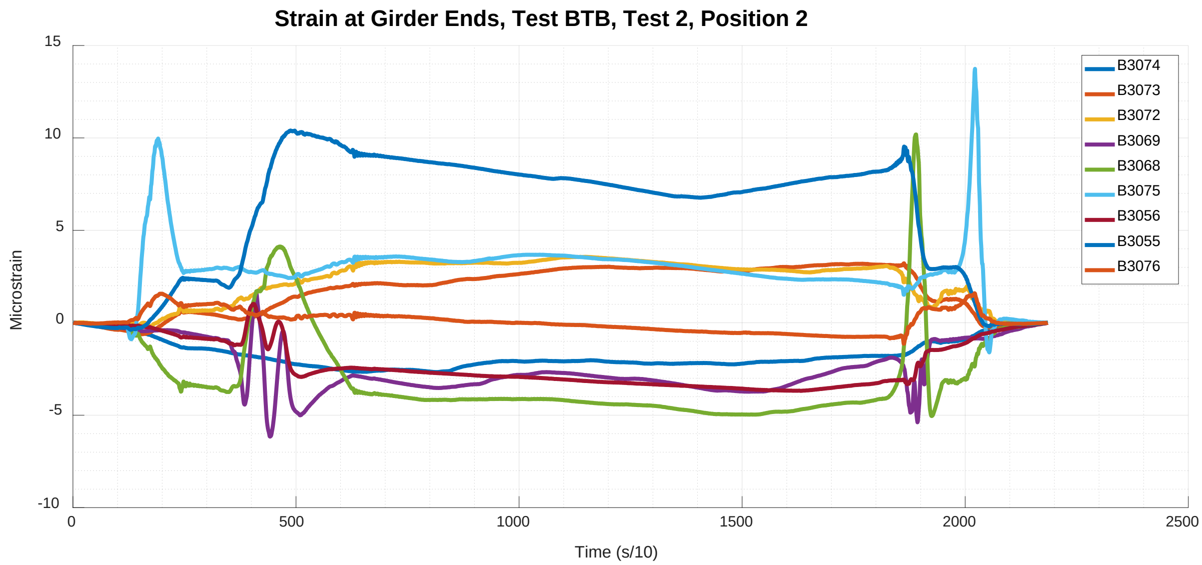

Figure 35: Peru No. 5432 – BTB_2_1 strains - Ends

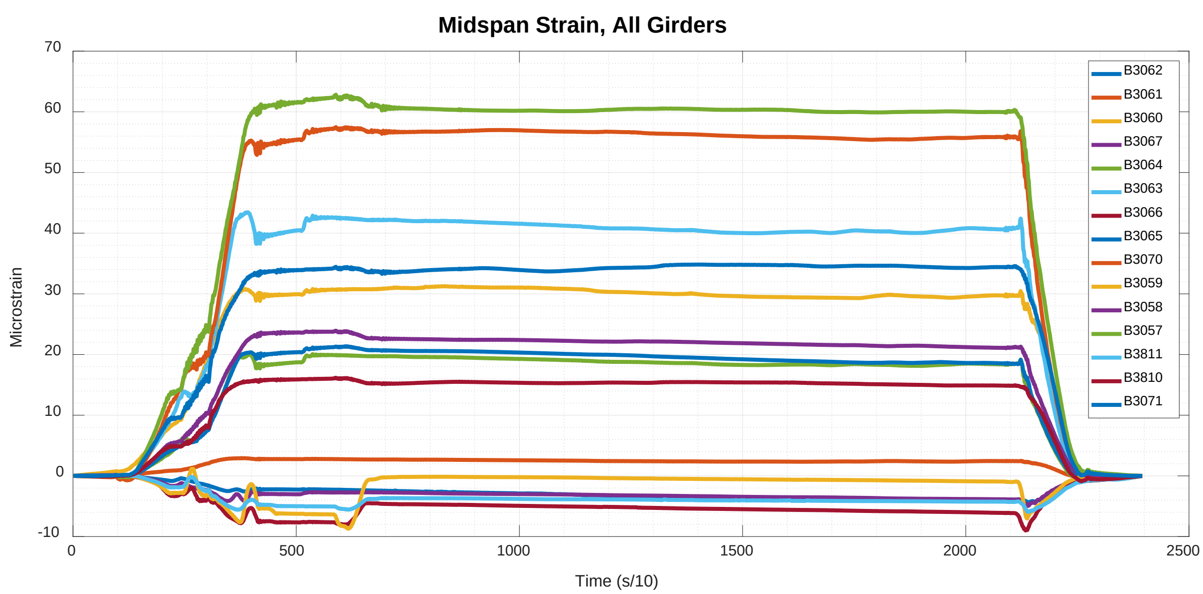

Figure 36: Peru No. 5432 – SBS_2_1 - Midspan

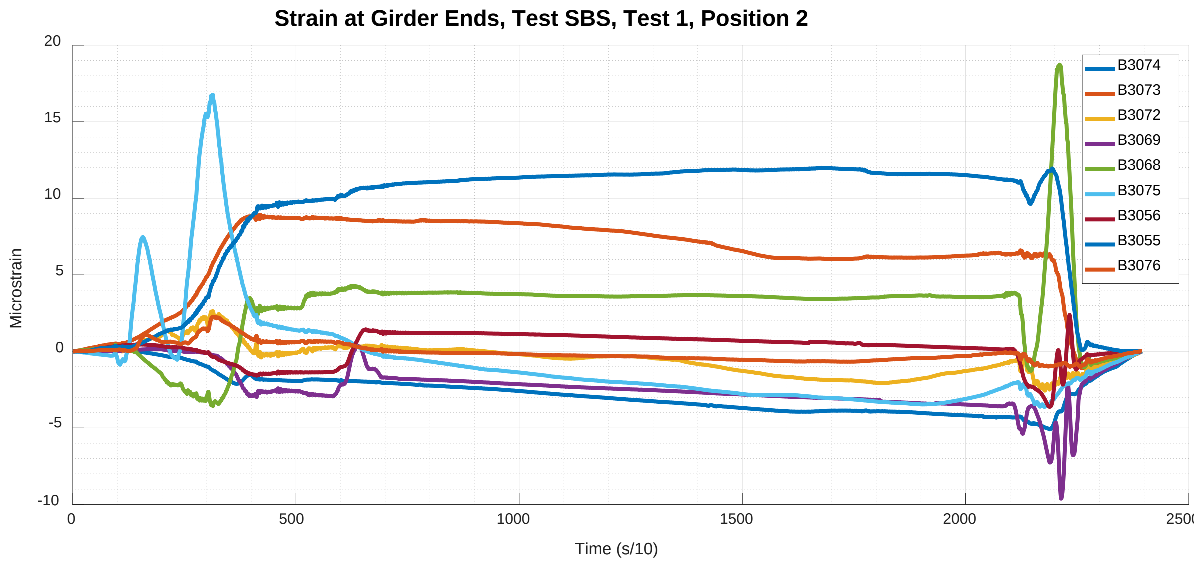

Figure 37: Peru No. 5432 – SBS_2_1 – Ends

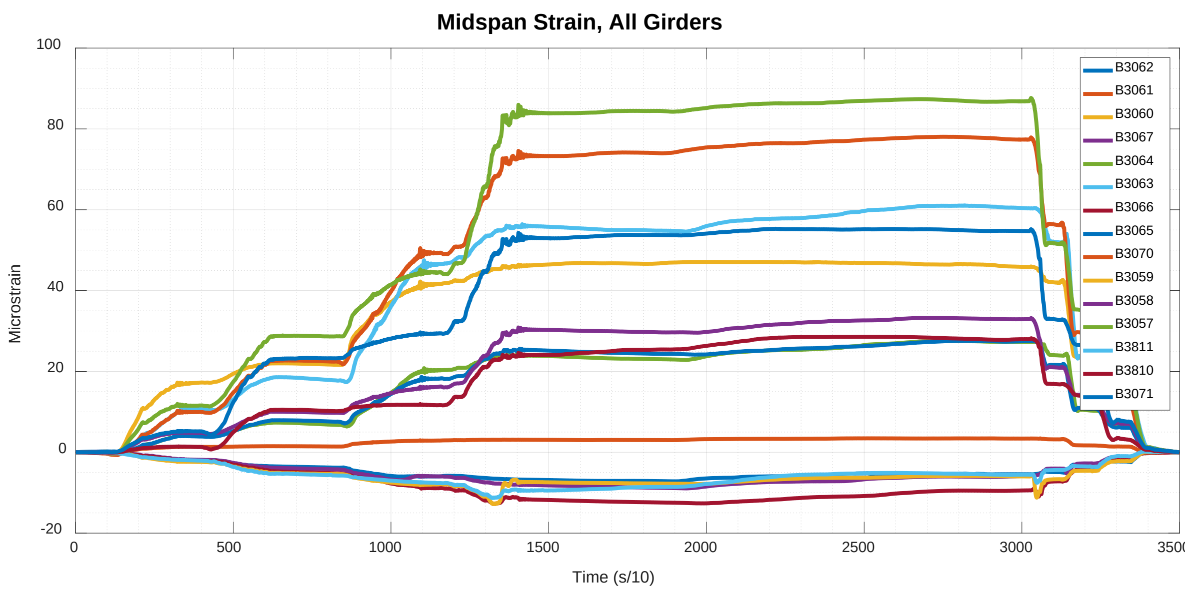

Figure 38: Peru No. 5432 – MAX_2_1 - Midspan

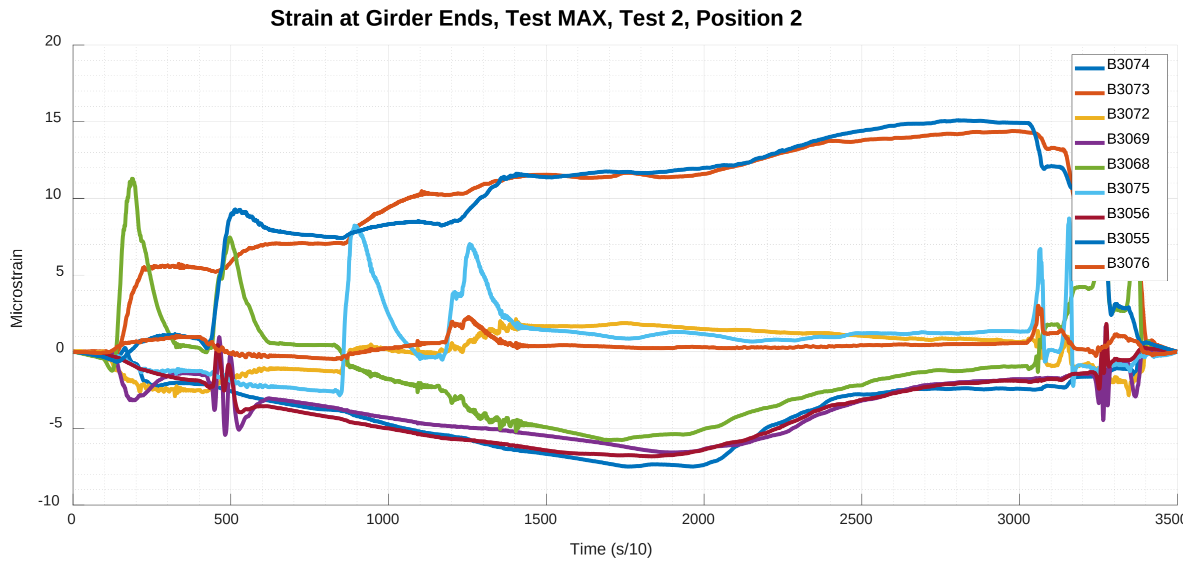

Figure 39: Peru No. 5432 – MAX_2_1 - Ends

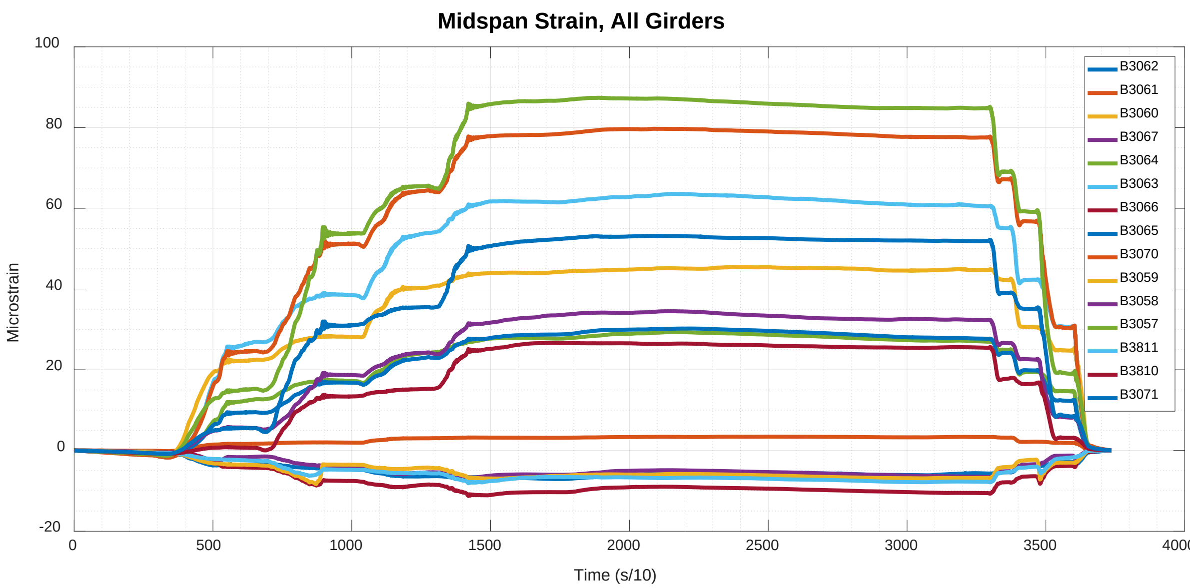

Figure 40: Peru No. 5432 – ALT_2_1 - Midspan

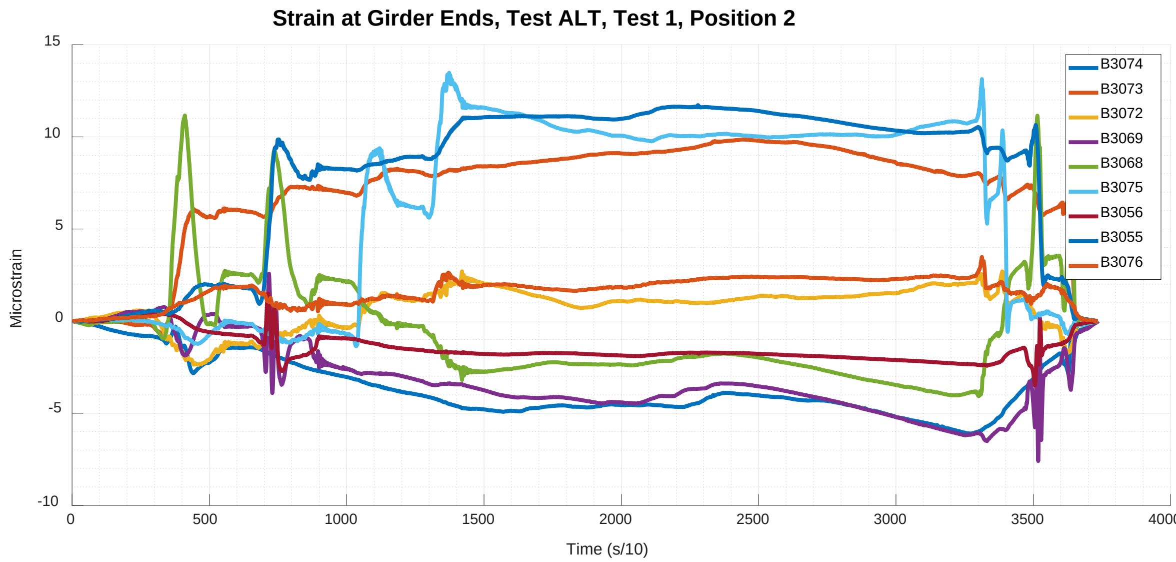

Figure 41: Peru No. 5432 – ALT_2_1 - Ends

A.3.5 Rating Factor Calculations

Figure 42: Peru No. 5432 Calculations

AASHTO Rating Calculations:

Bridge 5432 - Peru, Maine

Material Parameters:

Concrete Compressive Strength $f'_c := 5\ ksi$

Reinforcement Yield Strength $F_y := 33\ ksi$

Unit Weights: Reinforced Concrete, Wearing Surface $\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$ $\gamma_{ws} := 0.144\ \frac{kip}{ft^3}$

Concrte Elastic Modulus - LRFD Design C5.4.2.4-1

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Geometric Properties:

Span Length $L := 40\ ft$

Girder Spacing - Interior and Exterior $S := 76\ in$ $S_x := 51\ in$

Number of Girders $NG := 5$

Skew Angle $skew := 0\ °$

Width and Number of Lanes $lanewidth := 12\ ft$ $Nlane := 2$

Wearing Surface Thickness $ws := 4\ in$

Girder Height - Interior, Exterior $h := 35.75\ in$ $h_x := 35.75\ in$

Deck Thickness $d_s := 5.5\ in$

Web Height - Interior, Exterior $d_g := h - d_s$ $d_{gx} := h_x - d_s$

Web Thickness - Interior, Exterior $b_w := 20\ in$ $b_{wx} := 20\ in$

Distance from Centerline of Girder to Edge of Curb $d_e := -8\ in$

Height of Curb Above Deck $h_{curb} := 12\ in$

Width of Curb $b_{curb} := 21\ in$

Height to Centroid of Reinforcement - Interior, Exterior

$$y_{bar} := \begin{bmatrix} 4.346 \\ 5.028 \\ 6.24 \\ 7.068 \\ 6.24 \\ 5.028 \\ 4.346 \end{bmatrix} in \qquad y_{barx} := \begin{bmatrix} 4.346 \\ 5.028 \\ 6.24 \\ 7.068 \\ 6.24 \\ 5.028 \\ 4.346 \end{bmatrix} in$$

Dictance Between Centroids of Deck and Web

$$e_g := \frac{d_g + d_s}{2} = 17.875 \ in$$

Depth of Reinforcement - Interior, Exterior

$$d := h - y_{bar} \qquad d_x := h_x - y_{barx} + h_{curb}$$

Area of Reinforcement at All Changes - Interior, Exterior

$$A_s := \begin{bmatrix} 7.0625 \\ 9.0625 \\ 11.0625 \\ 13.0625 \\ 11.0625 \\ 9.0625 \\ 7.0625 \end{bmatrix} in^2 \qquad A_{sx} := \begin{bmatrix} 7.0625 \\ 9.0625 \\ 11.0625 \\ 13.0625 \\ 11.0625 \\ 9.0625 \\ 7.0625 \end{bmatrix} in^2$$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder

$$P_{dint} := 2.042 \ kip$$

Resistance Factors - Structural Dead load, Weading Surface, Live Load

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.25 \quad \gamma_{LL} := 1.35$$

Impact Factor

$$IM := 0.33$$

Resistance Factor

$$\phi := .9$$

System Factor

$$\phi_s := 1.0$$

Condition Factor

$$\phi_c := 1.0$$

Distributed Load Due to Rail

$$w_{rail} := 0.109 \ \frac{kip}{ft}$$

Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway

$$exc := 0 \ in$$

$$P_{dext} := \frac{P_{dint}}{2}$$

Distribution Factors

Area and Moment of Inertia of Web $A := d_g \cdot b_w = 605\ in^2$ $I := \frac{b_w \cdot d_g^{\ 3}}{12} = \left(4.613 \cdot 10^4\right)\ in^4$

$n := 1$

Modular Ratio - Deck and Web

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(2.394 \cdot 10^5\right)\ in^4$

Interior Moment Distribution Factor $g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.527$

$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.68$

$g_m := \max\left(g_{m1}, g_{m2}\right)$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From Center of Gravity of Girders $e_1 := \frac{W_r}{2} - 5\ ft + exc = 7\ ft$

Eccentricity of Exterior Girder From Center of Gravity of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 12.667\ ft$

Eccentricity of Each Girder

$x_1 := X_{ext}$

$x_2 := X_{ext} - S$

$x_3 := X_{ext} - 2 \cdot S$

$x_4 := \text{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ ft\right)$

$x_5 := \text{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ ft\right)$

$x_6 := \text{if}\left(NG > 5, X_{ext} - 5 \cdot S, 0\ ft\right)$

$x_7 := \text{if}\left(NG > 6, X_{ext} - 6 \cdot S, 0\ ft\right)$

Lever Rule - One Design Lane $R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2} + x_6^{\ 2} + x_7^{\ 2}} = 0.421$

$g_{mR1} := \text{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0.505$

Lever Rule - Two Design Lanes $R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot (e_1 - 5\ ft)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2 + x_7^2} = 0.463$

$g_{mR2} := \text{if}\,(P_{dint} > 0, R_2, 0) = 0.463$

Exterior Moment Distribution Factor $g_{mx1} := \frac{1.2\ (S + d_e - 2\ ft)}{2 \cdot S} = 0.347$

$ee := 0.77 + \frac{d_e}{9\ ft} = 0.696$

$g_{mx2} := g_{m2} \cdot ee = 0.473$

$g_{mx} := \max\,(g_{mx1}, g_{mx2})$

Interior	Exterior
$g_m = 0.68$	$g_{mx} = 0.473$

Loading

Interior Girder Dead Load $w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.63\ \frac{kip}{ft}$

Deck Dead Load $w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.435\ \frac{kip}{ft}$

Curb Dead Load $w_{curb} := \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.263\ \frac{kip}{ft}$

Dead Load from Nonstructural Components $w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.162\ \frac{kip}{ft}$

Total Dead Load $DC := w_{girder} + w_{deck} + w_{ns} = 1.227\ \frac{kip}{ft}$

Exterior Girder Dead Load $w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.63\ \frac{kip}{ft}$

Exterior Deck Dead Load $w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.292\ \frac{kip}{ft}$

Total Exterior Dead Load

$$DC_x := w_{girderx} + w_{deckx} + w_{ns} = 1.084 \ \frac{kip}{ft}$$

Wearing Surface Dead Load

$$DW := \gamma_{ws} \cdot ws \cdot S = 0.304 \ \frac{kip}{ft}$$

Exterior Wearing Surface Dead Load

$$DW_x := \gamma_{ws} \cdot ws \cdot S_x = 0.204 \ \frac{kip}{ft}$$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} + P_{dint} \cdot \frac{L}{4} = 265.845 \ ft \cdot kip \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} + P_{dext} \cdot \frac{L}{4} = 226.989 \ ft \cdot kip$$

$$M_{DW} := \frac{DW \cdot L^2}{8} = 60.8 \ ft \cdot kip \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8} = 40.8 \ ft \cdot kip$$

Live Load Moment - Truck Load

$$M_{Truck} := 32 \ kip \cdot \left(\frac{L}{4}\right) + \frac{40 \ kip}{2} \cdot \left(\frac{L}{2} - 14 \ ft\right) = 440 \ ft \cdot kip$$

Live Load Moment - Tandem

$$M_{Tandem} := 25 \ kip \cdot \frac{L}{4} + \frac{25 \ kip}{2} \cdot \left(\frac{L}{2} - 4 \ ft\right) = 450 \ ft \cdot kip$$

Live Load Moment - Lane

$$M_{Lane} := 0.64 \ \frac{kip}{ft} \cdot \frac{L^2}{8} = 128 \ ft \cdot kip$$

Total HL-93 Live Load

$$M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right) = 726.5 \ ft \cdot kip$$

Nominal Resistance

Depth of Interior Whitney Stress Block

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 0.722 \\ 0.926 \\ 1.13 \\ 1.335 \\ 1.13 \\ 0.926 \\ 0.722 \end{bmatrix} \; \boldsymbol{in}$$

Depth of Exterior Whitney Stress Block

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 1.075 \\ 1.38 \\ 1.684 \\ 1.989 \\ 1.684 \\ 1.38 \\ 1.075 \end{bmatrix} \; \boldsymbol{in}$$

Nominal Interior Moment Resistance

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 602.918 \\ 754.112 \\ 880.558 \\ 1.006 \cdot 10^3 \\ 880.558 \\ 754.112 \\ 602.918 \end{bmatrix} \; \boldsymbol{ft \cdot kip}$$

Midspan Moment Capacity

$$M_{capacity} := \max\left(M_n\right) = \left(1.006 \cdot 10^3\right) \; \boldsymbol{ft \cdot kip}$$

Nominal Exterior Moment Resistance

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 832.545 \\ 1.048 \cdot 10^3 \\ 1.237 \cdot 10^3 \\ 1.426 \cdot 10^3 \\ 1.237 \cdot 10^3 \\ 1.048 \cdot 10^3 \\ 832.545 \end{bmatrix} \; (\boldsymbol{ft \cdot kip})$$

Midspan Moment Capacity

$$M_{capacityx} := \max\left(M_{nx}\right) = \left(1.426 \cdot 10^3\right) \; \boldsymbol{ft \cdot kip}$$

Rating Factors

$$RF_{Interior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacity} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m}$$

$$RF_{Exterior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx} - \gamma_{DC} \cdot M_{DCx} - \gamma_{DW} \cdot M_{DWx}}{\gamma_{LL} \cdot M_{LL} \cdot g_{mx}}$$

Interior	Exterior
$RF_{Interior} = 0.746$	$RF_{Exterior} = 2.042$

Rating Factor Improvements

Interior

Maximum Recorded Strain $\varepsilon_T := 87.34 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 629.99\ \boldsymbol{ft \cdot kip}$

Uncracked Section Modulus $S_{unc} := 7434\ \boldsymbol{in}^3$

Cracked Section Modulus $S_{cr} := 2519\ \boldsymbol{in}^3$

Section Behavior $Behavior := \text{“Uncracked”}$

Section Modulus Effective for Behavior

$$S_e := \textbf{if}\left(Behavior = \text{“Uncracked”}, S_{unc}, S_{cr}\right) = \left(7.434 \cdot 10^3\right)\ \boldsymbol{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.7 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.946$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.867$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.473$

Improved Rating Factor $RF_{Improved} := RF_{Interior} \cdot k = 1.098$

Exterior

Maximum Recorded Strain $\varepsilon_T := 58.62 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 629.99\ ft \cdot kip$

Uncracked Section Modulus $S_{unc} := 10052\ in^3$

Cracked Section Modulus $S_{cr} := 3475\ in^3$

Section Behavior $Behavior := \text{"Uncracked"}$

Section Modulus Effective for Behavior

$$S_e := \text{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(1.005 \cdot 10^4\right)\ in^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 8.749 \cdot 10^{-5}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.492$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.867$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.246$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 2.545$

Summary

Interior	Exterior
$RF_{Improved} = 1.098$	$RF_{ImprovedExt} = 2.545$

Bridge 5432

Interior Section Properties

$y_{unc} := 20.8\ in$ $\quad I_{unc} := 154658\ in^4$ $\quad S_{unc} := 7434\ in^3$ $\quad N_i := 3$

$y_{cr} := 27.1\ in$ $\quad I_{cr} := 68243\ in^4$ $\quad S_{cr} := 2519\ in^3$

Exterior Section Properties

$y_{uncx} := 23.6\ in$ $\quad I_{uncx} := 237055\ in^4$ $\quad S_{uncx} := 10052\ in^3$ $\quad N_x := 2$

$y_{crx} := 31.2\ in$ $\quad I_{crx} := 108622\ in^4$ $\quad S_{crx} := 3479\ in^3$

Test Parameters

Maximum Applied Moment

$M := 1259.95\ ft \cdot kip$

Maximum Measured Strain

$\varepsilon_m := 87.40 \cdot 10^{-6}$

Using Nominal Compressive Strength - 2.5 ksi

$f'_c := 2.5\ ksi$ $\qquad E_c := 1820\ ksi \cdot \sqrt{\dfrac{f'_c}{ksi}} = \left(2.878 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 1.239 \cdot 10^{-4}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 3.62 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Using More Realistic Compressive Strength - 5 ksi

$f'_c := 5\ ksi$ $\qquad E_c := 1820\ ksi \cdot \sqrt{\dfrac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 8.761 \cdot 10^{-5}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.56 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Distributiuon Factors from Testing

Bridge 5432

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder

$$S := \begin{bmatrix} 10052 \\ 7434 \\ 7434 \\ 7434 \\ 10052 \end{bmatrix} in^3$$

Strains Recorded During SBS & MAX Tests

$$\varepsilon_2 := \begin{bmatrix} 29.96 \\ 20.97 \\ 56.34 \\ 60.52 \\ 34.77 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 46.47 \\ 26.56 \\ 77.96 \\ 87.34 \\ 55.16 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Moment Produced in Each Girder

$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 102.134 \\ 52.868 \\ 142.041 \\ 152.58 \\ 118.531 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 158.416 \\ 66.962 \\ 196.548 \\ 220.197 \\ 188.041 \end{bmatrix} ft \cdot kip$$

Total Moment

$$M_{sum2} := \sum M_2 = 568.154\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 830.164\ ft \cdot kip$$

Distribution Factors

$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.36 \\ 0.186 \\ 0.5 \\ 0.537 \\ 0.417 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.382 \\ 0.161 \\ 0.474 \\ 0.53 \\ 0.453 \end{bmatrix}$$

Distributiuon Factors from Testing

Bridge 5432

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder
$$S := \begin{bmatrix} 10052 \\ 7434 \\ 7434 \\ 7434 \\ 10052 \end{bmatrix} in^3$$

Strains Recorded During SBS & MAX Tests
$$\varepsilon_2 := \begin{bmatrix} 29.96 \\ 60.52 \\ 56.34 \\ 60.52 \\ 34.77 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 46.47 \\ 87.34 \\ 77.96 \\ 87.34 \\ 55.16 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity
$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right) ksi$$

Moment Produced in Each Girder
$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 102.134 \\ 152.58 \\ 142.041 \\ 152.58 \\ 118.531 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 158.416 \\ 220.197 \\ 196.548 \\ 220.197 \\ 188.041 \end{bmatrix} ft \cdot kip$$

Total Moment
$$M_{sum2} := \sum M_2 = 667.865\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 983.399\ ft \cdot kip$$

Distribution Factors
$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.306 \\ 0.457 \\ 0.425 \\ 0.457 \\ 0.355 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.322 \\ 0.448 \\ 0.4 \\ 0.448 \\ 0.382 \end{bmatrix}$$

A.4 Jackson No. 3776

A.4.1 Input Data, Experimental Configuration, and Experimental Data Collected

Table 13: Jackson No. 3776 input data, experimental configuration, and experimental data collected

File Contents	File Name	File Type
Exterior Section Data	Br3776_Geom _Ext.csv	CSV Format
Interior Section Data	Br3776_Geom _Int.csv	CSV Format
Sensors	Br3776 _Sensors.csv	CSV Format
Sensor Layout	Br3776 _SensorLayout.mat	MATLAB Data File
Truck Weight and Dimensions	Br3776_Trucks.mat	MATLAB Data File
Sensor Data	Br_3776_ALT_1_1_Strain.mat	MATLAB Data File
	Br_3776_ALT_2_1_Strain.mat	MATLAB Data File
	Br_3776_ALT_3_1_Strain.mat	MATLAB Data File
	Br_3776_BTB_1_1_Strain.mat	MATLAB Data File
	Br_3776_BTB_1_2_Strain.mat	MATLAB Data File
	Br_3776_BTB_2_1_Strain.mat	MATLAB Data File
	Br_3776_BTB_3_1_Strain.mat	MATLAB Data File
	Br_3776_MAX_1_1_Strain.mat	MATLAB Data File
	Br_3776_MAX_1_2_Strain.mat	MATLAB Data File
	Br_3776_MAX_2_1_Strain.mat	MATLAB Data File
	Br_3776_MAX_3_1_Strain.mat	MATLAB Data File
	Br_3776_SBS_1_1_Strain.mat	MATLAB Data File
	Br_3776_SBS_1_2_Strain.mat	MATLAB Data File
	Br_3776_SBS_2_1_Strain.mat	MATLAB Data File
	Br_3776_SBS_3_1_Strain.mat	MATLAB Data File

A.4.2 Instrumentation

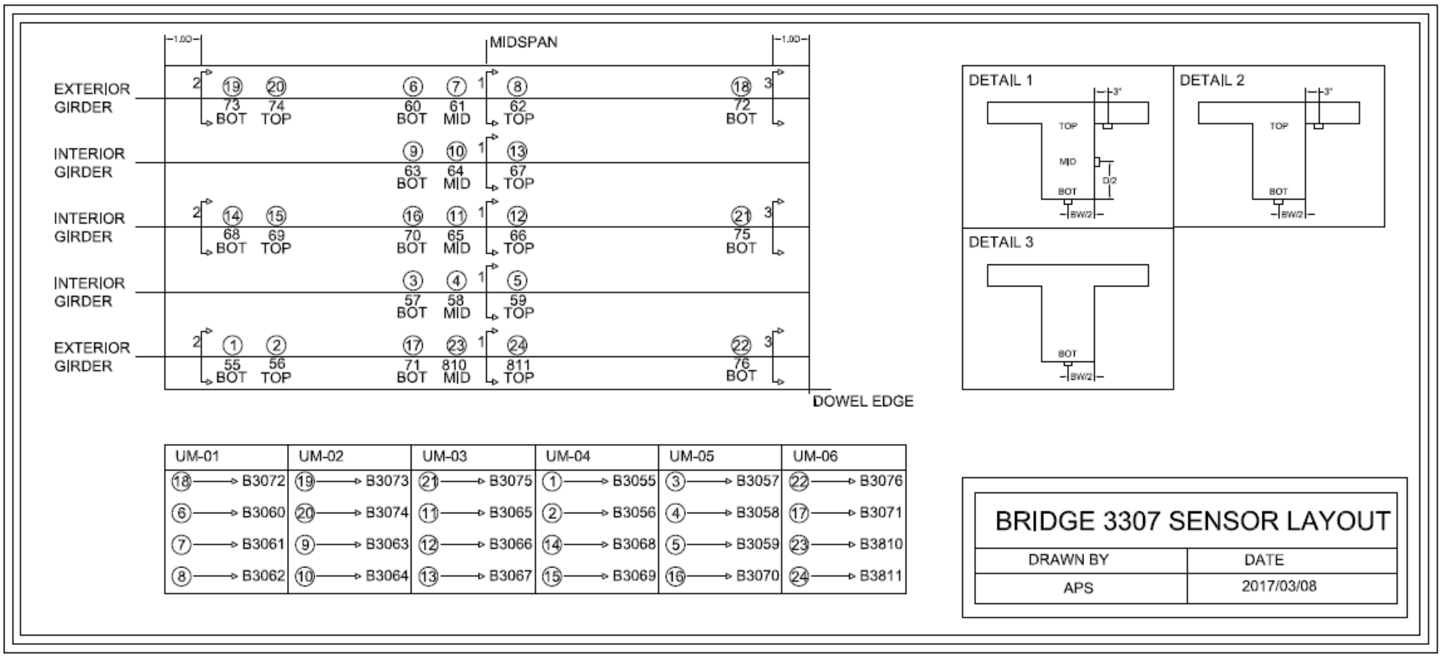

Figure 43: Jackson No. 3776 sensor layout

A.4.3 Loading

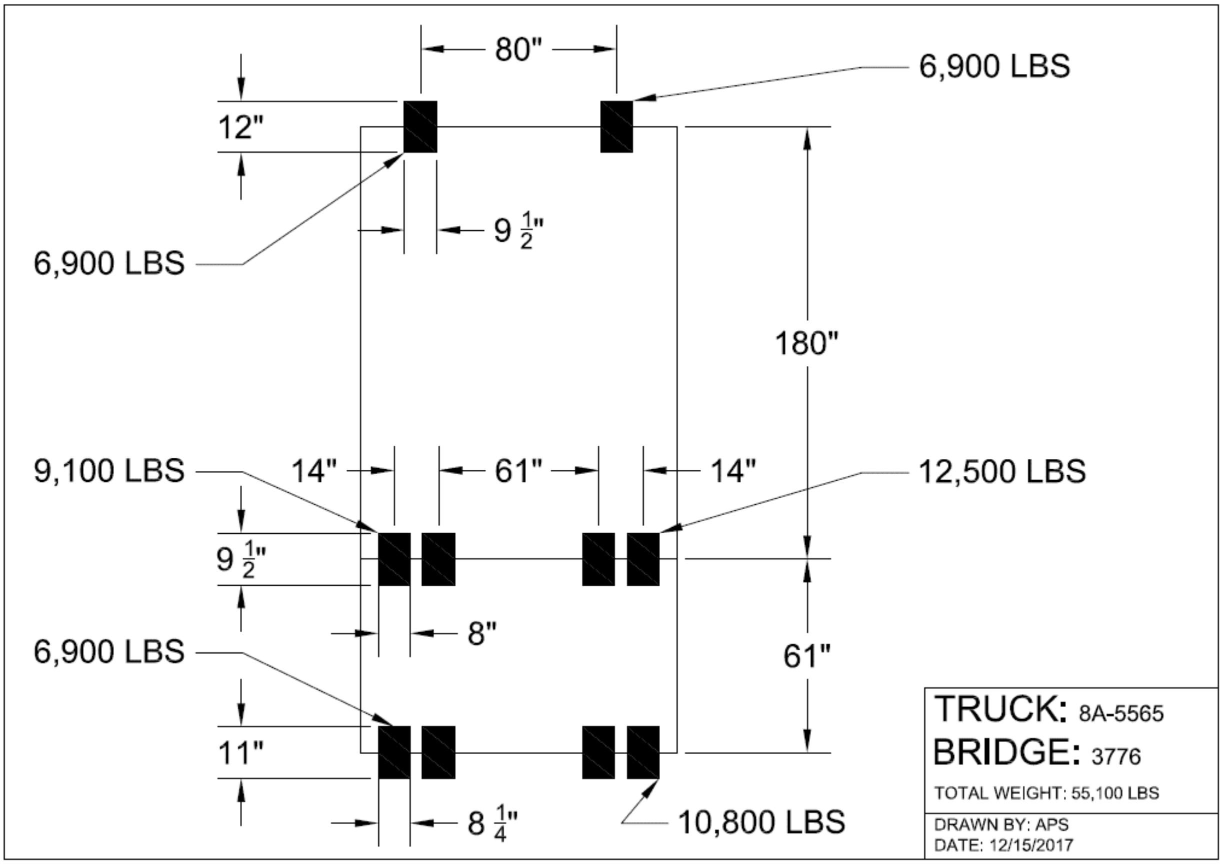

Figure 44: Jackson No. 3776 Truck 8A-5565 loading

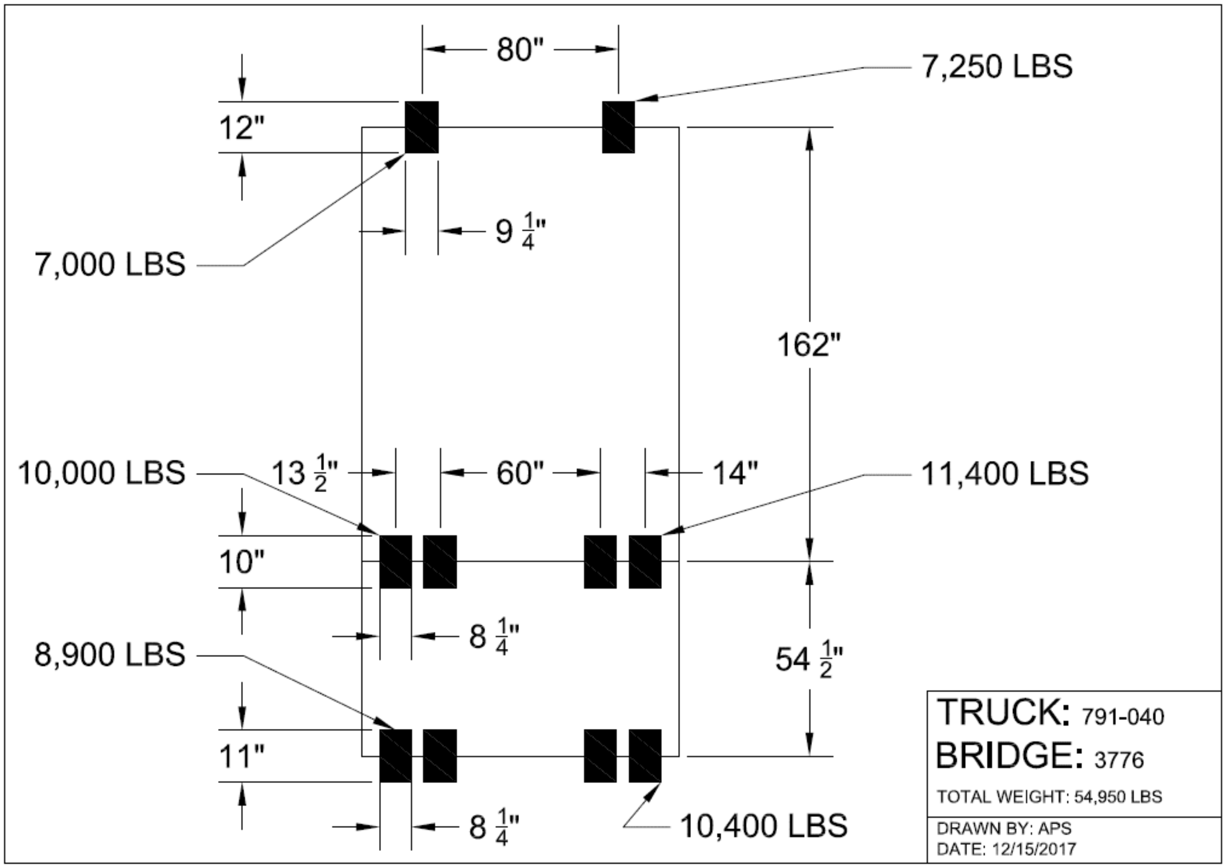

Figure 45: Jackson No. 3776 Truck 791-040 loading

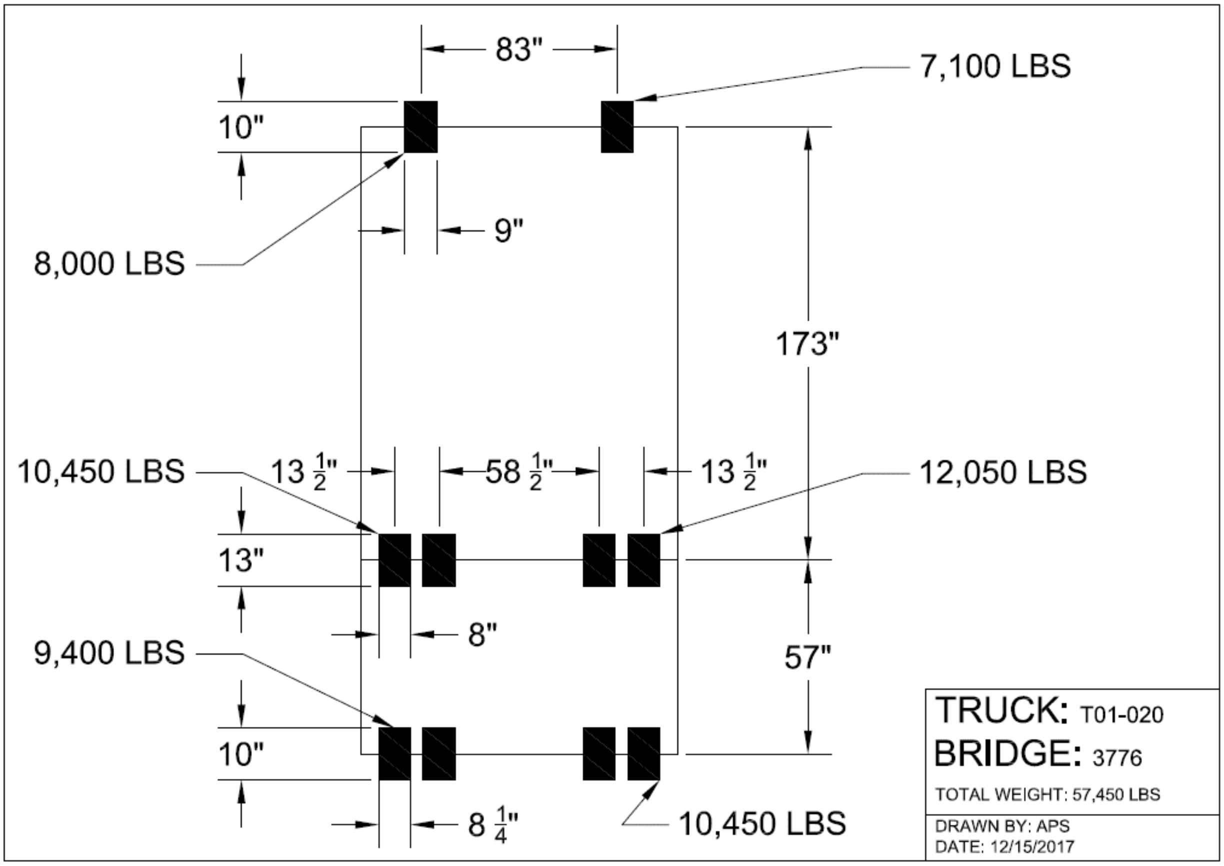

Figure 46: Jackson No. 3776 Truck T01-020 loading

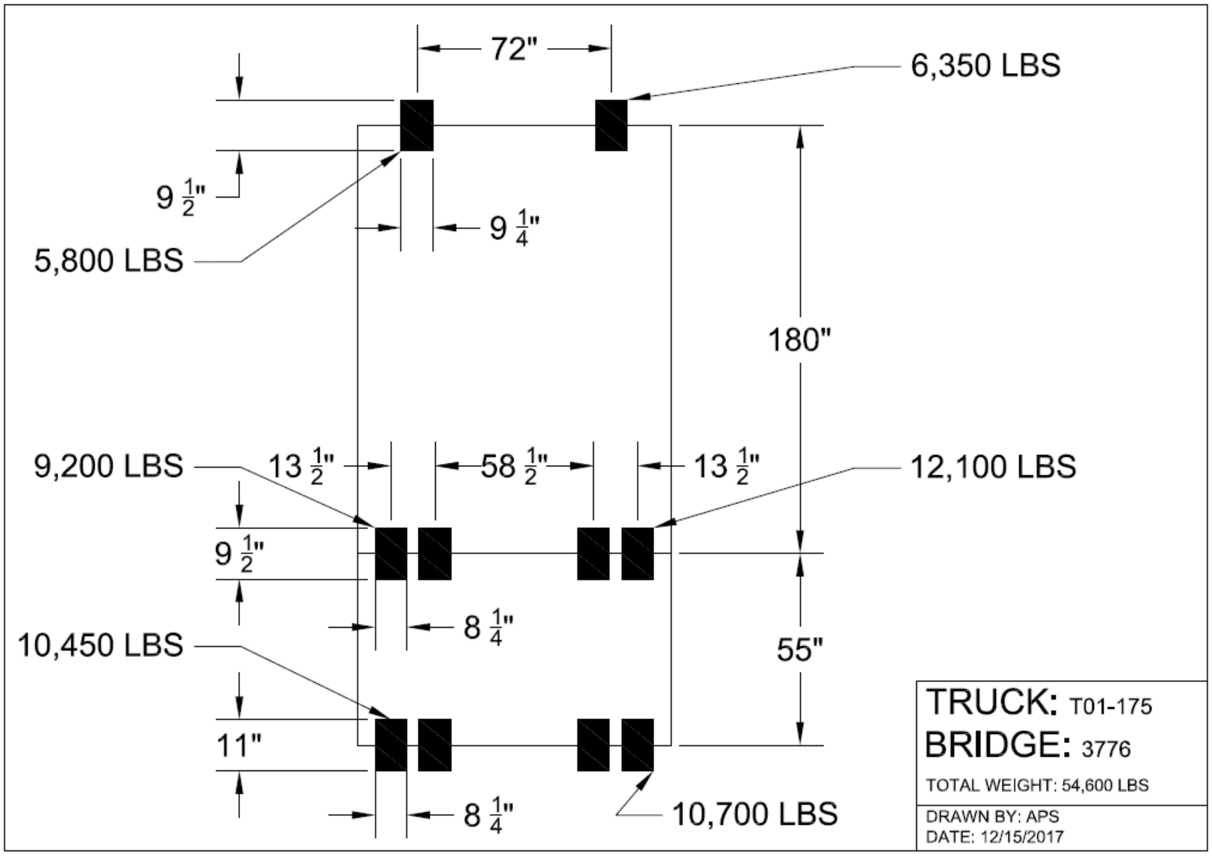

Figure 47: Jackson No. 3776 Truck T01-175 loading

A.4.4 Representative Data Plots

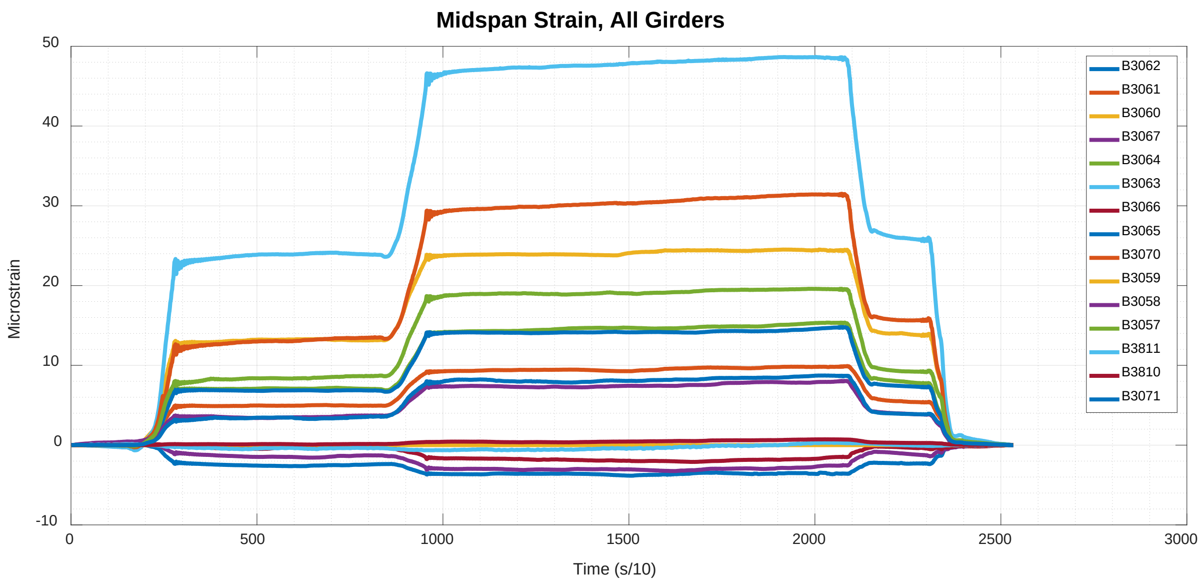

Figure 48: Jackson No. 3776 – BTB_2_1 strains - Midspan

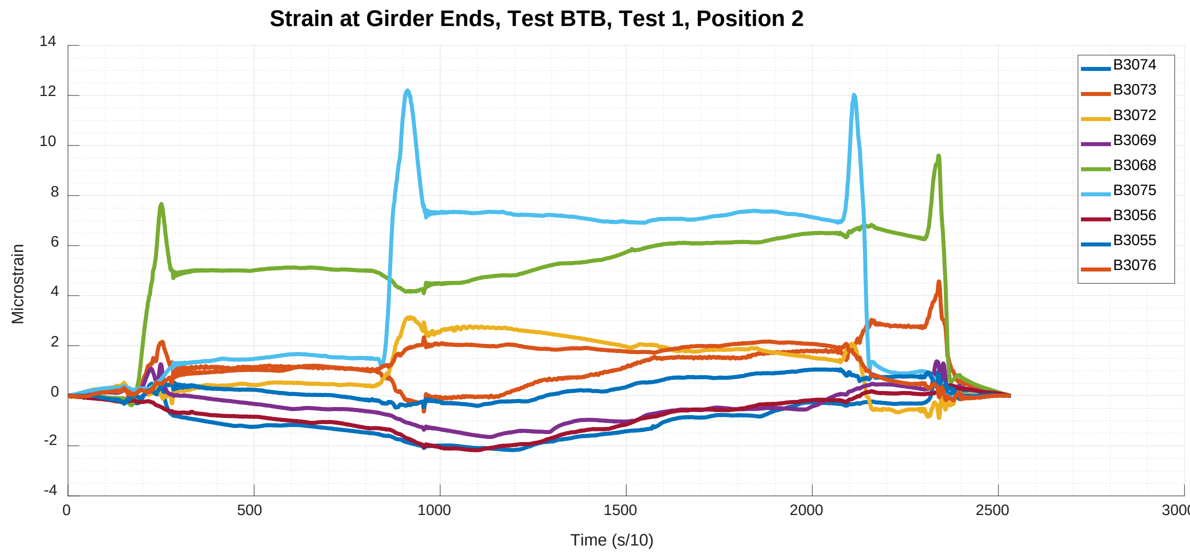

Figure 49: Jackson No. 3776 – BTB_2_1 strains - Ends

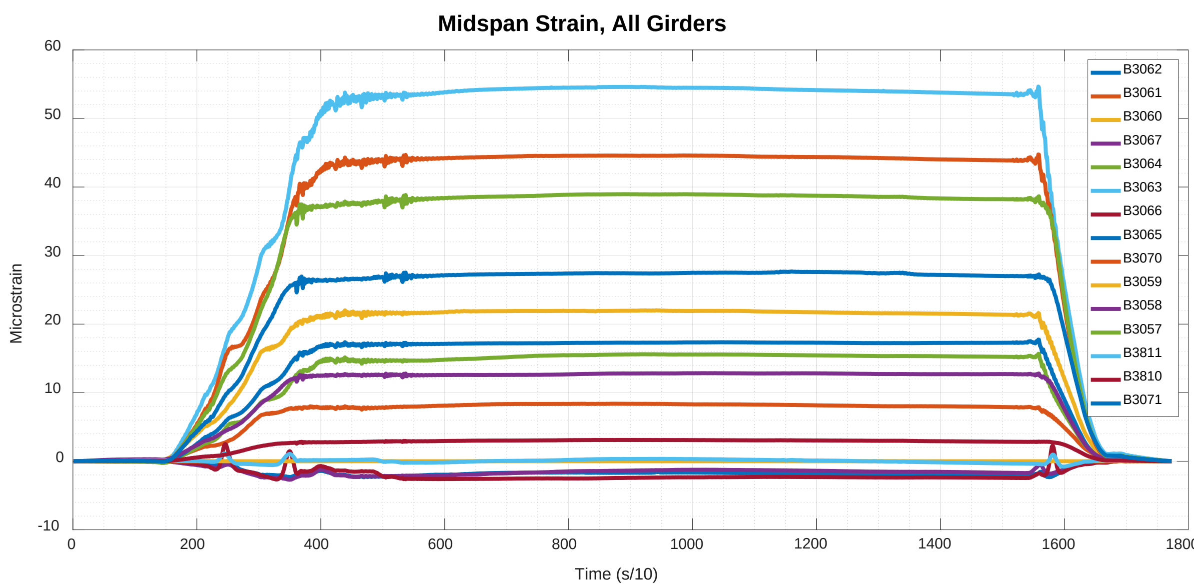

Figure 50: Jackson No. 3776 – SBS_2_1 strains - Midspan

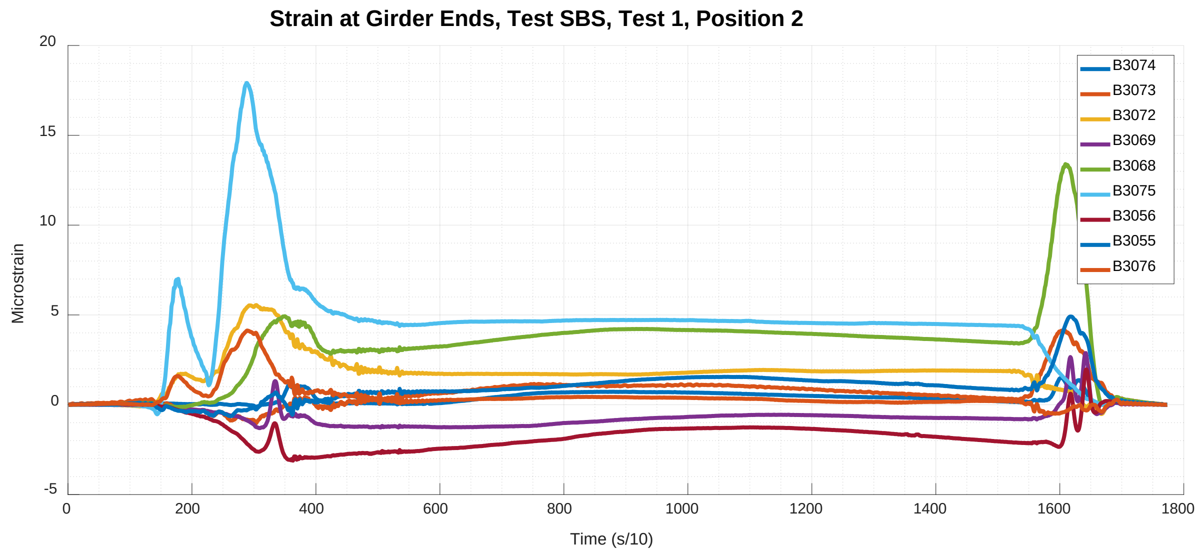

Figure 51: Jackson No. 3776 – SBS_2_1 strains – Ends

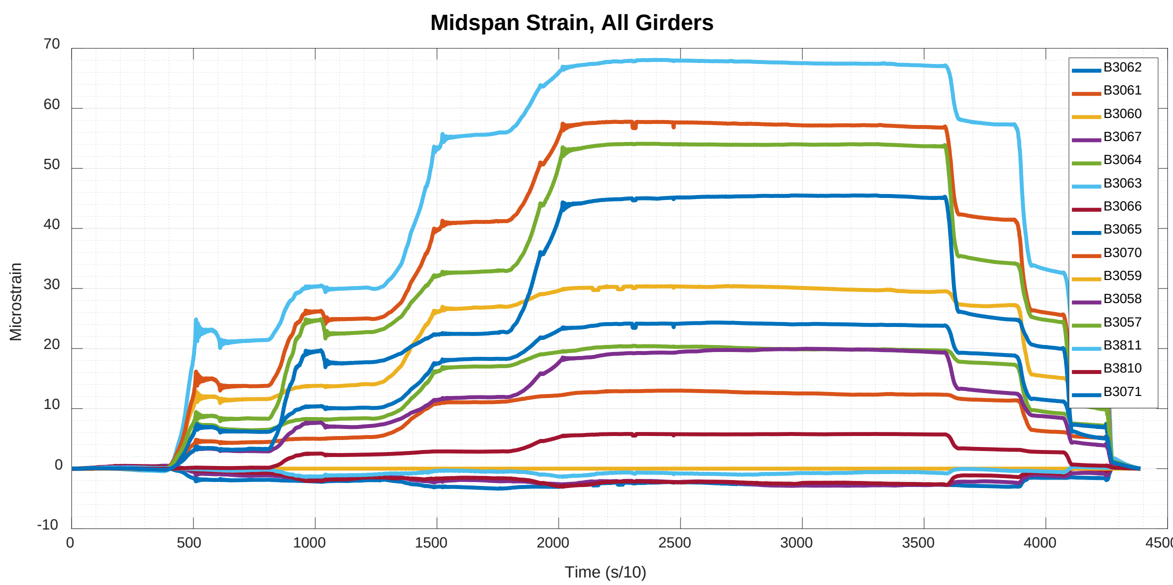

Figure 52: Jackson No. 3776 – MAX_2_1 strains - Midspan

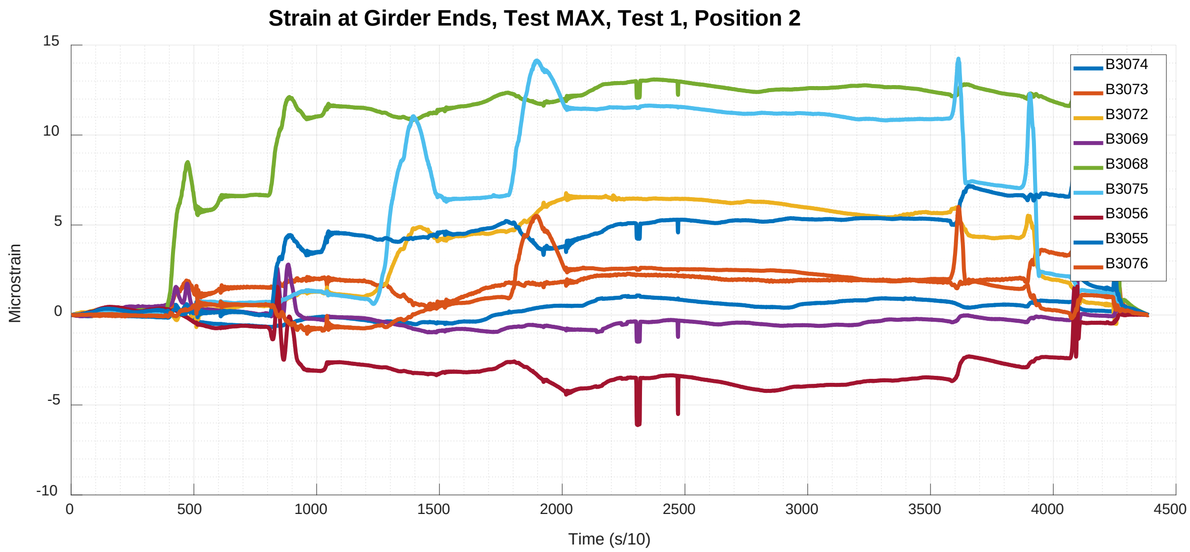

Figure 53: Jackson No. 3776 – MAX_2_1 strains - Ends

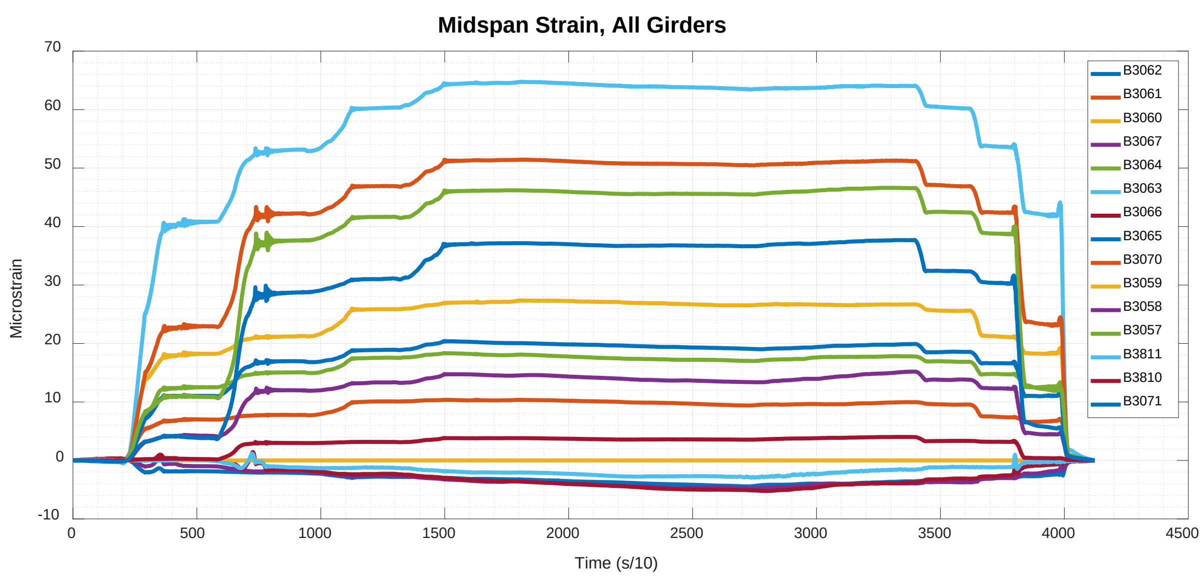

Figure 54: Jackson No. 3776 – ALT_2_1 strains - Midspan

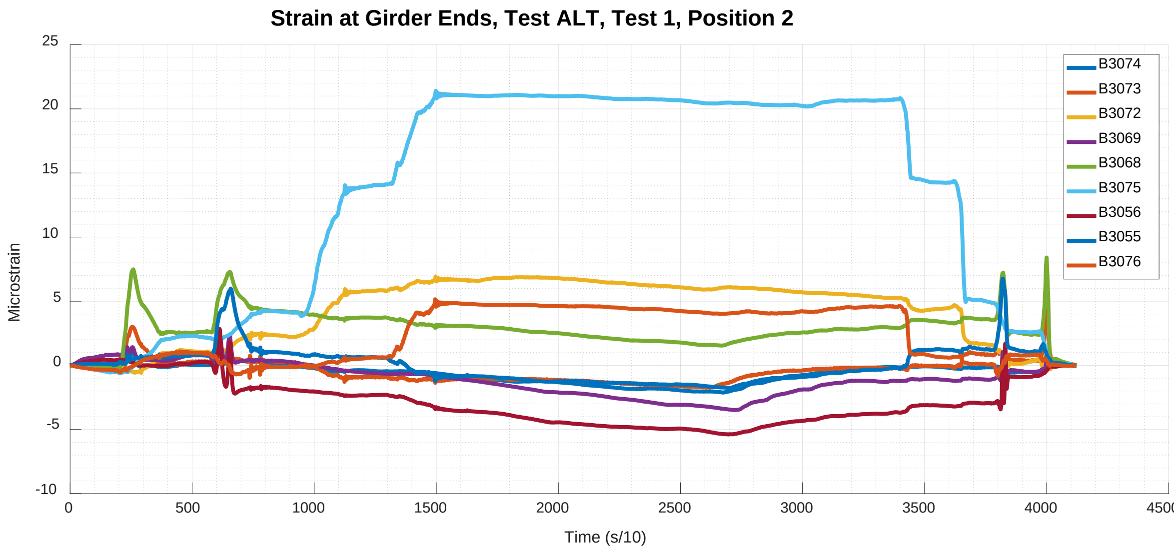

Figure 55: Jackson No. 3776 – ALT_2_1 strains - Ends

A.4.5 Rating Factor Calculations

Figure 56: Jackson No. 3776 Calculations

AASHTO Rating Calculations:

Bridge 3776 - Jackson, Maine

Material Parameters:

Concrete Compressive Strength $f'_c := 5\ \textbf{ksi}$

Reinforcement Yield Strength $F_y := 33\ \textbf{ksi}$

Unit Weights: Reinforced Concrete, Wearing Surface $\gamma_{RC} := 0.150\ \dfrac{\textbf{kip}}{\textbf{ft}^3}$ $\gamma_{ws} := 0.144\ \dfrac{\textbf{kip}}{\textbf{ft}^3}$

Concrte Elastic Modulus - LRFD Design C5.4.2.4-1

$$E_c := 1820\ \textbf{ksi} \cdot \sqrt{\frac{f'_c}{\textbf{ksi}}} = \left(4.07 \cdot 10^3\right)\ \textbf{ksi}$$

Geometric Properties:

Span Length $L := 32.96\ \textbf{ft}$

Girder Spacing - Interior and Exterior $S := 68.625\ \textbf{in}$ $S_x := 47.063\ \textbf{in}$

Number of Girders $NG := 5$

Skew Angle $skew := 0\ °$

Width and Number of Lanes $lanewidth := 11\ \textbf{ft}$ $Nlane := 2$

Wearing Surface Thickness $ws := 4\ \textbf{in}$

Girder Height - Interior, Exterior $h := 30.5\ \textbf{in}$ $h_x := 30.5\ \textbf{in}$

Deck Thickness $d_s := 5.5\ \textbf{in}$

Web Height - Interior, Exterior $d_g := h - d_s$ $d_{gx} := h_x - d_s$

Web Thickness - Interior, Exterior $b_w := 19.5\ \textbf{in}$ $b_{wx} := 19.5\ \textbf{in}$

Distance from Centerline of Girder to Edge of Curb $d_e := -15.75\ \textbf{in}$

Height of Curb Above Deck $h_{curb} := 12\ \textbf{in}$

Width of Curb $b_{curb} := 18\ \textbf{in}$

Height to Centroid of Reinforcement - Interior, Exterior

$$y_{bar} := \begin{bmatrix} 3.063 \\ 4.337 \\ 5.049 \\ 4.337 \\ 3.063 \end{bmatrix} in \qquad y_{barx} := \begin{bmatrix} 3.063 \\ 4.337 \\ 5.049 \\ 4.337 \\ 3.063 \end{bmatrix} in$$

Dictance Between Centroids of Deck and Web

$$e_g := \frac{d_g + d_s}{2} = 15.25\ in$$

Depth of Reinforcement - Interior, Exterior

$$d := h - y_{bar} \qquad d_x := h_x - y_{barx} + h_{curb}$$

Area of Reinforcement at All Changes - Interior, Exterior

$$A_s := \begin{bmatrix} 5.0625 \\ 7.0625 \\ 9.0625 \\ 7.0625 \\ 5.0625 \end{bmatrix} in\ A_{sx} := \begin{bmatrix} 5.0625 \\ 7.0625 \\ 9.0625 \\ 7.0625 \\ 5.0625 \end{bmatrix} in^2$$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder $P_{dint} := 0\ kip$

Resistance Factors - Structural Dead load, Weading Surface, Live Load

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.25 \quad \gamma_{LL} := 1.35$$

Impact Factor $IM := 0.33$

Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 1.0$

Distributed Load Due to Rail $w_{rail} := 0.109\ \frac{kip}{ft}$

Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway $exc := 0\ in$

$$P_{dext} := \frac{P_{dint}}{2}$$

Distribution Factors

Area and Moment of Inertia of Web $A := d_g \cdot b_w = 487.5\ in^2$ $I := \frac{b_w \cdot d_g^{\ 3}}{12} = \left(2.539 \cdot 10^4\right)\ in^4$

Modular Ratio - Deck and Web $n := 1$

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(1.388 \cdot 10^5\right)\ in^4$

Interior Moment Distribution Factor

$$g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.505$$

$$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.635$$

$$g_m := \max\left(g_{m1}, g_{m2}\right)$$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From Center of Gravity of Girders $e_1 := \frac{W_r}{2} - 5\ ft + exc = 6\ ft$

Eccentricity of Exterior Girder From Center of Gravity of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 11.438\ ft$

Eccentricity of Each Girder

$$x_1 := X_{ext}$$
$$x_2 := X_{ext} - S$$
$$x_3 := X_{ext} - 2 \cdot S$$

$$x_4 := \text{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ ft\right)$$
$$x_5 := \text{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ ft\right)$$
$$x_6 := \text{if}\left(NG > 5, X_{ext} - 5 \cdot S, 0\ ft\right)$$
$$x_7 := \text{if}\left(NG > 6, X_{ext} - 6 \cdot S, 0\ ft\right)$$

Lever Rule - One Design Lane

$$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2} + x_6^{\ 2} + x_7^{\ 2}} = 0.41$$

$$g_{mR1} := \text{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0$$

Lever Rule - Two Design Lanes

$$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot (e_1 - 5\ ft)}{x_1^2 + x_2^2 + x_3^2 + x_4^2 + x_5^2 + x_6^2 + x_7^2} = 0.435$$

$$g_{mR2} := \mathbf{if}\,(P_{dint} > 0, R_2, 0) = 0$$

Exterior Moment Distribution Factor

$$g_{mx1} := \frac{1.2\ (S + d_e - 2\ ft)}{2 \cdot S} = 0.252$$

$$ee := 0.77 + \frac{d_e}{9\ ft} = 0.624$$

$$g_{mx2} := g_{m2} \cdot ee = 0.396$$

$$g_{mx} := \max\,(g_{mx1}, g_{mx2})$$

Interior	Exterior
$g_m = 0.635$	$g_{mx} = 0.396$

Loading

Interior Girder Dead Load

$$w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.508\ \frac{kip}{ft}$$

Deck Dead Load

$$w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.393\ \frac{kip}{ft}$$

Curb Dead Load

$$w_{curb} := \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.225\ \frac{kip}{ft}$$

Dead Load from Nonstructural Components

$$w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.154\ \frac{kip}{ft}$$

Total Dead Load

$$DC := w_{girder} + w_{deck} + w_{ns} = 1.055\ \frac{kip}{ft}$$

Exterior Girder Dead Load

$$w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.508\ \frac{kip}{ft}$$

Exterior Deck Dead Load

$$w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.27\ \frac{kip}{ft}$$

Total Exterior Dead Load $DC_x := w_{girderx} + w_{deckx} + w_{ns} = 0.931\ \frac{kip}{ft}$

Wearing Surface Dead Load $DW := \gamma_{ws} \cdot ws \cdot S = 0.275\ \frac{kip}{ft}$

Exterior Wearing Surface Dead Load $DW_x := \gamma_{ws} \cdot ws \cdot S_x = 0.188\ \frac{kip}{ft}$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} = 143.261\ ft \cdot kip \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} = 126.486\ ft \cdot kip$$

$$M_{DW} := \frac{DW \cdot L^2}{8} = 37.276\ ft \cdot kip \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8} = 25.564\ ft \cdot kip$$

Live Load Moment - Truck Load

$$M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 313.28\ ft \cdot kip$$

Live Load Moment - Tandem

$$M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 362\ ft \cdot kip$$

Live Load Moment - Lane

$$M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 86.909\ ft \cdot kip$$

Total HL-93 Live Load

$$M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right) = 568.369\ ft \cdot kip$$

Nominal Resistance

Depth of Interior Whitney Stress Block

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 0.573 \\ 0.799 \\ 1.025 \\ 0.799 \\ 0.573 \end{bmatrix} \boldsymbol{in}$$

Depth of Exterior Whitney Stress Block

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 0.835 \\ 1.165 \\ 1.495 \\ 1.165 \\ 0.835 \end{bmatrix} \boldsymbol{in}$$

Nominal Interior Moment Resistance

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 377.987 \\ 500.375 \\ 621.509 \\ 500.375 \\ 377.987 \end{bmatrix} \boldsymbol{ft \cdot kip}$$

Midspan Moment Capacity

$$M_{capacity} := \max\left(M_n\right) = 621.509 \ \boldsymbol{ft \cdot kip}$$

Nominal Exterior Moment Resistance

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 543.223 \\ 729.882 \\ 914.718 \\ 729.882 \\ 543.223 \end{bmatrix} (\boldsymbol{ft \cdot kip})$$

Midspan Moment Capacity

$$M_{capacityx} := \max\left(M_{nx}\right) = 914.718 \ \boldsymbol{ft \cdot kip}$$

Rating Factors

$$RF_{Interior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacity} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m}$$

$$RF_{Exterior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx} - \gamma_{DC} \cdot M_{DCx} - \gamma_{DW} \cdot M_{DWx}}{\gamma_{LL} \cdot M_{LL} \cdot g_{mx}}$$

<u>Interior</u>

$RF_{Interior} = 0.685$

<u>Exterior</u>

$RF_{Exterior} = 2.083$

Rating Factor Improvements

Interior

Maximum Recorded Strain $\varepsilon_T := 57.69 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 397.94\ ft \cdot kip$

Uncracked Section Modulus $S_{unc} := 5171\ in^3$

Cracked Section Modulus $S_{cr} := 1651\ in^3$

Section Behavior $Behavior := \text{"Uncracked"}$

Section Modulus Effective for Behavior

$$S_e := \text{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(5.171 \cdot 10^3\right)\ in^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.44 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 1.497$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.7$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.748$

Improved Rating Factor $RF_{Improved} := RF_{Interior} \cdot k = 1.198$

Exterior

Maximum Recorded Strain $\varepsilon_T := 48.51 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 397.94\ \boldsymbol{ft} \cdot \boldsymbol{kip}$

Uncracked Section Modulus $S_{unc} := 7324\ \boldsymbol{in}^3$

Cracked Section Modulus $S_{cr} := 2406\ \boldsymbol{in}^3$

Section Behavior $Behavior :=$ "Uncracked"

Section Modulus Effective for Behavior

$$S_e := \textbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(7.324 \cdot 10^3\right)\ \boldsymbol{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 6.348 \cdot 10^{-5}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.309$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.7$

Test Understanding Factor $k_b := \textbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.154$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 2.404$

Summary

Interior	Exterior
$RF_{Improved} = 1.198$	$RF_{ImprovedExt} = 2.404$

Bridge 3776

Interior Section Properties

$y_{unc} := 17.8\ in$ $\quad I_{unc} := 92135\ in^4$ $\quad S_{unc} := 5171\ in^3$ $\quad N_i := 3$

$y_{cr} := 23.4\ in$ $\quad I_{cr} := 38636\ in^4$ $\quad S_{cr} := 1651\ in^3$

Exterior Section Properties

$y_{uncx} := 20.5\ in$ $\quad I_{uncx} := 150368\ in^4$ $\quad S_{uncx} := 7324\ in^3$ $\quad N_x := 2$

$y_{crx} := 27.6\ in$ $\quad I_{crx} := 66328\ in^4$ $\quad S_{crx} := 2406\ in^3$

Test Parameters

Maximum Applied Moment

$M := 795.88\ ft \cdot kip$

Maximum Measured Strain

$\varepsilon_m := 74.85 \cdot 10^{-6}$

Using Nominal Compressive Strength - 2.5 ksi

$f'_c := 2.5\ ksi$ $\quad E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(2.878 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 1.1 \cdot 10^{-4}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 3.399 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Using More Realistic Compressive Strength - 5 ksi

$f'_c := 5\ ksi$ $\quad E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 7.781 \cdot 10^{-5}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.403 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Distributiuon Factors from Testing

Bridge 2130

Compressive Strength

$$f'_c := 5\ ksi$$

Section Modulus for Each Girder

$$S := \begin{bmatrix} 7324 \\ 5171 \\ 5171 \\ 5171 \\ 7324 \end{bmatrix} in^3$$

Strains Recorded During SBS & MAX Tests

$$\varepsilon_2 := \begin{bmatrix} 21.95 \\ 54.6 \\ 44.57 \\ 38.95 \\ 27.41 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 30.27 \\ 67.69 \\ 57.69 \\ 53.94 \\ 44.79 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Moment Produced in Each Girder

$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 54.52 \\ 95.751 \\ 78.161 \\ 68.306 \\ 68.082 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 75.186 \\ 118.706 \\ 101.17 \\ 94.593 \\ 111.251 \end{bmatrix} ft \cdot kip$$

Total Moment

$$M_{sum2} := \sum M_2 = 364.82\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 500.906\ ft \cdot kip$$

Distribution Factors

$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.299 \\ 0.525 \\ 0.428 \\ 0.374 \\ 0.373 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.3 \\ 0.474 \\ 0.404 \\ 0.378 \\ 0.444 \end{bmatrix}$$

Distributiuon Factors from Testing
Bridge 3776

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder $S := \begin{bmatrix} 7324 \\ 5171 \\ 5171 \\ 5171 \\ 7324 \end{bmatrix} in^3$

Strains Recorded During SBS & MAX Tests

$\varepsilon_2 := \begin{bmatrix} 21.95 \\ 54.6 \\ 44.57 \\ 38.95 \\ 27.41 \end{bmatrix} \cdot 10^{-6}$ $\varepsilon_4 := \begin{bmatrix} 30.27 \\ 67.69 \\ 57.69 \\ 53.94 \\ 44.79 \end{bmatrix} \cdot 10^{-6}$

Modulus of Elasticity $E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

Moment Produced in Each Girder

$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 54.52 \\ 95.751 \\ 78.161 \\ 68.306 \\ 68.082 \end{bmatrix} ft \cdot kip$ $M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 75.186 \\ 118.706 \\ 101.17 \\ 94.593 \\ 111.251 \end{bmatrix} ft \cdot kip$

Total Moment

$M_{sum2} := \sum M_2 = 364.82\ ft \cdot kip$ $M_{sum4} := \sum M_4 = 500.906\ ft \cdot kip$

Distribution Factors

$DF_2 := \frac{2 \cdot M_2}{M_{sum2}}$ $DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$

$DF_2 = \begin{bmatrix} 0.299 \\ 0.525 \\ 0.428 \\ 0.374 \\ 0.373 \end{bmatrix}$ $DF_4 = \begin{bmatrix} 0.3 \\ 0.474 \\ 0.404 \\ 0.378 \\ 0.444 \end{bmatrix}$

A.5 Alna No. 2130

A.5.1 Input Data, Experimental Configuration, and Experimental Data Collected

Table 20: Alna No. 2130 input data, experimental configuration, and experimental data collected

File Contents	***File Name***	***File Type***
Exterior Section Data	Br2130_Geom_Ext.csv	CSV Format
Interior Section Data	Br2130_Geom_Int.csv	CSV Format
Sensors	Br2130_Sensors.csv	CSV Format
Sensor Layout	Br2130_SensorLayout.csv	MATLAB Data File
Truck Weight and Dimensions	Br2130_Trucks.mat	MATLAB Data File
Sensor Data	Br_2130_ALT_1_1_Strain.mat	MATLAB Data File
	Br_2130_ALT_2_1_Strain.mat	MATLAB Data File
	Br_2130_ALT_3_1_Strain.mat	MATLAB Data File
	Br_2130_BTB_1_1_Strain.mat	MATLAB Data File
	Br_2130_BTB_1_2_Strain.mat	MATLAB Data File
	Br_2130_BTB_2_1_Strain.mat	MATLAB Data File
	Br_2130_BTB_3_1_Strain.mat	MATLAB Data File
	Br_2130_MAX_1_1_Strain.mat	MATLAB Data File
	Br_2130_MAX_1_2_Strain.mat	MATLAB Data File
	Br_2130_MAX_2_1_Strain.mat	MATLAB Data File
	Br_2130_MAX_3_1_Strain.mat	MATLAB Data File
	Br_2130_SBS_1_1_Strain.mat	MATLAB Data File
	Br_2130_SBS_1_2_Strain.mat	MATLAB Data File
	Br_2130_SBS_2_1_Strain.mat	MATLAB Data File
	Br_2130_SBS_3_1_Strain.mat	MATLAB Data File

A.5.2 Instrumentation

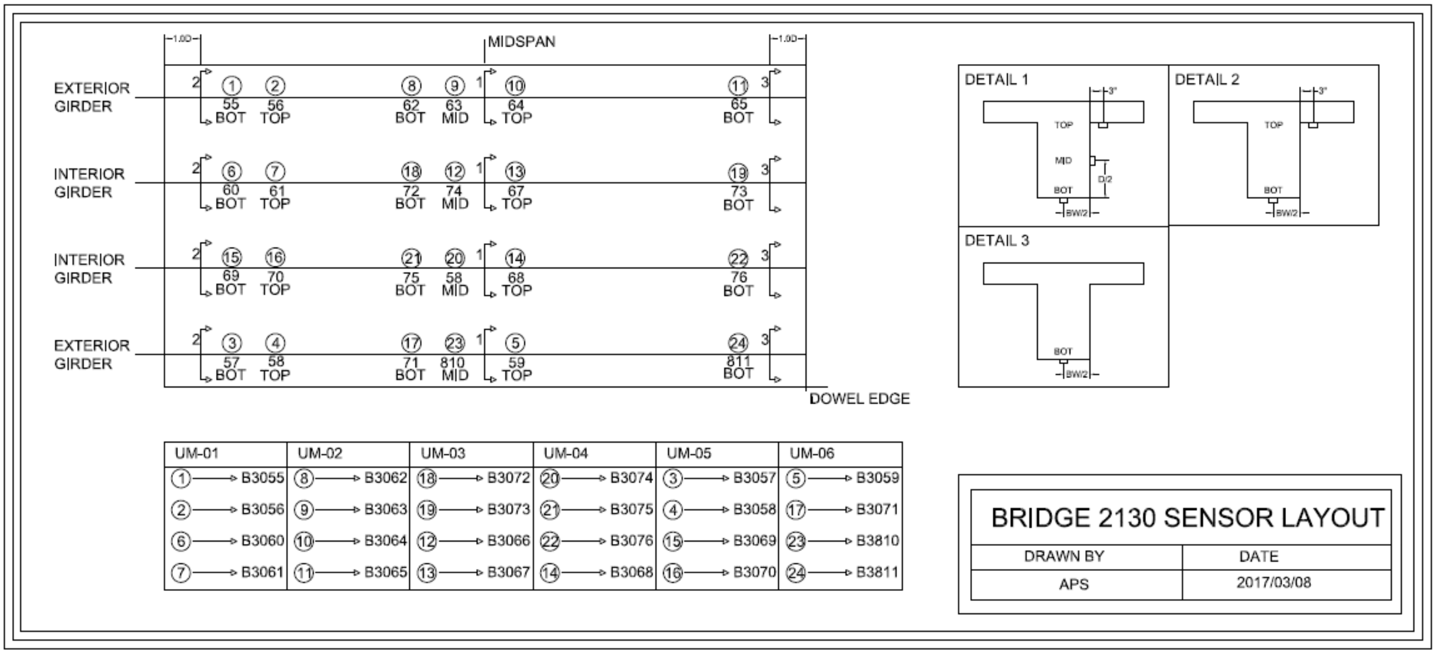

Figure 57: Alna No. 2130 sensor layout

A.5.3 Loading

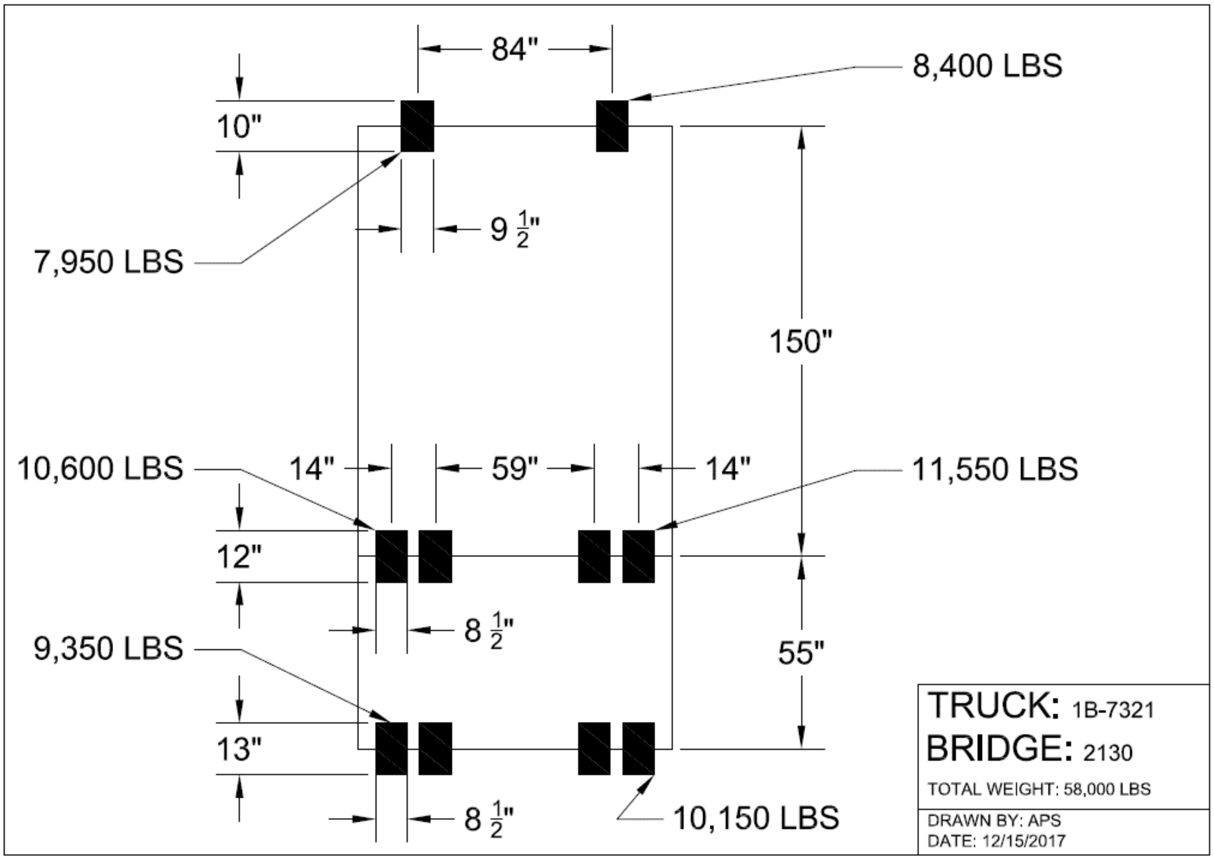

Figure 58: Alna No. 2130 Truck 1B-7321 loading

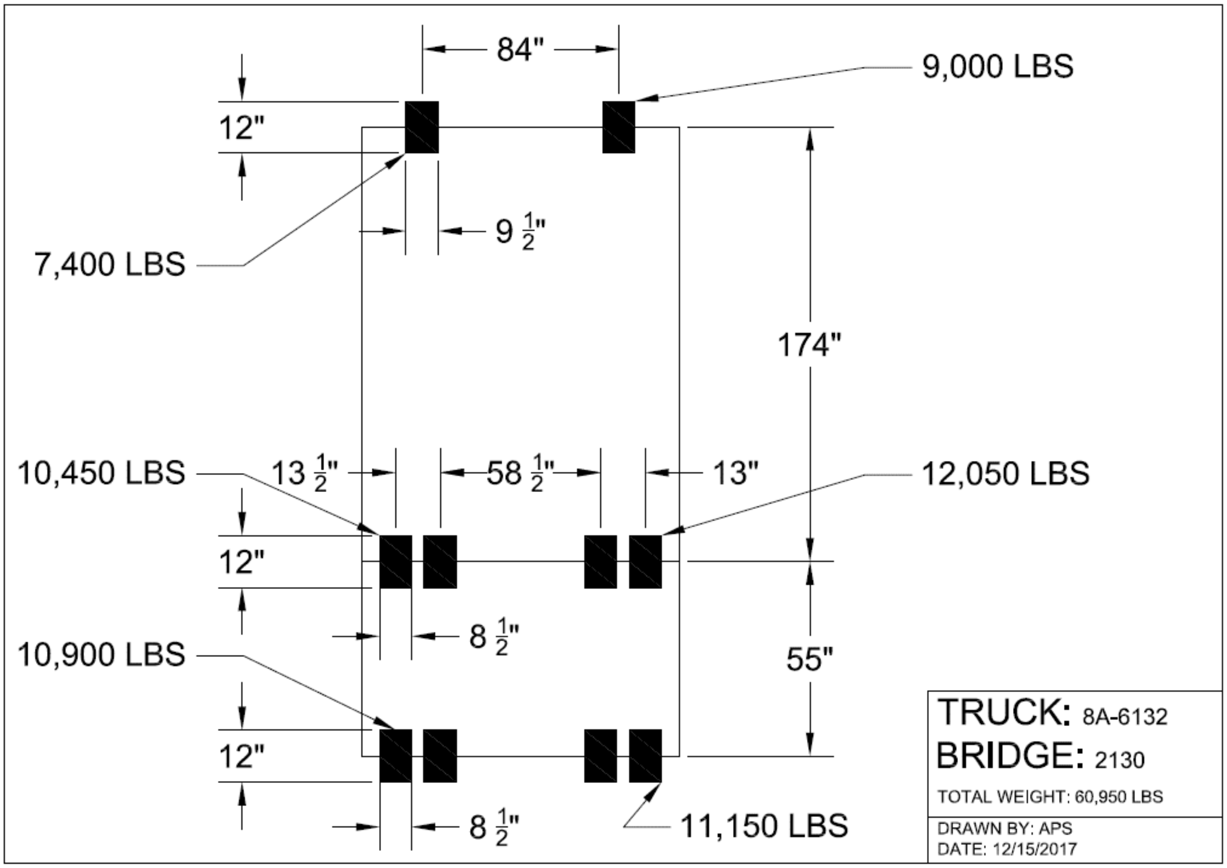

Figure 59: Alna No. 2130 Truck 8A-6132 loading

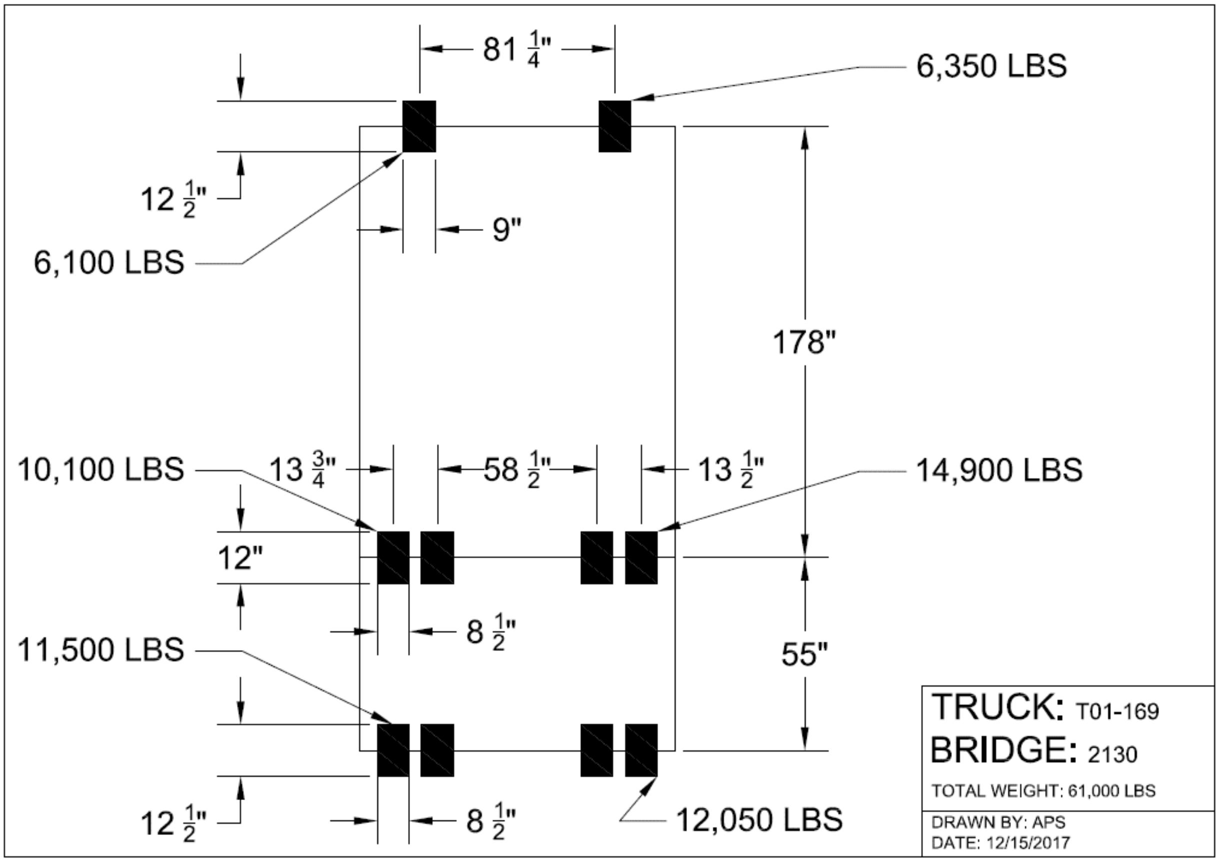

Figure 60: Alna No. 2130 Truck T01-169 loading

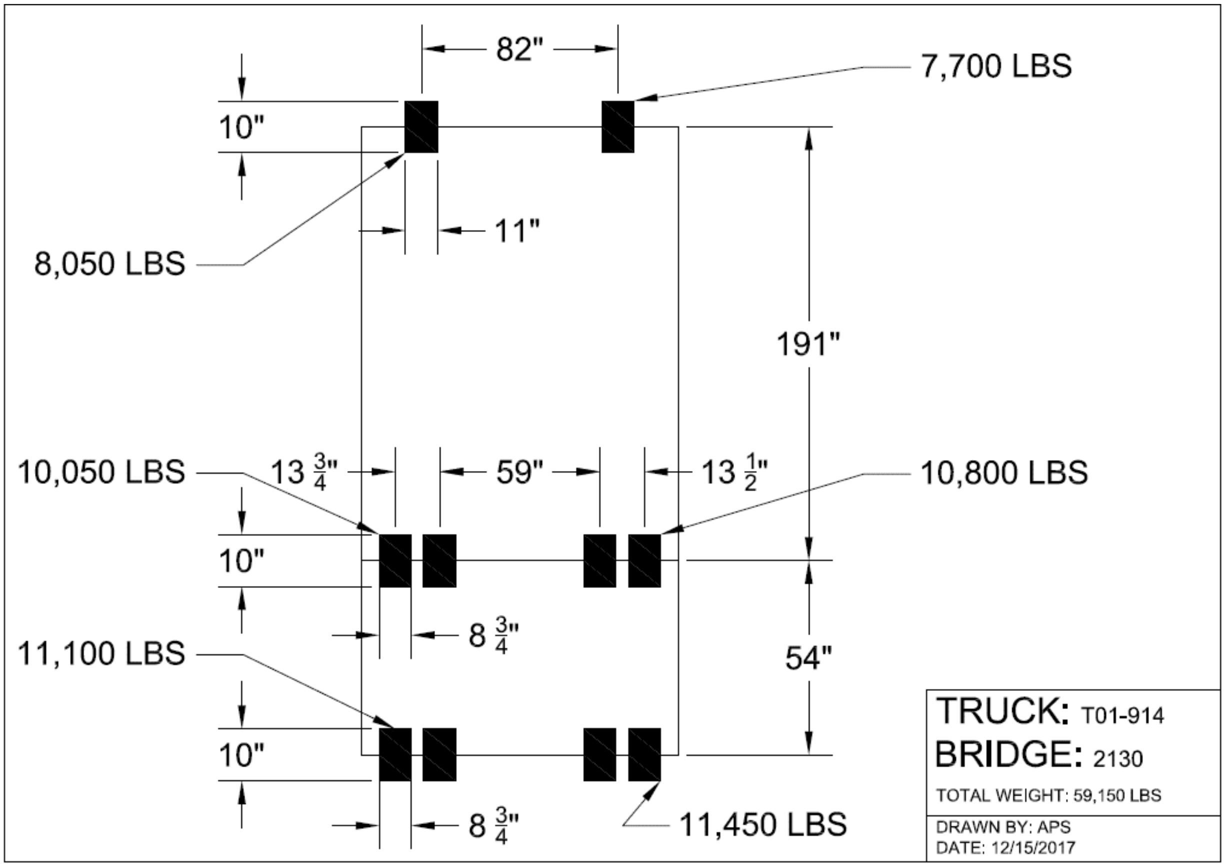

Figure 61: Alna No. 2130 Truck T01-914 loading

A.5.4 Representative Data Plots

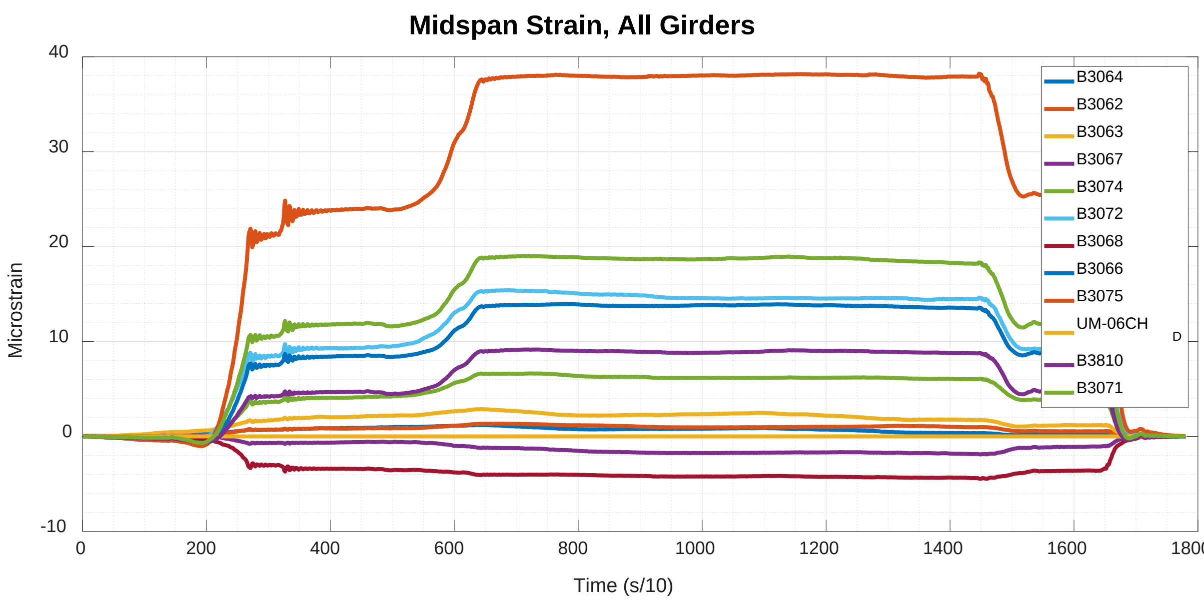

Figure 62: Alna No. 2130 – BTB_2_1 strains - Midspan

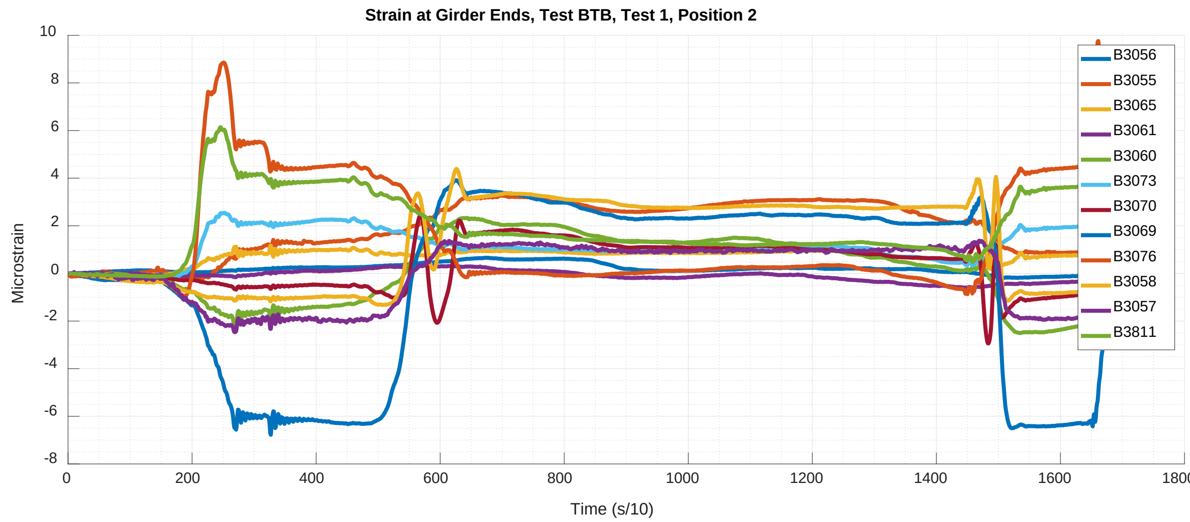

Figure 63: Alna No. 2130 – BTB_2_1 strains - Ends

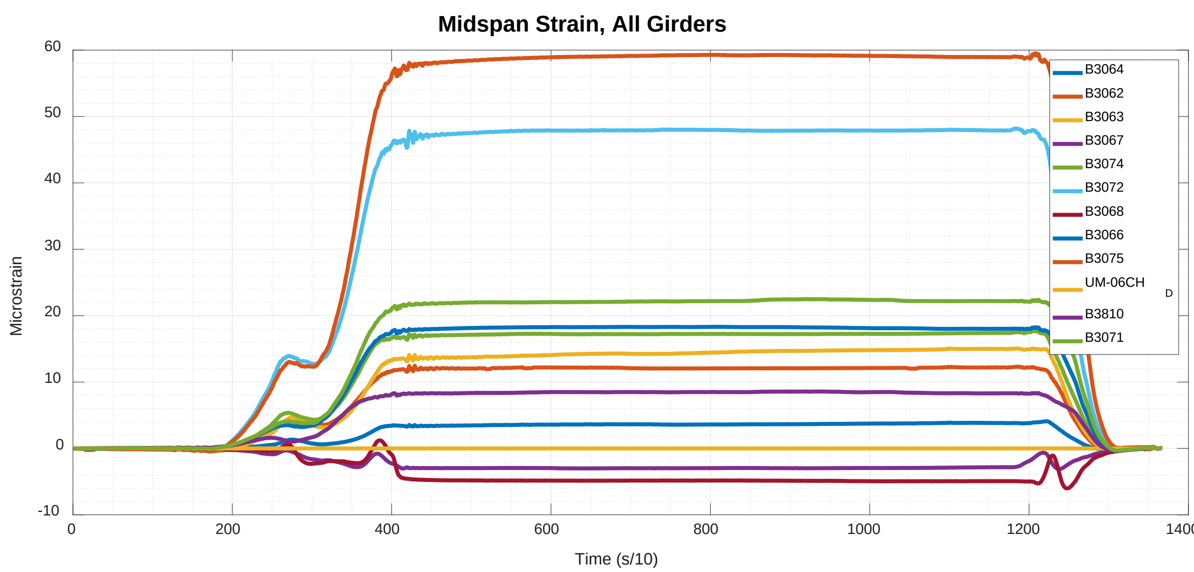

Figure 64: Alna No. 2130 – SBS_2_1 strains - Midspan

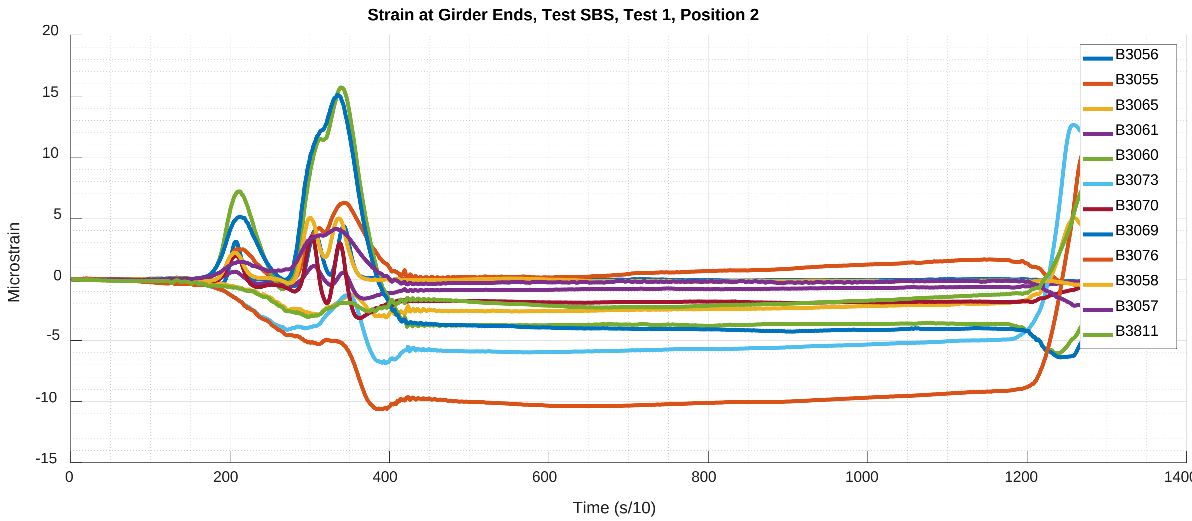

Figure 65: Alna No. 2130 – SBS_2_1 strains – Ends

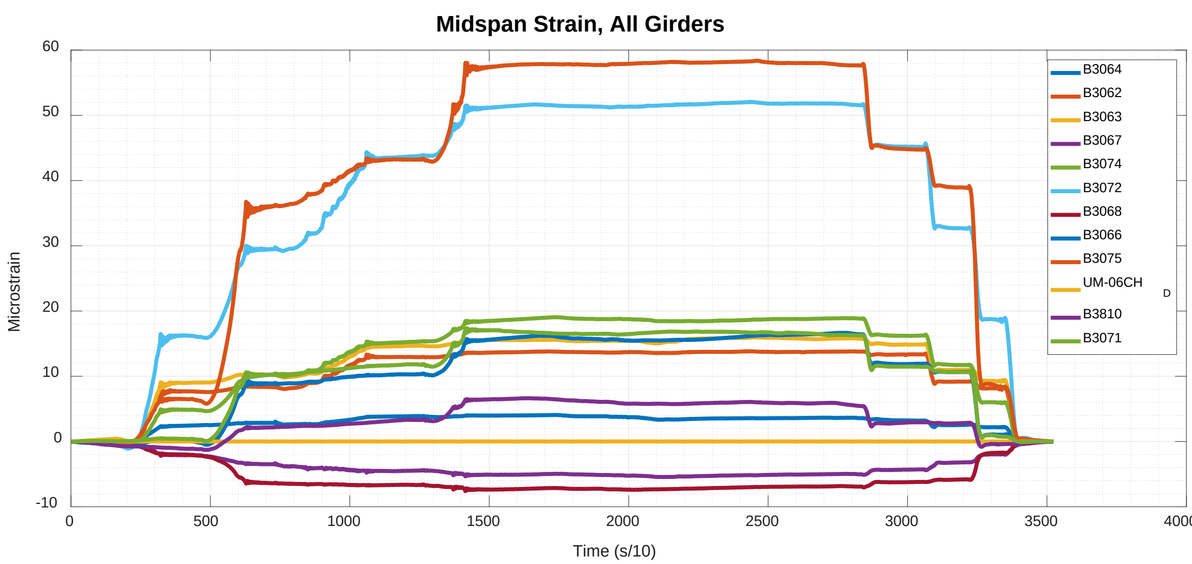

Figure 66: Alna No. 2130 – MAX_3_1 strains - Midspan

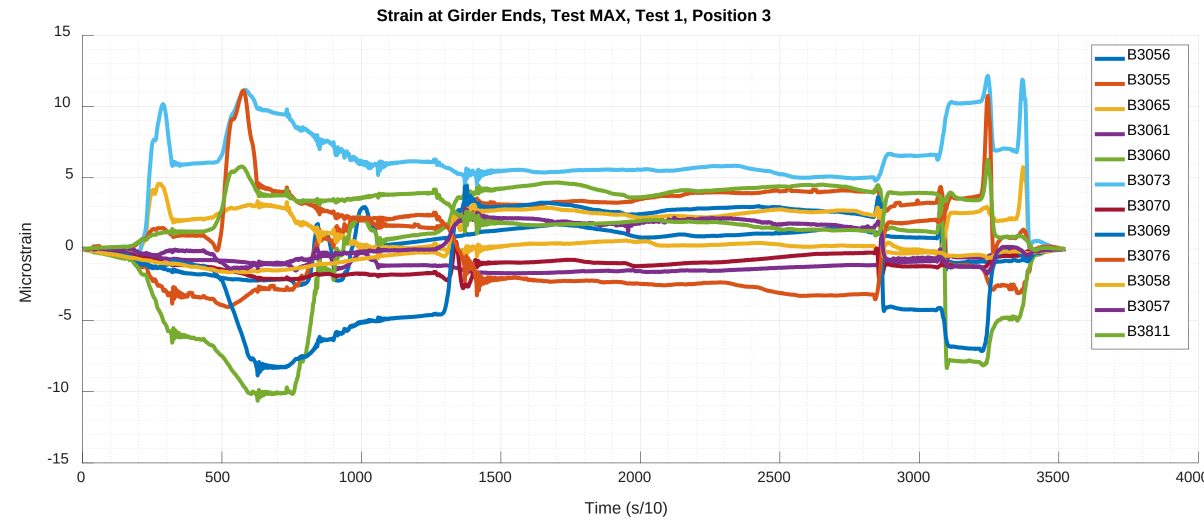

Figure 67: Alna No. 2130 – MAX_3_1 strains - Ends

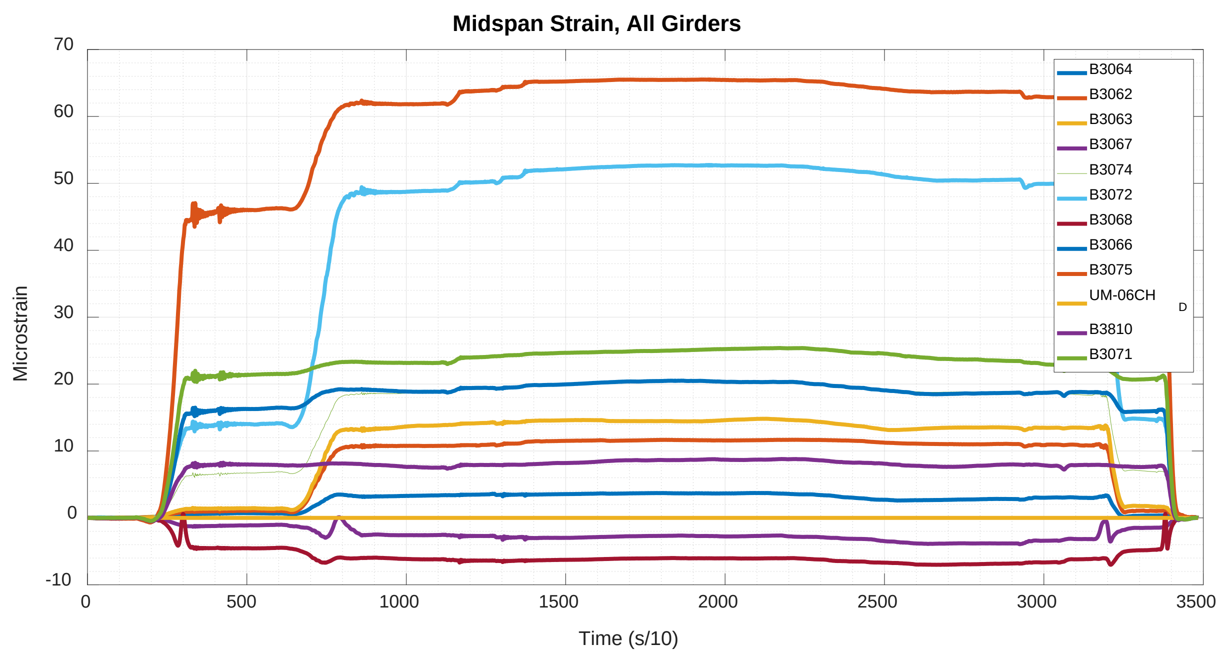

Figure 68: Alna No. 2130 – ALT_2_1 strains - Midspan

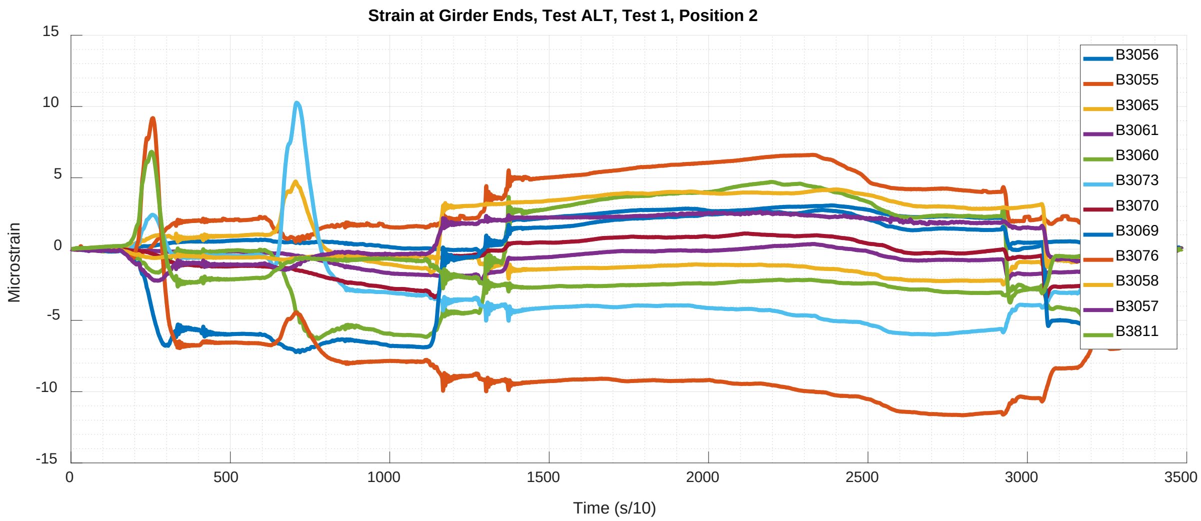

Figure 69: Alna No. 2130 – ALT_2_1 strains - Ends

A.5.5 Rating Factor Calculations

Figure 70: Alna No. 2130 Calculations

AASHTO Rating Calculations:

Bridge 2130 - Alna, Maine

Material Parameters:

Concrete Compressive Strength $f'_c := 5\ ksi$

Reinforcement Yield Strength $F_y := 33\ ksi$

Unit Weights: Reinforced Concrete, Wearing Surface $\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$ $\gamma_{ws} := 0.144\ \frac{kip}{ft^3}$

Concrte Elastic Modulus - LRFD Design C5.4.2.4-1

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Geometric Properties:

Span Length $L := 27\ ft$

Girder Spacing - Interior and Exterior $S := 86\ in$ $S_x := 34\ in$

Number of Girders $NG := 4$

Skew Angle $skew := 0\ °$

Width and Number of Lanes $lanewidth := 10\ ft$ $Nlane := 2$

Wearing Surface Thickness $ws := 13\ in$

Girder Height - Interior, Exterior $h := 33\ in$ $h_x := 33\ in$

Deck Thickness $d_s := 8\ in$

Web Height - Interior, Exterior $d_g := h - d_s$ $d_{gx} := h_x - d_s$

Web Thickness - Interior, Exterior $b_w := 16\ in$ $b_{wx} := 12\ in$

Distance from Centerline of Girder to Edge of Curb $d_e := -0.75\ ft$

Height of Curb Above Deck $h_{curb} := 17\ in$

Width of Curb $b_{curb} := 18\ in$

Height to Centroid of Reinforcement - Interior, Exterior

$$y_{bar} := \begin{bmatrix} 3 \\ 3.5 \\ 4.4 \\ 3.5 \\ 3 \end{bmatrix} \mathbf{in} \qquad y_{barx} := \begin{bmatrix} 3 \\ 3.5 \\ 3.5 \\ 3.5 \\ 3 \end{bmatrix} \mathbf{in}$$

Dictance Between Centroids of Deck and Web

$$e_g := \frac{d_g + d_s}{2} = 16.5 \ \mathbf{in}$$

Depth of Reinforcement - Interior, Exterior

$$d := h - y_{bar} \qquad d_x := h_x - y_{barx} + h_{curb}$$

Area of Reinforcement at All Changes - Interior, Exterior

$$A_s := \begin{bmatrix} 6 \\ 8 \\ 10 \\ 8 \\ 6 \end{bmatrix} \mathbf{in}^2 \qquad A_{sx} := \begin{bmatrix} 2 \\ 4 \\ 4 \\ 4 \\ 2 \end{bmatrix} \mathbf{in}^2$$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder $P_{dint} := 0 \ \mathbf{kip}$

Resistance Factors - Structural Dead load, Weading Surface, Live Load

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.25 \quad \gamma_{LL} := 1.35$$

Impact Factor $IM := 0.33$

Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 0.95$

Distributed Load Due to Rail $w_{rail} := 0.1 \ \dfrac{\mathbf{kip}}{\mathbf{ft}}$

Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway $exc := 0 \ \mathbf{in}$

$$P_{dext} := \frac{P_{dint}}{2}$$

Distribution Factors

Area and Moment of Inertia of Web $A := d_g \cdot b_w = 400\ in^2$ $\quad I := \dfrac{b_w \cdot {d_g}^3}{12} = (2.083 \cdot 10^4)\ in^4$

$n := 1$

Modular Ratio - Deck and Web

Longitudinal Stiffness Parameter $K_g := n \cdot (I + A \cdot {e_g}^2) = (1.297 \cdot 10^5)\ in^4$

Interior Moment Distribution Factor

$$g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot {d_s}^3}\right)^{0.1} = 0.561$$

$$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot {d_s}^3}\right)^{0.1} = 0.707$$

$$g_m := \max(g_{m1}, g_{m2})$$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From Center of Gravity of Girders $e_1 := \dfrac{W_r}{2} - 5\ ft + exc = 5\ ft$

Eccentricity of Exterior Girder From Center of Gravity of Girders $X_{ext} := (NG - 1) \cdot \dfrac{S}{2} = 10.75\ ft$

Eccentricity of Each Girder

$x_1 := X_{ext}$
$x_2 := X_{ext} - S$
$x_3 := X_{ext} - 2 \cdot S$

$x_4 := \text{if}(NG > 3, X_{ext} - 3 \cdot S, 0\ ft)$
$x_5 := \text{if}(NG > 4, X_{ext} - 4 \cdot S, 0\ ft)$
$x_6 := \text{if}(NG > 5, X_{ext} - 5 \cdot S, 0\ ft)$
$x_7 := \text{if}(NG > 6, X_{ext} - 6 \cdot S, 0\ ft)$

Lever Rule - One Design Lane

$$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{{x_1}^2 + {x_2}^2 + {x_3}^2 + {x_4}^2 + {x_5}^2 + {x_6}^2 + {x_7}^2} = 0.459$$

$$g_{mR1} := \text{if}(P_{dint} > 0, 1.2 \cdot R_1, 0) = 0$$

Lever Rule - Two Design Lanes

$$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot (e_1 - 5\ ft)}{{x_1}^2 + {x_2}^2 + {x_3}^2 + {x_4}^2 + {x_5}^2 + {x_6}^2 + {x_7}^2} = 0.5$$

$$g_{mR2} := \text{if}\left(P_{dint} > 0, R_2, 0\right) = 0$$

Exterior Moment Distribution Factor

$$g_{mx1} := \frac{1.2\ (S + d_e - 2\ ft)}{2 \cdot S} = 0.37$$

$$ee := 0.77 + \frac{d_e}{9\ ft} = 0.687$$

$$g_{mx2} := g_{m2} \cdot ee = 0.485$$

$$g_{mx} := \max\left(g_{mx1}, g_{mx2}\right)$$

Interior	Exterior
$g_m = 0.707$	$g_{mx} = 0.485$

Loading

Interior Girder Dead Load

$$w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.417\ \frac{kip}{ft}$$

Deck Dead Load

$$w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.717\ \frac{kip}{ft}$$

Curb Dead Load

$$w_{curb} := \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.319\ \frac{kip}{ft}$$

Dead Load from Nonstructural Components

$$w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.18\ \frac{kip}{ft}$$

Total Dead Load

$$DC := w_{girder} + w_{deck} + w_{ns} = 1.313\ \frac{kip}{ft}$$

Exterior Girder Dead Load

$$w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.313\ \frac{kip}{ft}$$

Exterior Deck Dead Load

$$w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.283\ \frac{kip}{ft}$$

Total Exterior Dead Load $$DC_x := w_{girderx} + w_{deckx} + w_{ns} = 0.776\ \frac{kip}{ft}$$

Wearing Surface Dead Load $$DW := \gamma_{ws} \cdot ws \cdot S = 1.118\ \frac{kip}{ft}$$

Exterior Wearing Surface Dead Load $$DW_x := \gamma_{ws} \cdot ws \cdot S_x = 0.442\ \frac{kip}{ft}$$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} = 119.649\ ft \cdot kip \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} = 70.669\ ft \cdot kip$$

$$M_{DW} := \frac{DW \cdot L^2}{8} = 101.878\ ft \cdot kip \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8} = 40.277\ ft \cdot kip$$

Live Load Moment - Truck Load

$$M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 206\ ft \cdot kip$$

Live Load Moment - Tandem

$$M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 287.5\ ft \cdot kip$$

Live Load Moment - Lane

$$M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 58.32\ ft \cdot kip$$

Total HL-93 Live Load

$$M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right) = 440.695\ ft \cdot kip$$

Nominal Resistance

Depth of Interior Whitney Stress Block

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 0.542 \\ 0.722 \\ 0.903 \\ 0.722 \\ 0.542 \end{bmatrix} \; in$$

Depth of Exterior Whitney Stress Block

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 0.457 \\ 0.913 \\ 0.913 \\ 0.913 \\ 0.457 \end{bmatrix} \; in$$

Nominal Interior Moment Resistance

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 490.531 \\ 641.055 \\ 774.085 \\ 641.055 \\ 490.531 \end{bmatrix} \; ft \cdot kip$$

Midspan Moment Capacity

$$M_{capacity} := \max\left(M_n\right) = 774.085 \; ft \cdot kip$$

Nominal Exterior Moment Resistance

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 257.244 \\ 506.476 \\ 506.476 \\ 506.476 \\ 257.244 \end{bmatrix} \; (ft \cdot kip)$$

Midspan Moment Capacity

$$M_{capacityx} := \max\left(M_{nx}\right) = 506.476 \; ft \cdot kip$$

Rating Factors

$$RF_{Interior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacity} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m}$$

$$RF_{Exterior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx} - \gamma_{DC} \cdot M_{DCx} - \gamma_{DW} \cdot M_{DWx}}{\gamma_{LL} \cdot M_{LL} \cdot g_{mx}}$$

Interior	Exterior
$RF_{Interior} = 0.915$	$RF_{Exterior} = 1.019$

Rating Factor Improvements

Interior

Maximum Recorded Strain $\varepsilon_T := 65.45 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 329.14\ \mathbf{ft \cdot kip}$

Uncracked Section Modulus $S_{unc} := 5850\ \mathbf{in}^3$

Cracked Section Modulus $S_{cr} := 2185\ \mathbf{in}^3$

Section Behavior $Behavior := \text{“Uncracked”}$

Section Modulus Effective for Behavior

$$S_e := \mathbf{if}\left(Behavior = \text{“Uncracked”}, S_{unc}, S_{cr}\right) = \left(5.85 \cdot 10^3\right)\ \mathbf{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.173 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 0.792$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.747$

Test Understanding Factor $k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.396$

Improved Rating Factor $RF_{Improved} := RF_{Interior} \cdot k = 1.278$

Exterior

Maximum Recorded Strain $\varepsilon_T := 27.74 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 329.14\ \boldsymbol{ft \cdot kip}$

Uncracked Section Modulus $S_{unc} := 5894\ \boldsymbol{in}^3$

Cracked Section Modulus $S_{cr} := 1832\ \boldsymbol{in}^3$

Section Behavior $Behavior :=$ "Uncracked"

Section Modulus Effective for Behavior

$$S_e := \textbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(5.894 \cdot 10^3\right)\ \boldsymbol{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 7.993 \cdot 10^{-5}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 1.881$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.747$

Test Understanding Factor $k_b := \textbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.941$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 1.978$

Summary

Interior	Exterior
$RF_{Improved} = 1.278$	$RF_{ImprovedExt} = 1.978$

Bridge 2130

Interior Section Properties

$y_{unc} := 21.4\ in \quad I_{unc} := 125330\ in^4 \quad S_{unc} := 5850\ in^3 \quad N_i := 2$

$y_{cr} := 25.9\ in \quad I_{cr} := 56621\ in^4 \quad S_{cr} := 2185\ in^3$

Exterior Section Properties

$y_{uncx} := 25.9\ in \quad I_{uncx} := 152933\ in^4 \quad S_{uncx} := 5894\ in^3 \quad N_x := 2$

$y_{crx} := 32.1\ in \quad I_{crx} := 58763\ in^4 \quad S_{crx} := 1832\ in^3$

Test Parameters

Maximum Applied Moment
$M := 658.28\ ft \cdot kip$

Maximum Measured Strain
$\varepsilon_m := 65.45 \cdot 10^{-6}$

Using Nominal Compressive Strength - 2.5 ksi

$f'_c := 2.5\ ksi \qquad E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(2.878 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 1.169 \cdot 10^{-4}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 3.417 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Using More Realistic Compressive Strength - 5 ksi

$f'_c := 5\ ksi \qquad E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 8.264 \cdot 10^{-5}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.416 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  "Uncracked"
else
  "Cracked"

Distributiuon Factors from Testing
Bridge 2130

Compressive Strength $f'_c := 5\ ksi$

Section Modulus for Each Girder $S := \begin{bmatrix} 5894 \\ 5850 \\ 5850 \\ 5894 \end{bmatrix} in^3$

Strains Recorded During SBS & ALT Tests

$$\varepsilon_2 := \begin{bmatrix} 14.43 \\ 47.97 \\ 59.82 \\ 22.18 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 14.47 \\ 52.66 \\ 65.45 \\ 24.98 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity $E_c := 1820\ ksi \cdot \sqrt{\dfrac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

Moment Produced in Each Girder

$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 28.844 \\ 95.17 \\ 118.68 \\ 44.335 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 28.924 \\ 104.475 \\ 129.85 \\ 49.932 \end{bmatrix} ft \cdot kip$$

Total Moment

$$M_{sum2} := \sum M_2 = 287.029\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 313.18\ ft \cdot kip$$

Distribution Factors

$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.201 \\ 0.663 \\ 0.827 \\ 0.309 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.185 \\ 0.667 \\ 0.829 \\ 0.319 \end{bmatrix}$$

A.6 Franklin No. 3307

A.6.1 Input Data, Experimental Configuration, and Experimental Data Collected

Table 21:Franklin No. 3307 input data, experimental configuration, and experimental data collected

File Contents	***File Name***	***File Type***
Exterior Section Data	Br3307_Geom _Ext.csv	CSV Format
Interior Section Data	Br3307_Geom _Int.csv	CSV Format
Sensors	Br3307 _Sensors.csv	CSV Format
Sensor Layout	Br3307_SensorLayout.mat	MATLAB Data File
Truck Weight and Dimensions	Br3307_Trucks.mat	MATLAB Data File
Sensor Data	Br_3307_ALT_1_1_Strain.mat	MATLAB Data File
	Br_3307_BTB_1_1_Strain.mat	MATLAB Data File
	Br_3307_BTB_1_2_Strain.mat	MATLAB Data File
	Br_3307_BTB_2_1_Strain.mat	MATLAB Data File
	Br_3307_BTB_3_1_Strain.mat	MATLAB Data File
	Br_3307_MAX_1_1_Strain.mat	MATLAB Data File
	Br_3307_MAX_1_2_Strain.mat	MATLAB Data File
	Br_3307_MAX_2_1_Strain.mat	MATLAB Data File
	Br_3307_MAX_3_1_Strain.mat	MATLAB Data File
	Br_3307_SBS_1_1_Strain.mat	MATLAB Data File
	Br_3307_SBS_1_2_Strain.mat	MATLAB Data File
	Br_3307_SBS_2_1_Strain.mat	MATLAB Data File
	Br_3307_SBS_3_1_Strain.mat	MATLAB Data File

A.6.2 Instrumentation

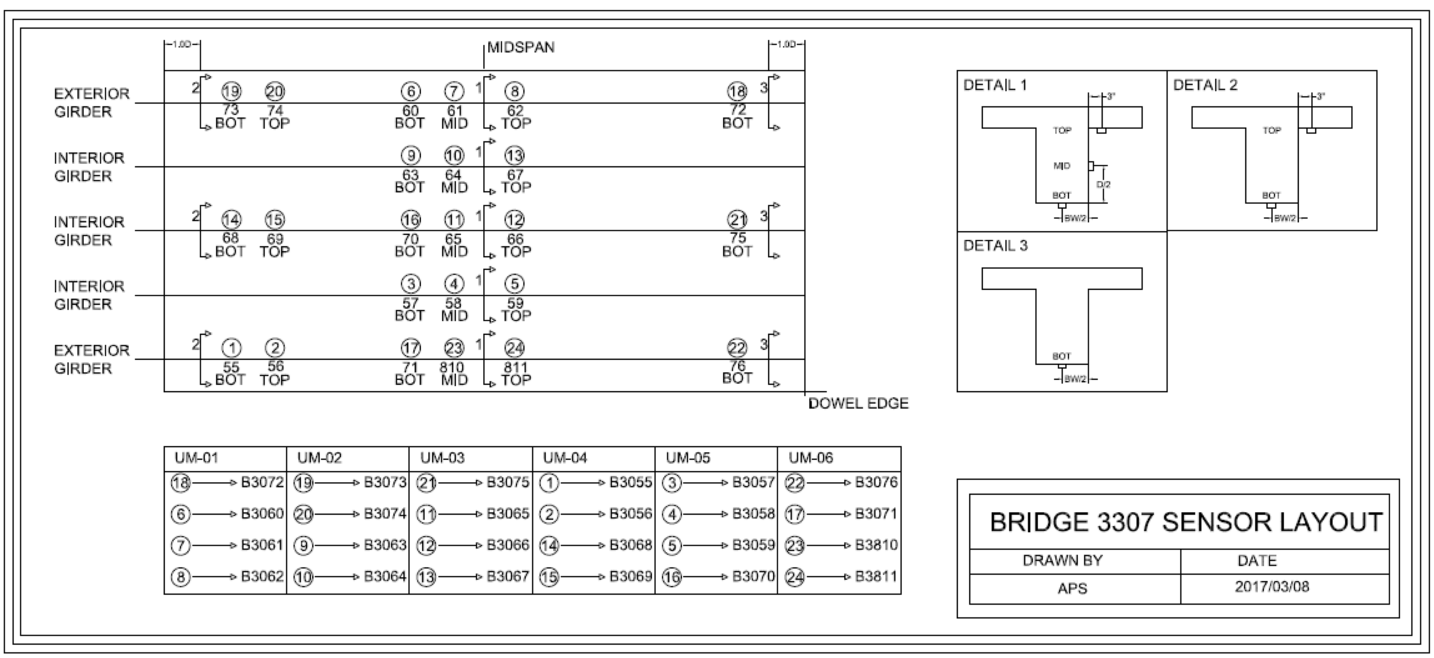

Figure 71: Franklin No. 3307 sensor layout

A.6.3 Loading

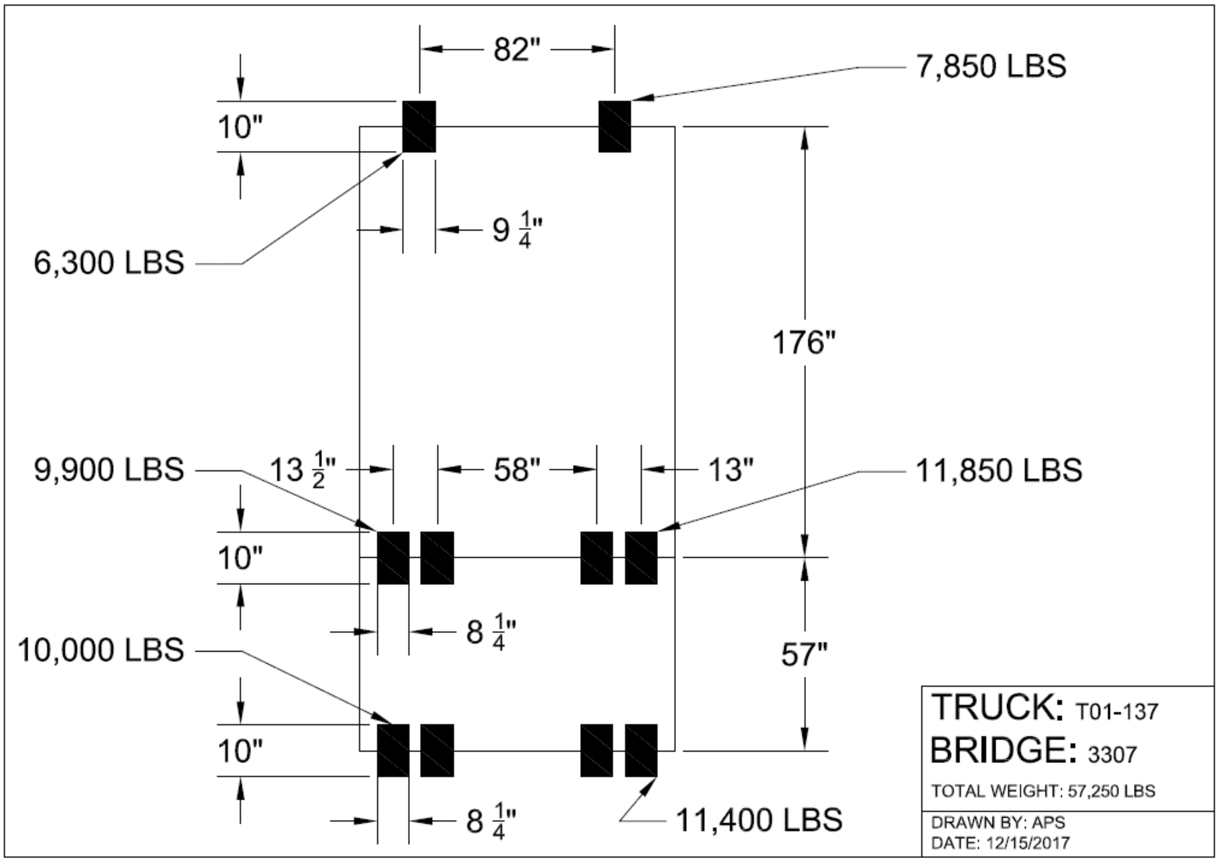

Figure 72: Franklin No. 3307 Truck T01-137 loading

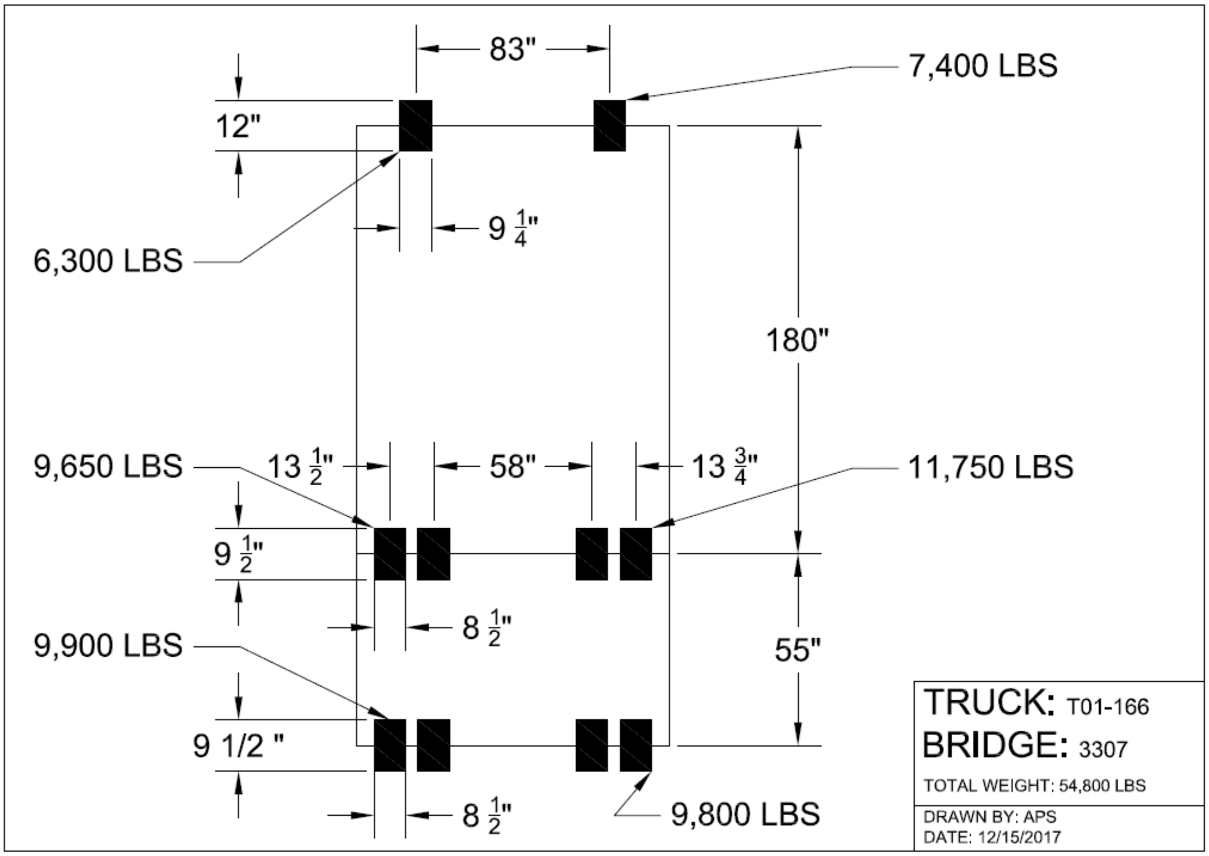

Figure 73: Franklin No. 3307 Truck T01-166 loading

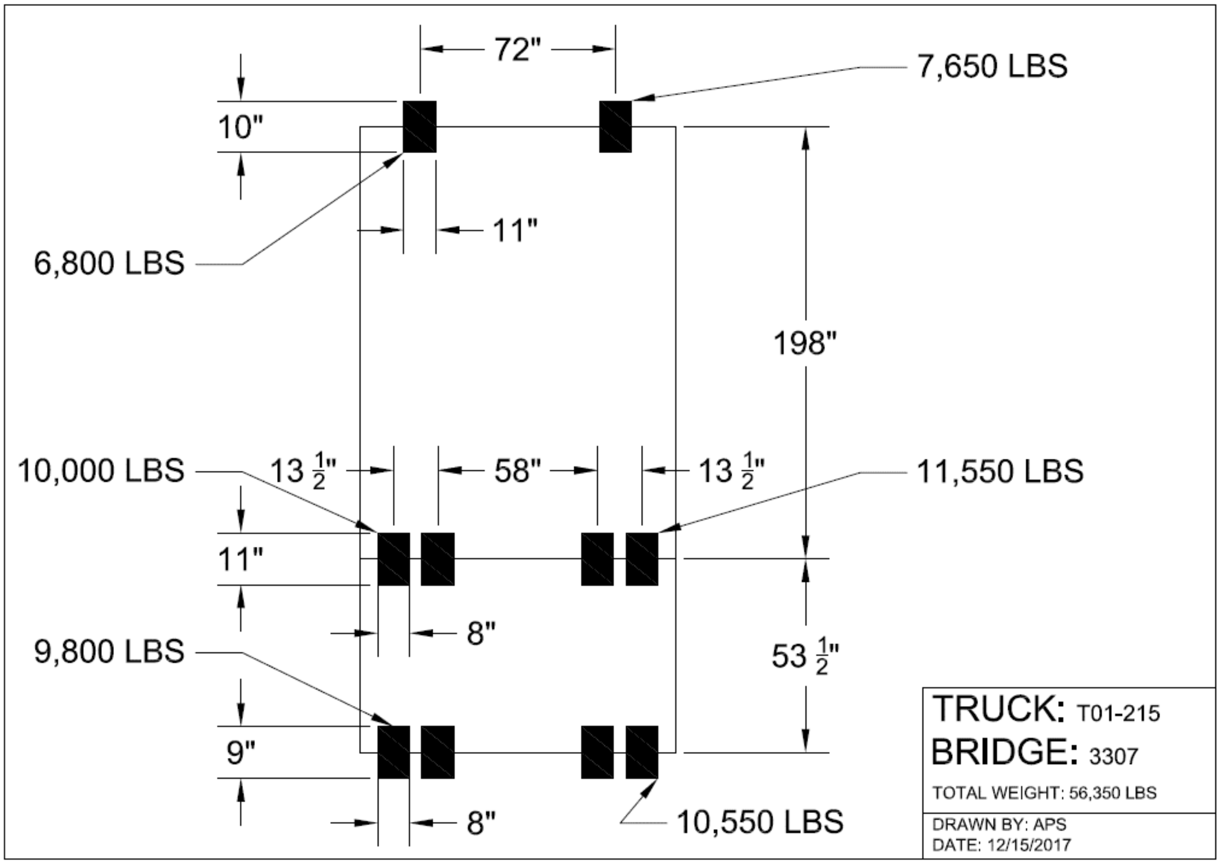

Figure 74: Franklin No. 3307 Truck T01-215 loading

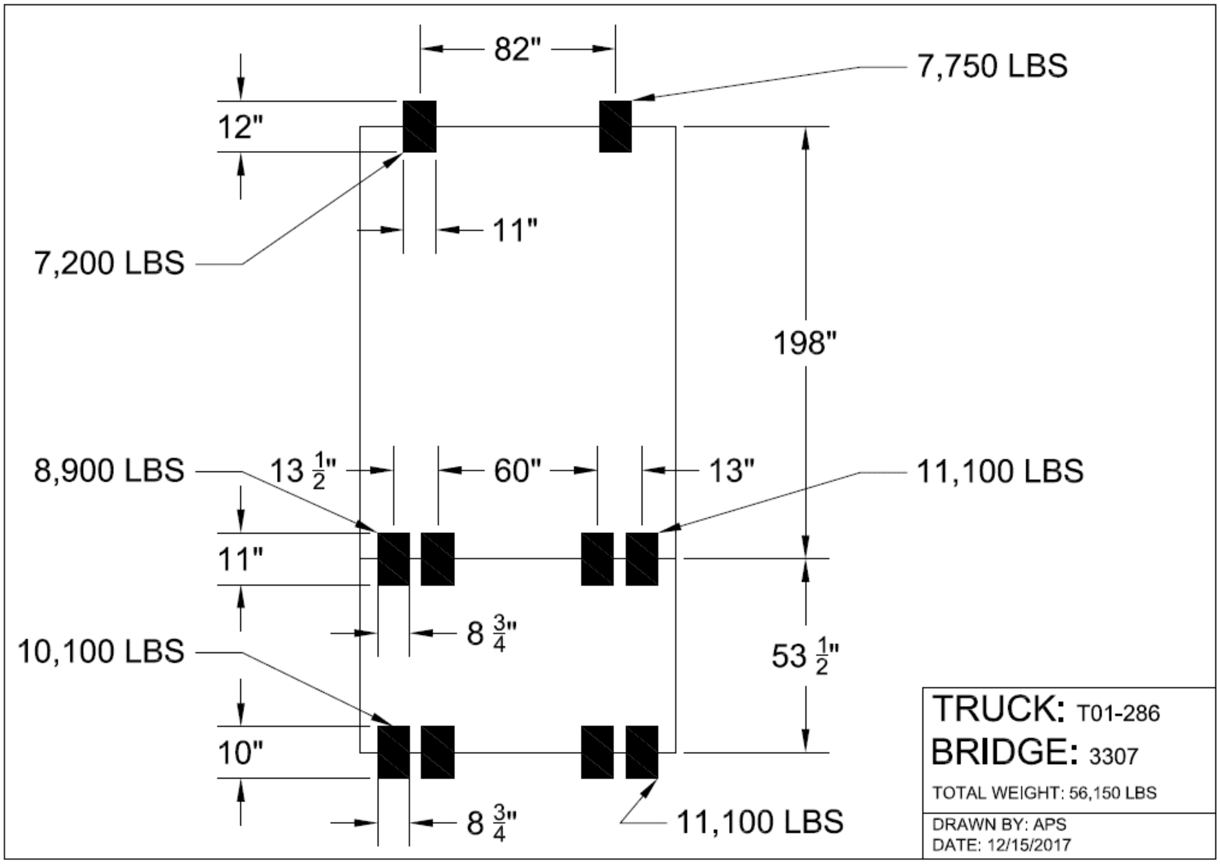

Figure 75: Franklin No. 3307 Truck T01-286 loading

A.6.4 Representative Data Plots

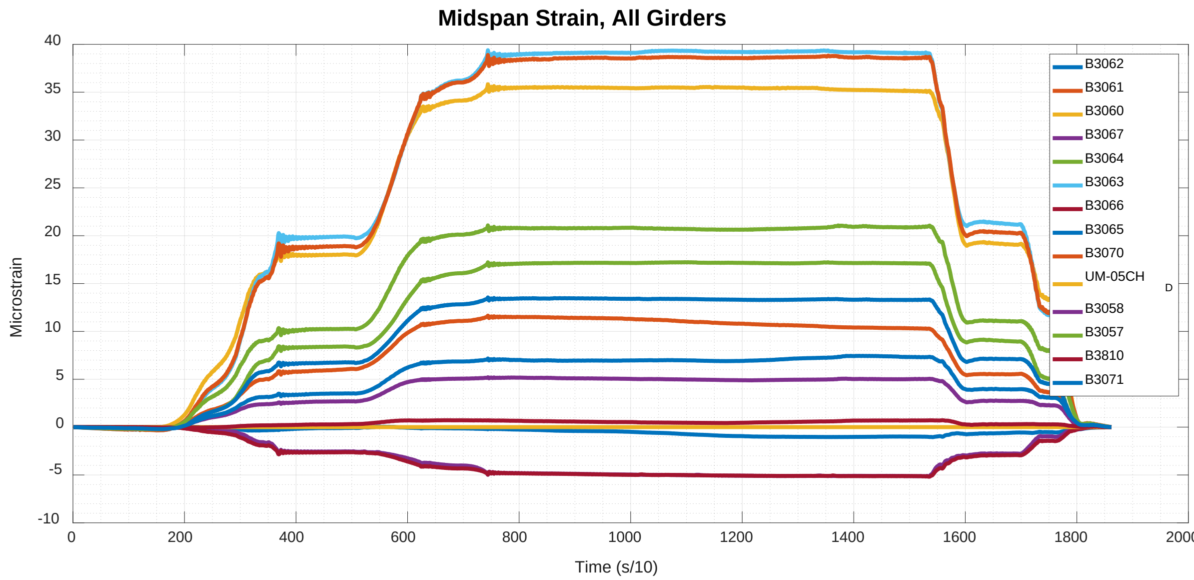

Figure 76: Franklin No. 3307 – BTB_2_1 strains - Midspan

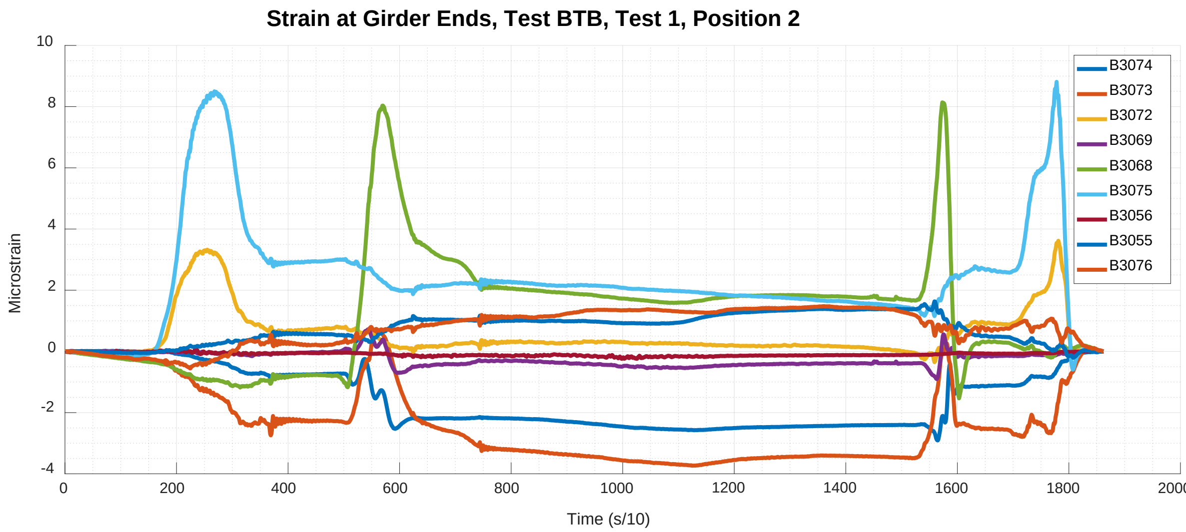

Figure 77: Franklin No. 3307 – BTB_2_1 strains – Ends

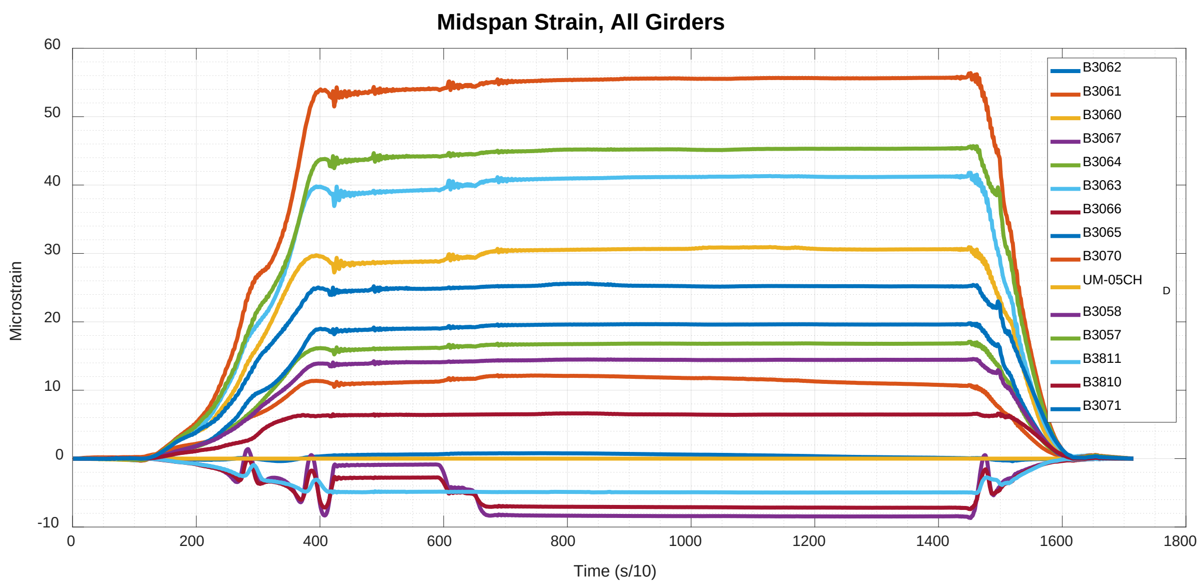

Figure 78: Franklin No. 3307 – SBS_2_1 strains - Midspan

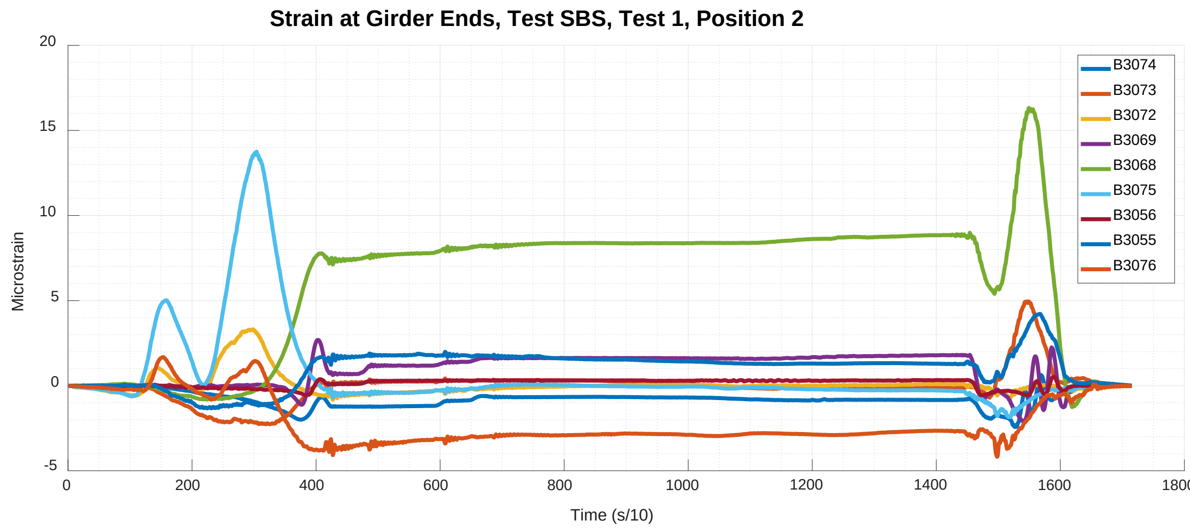

Figure 79: Franklin No. 3307 – SBS_2_1 strains - Ends

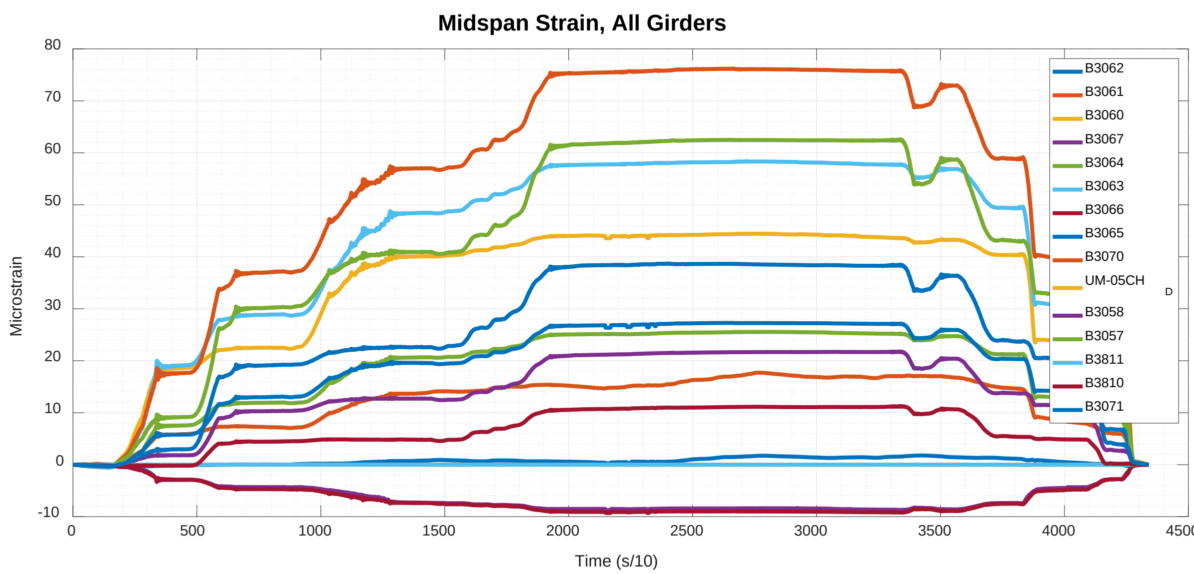

Figure 80: Franklin No. 3307 – MAX_2_1 strains - Midspan

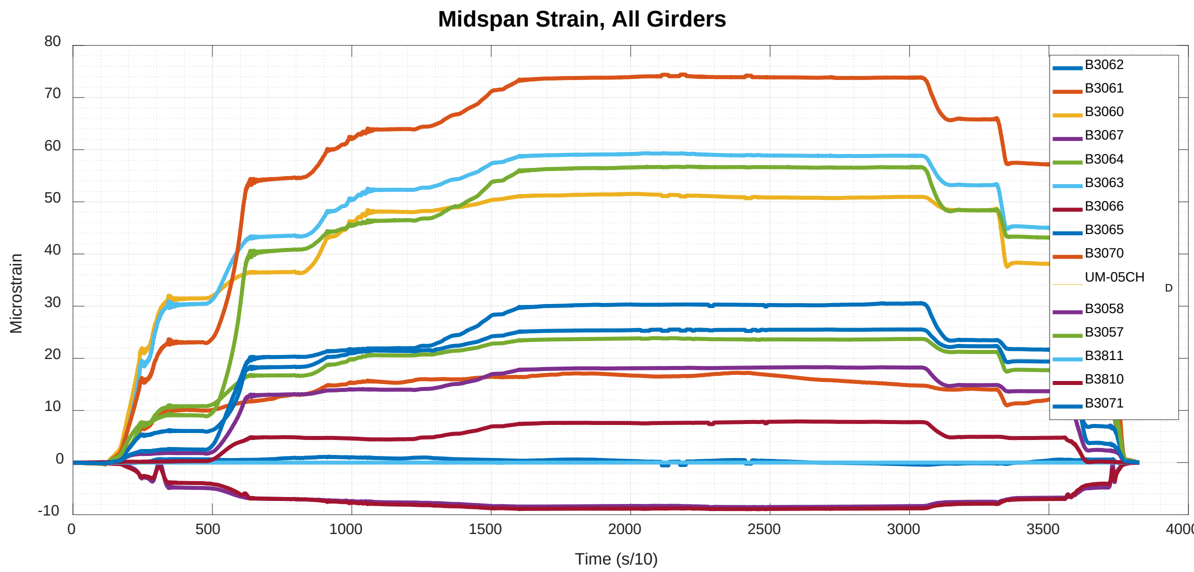

Figure 81: Franklin No. 3307 – BTB_2_1 strains - Ends

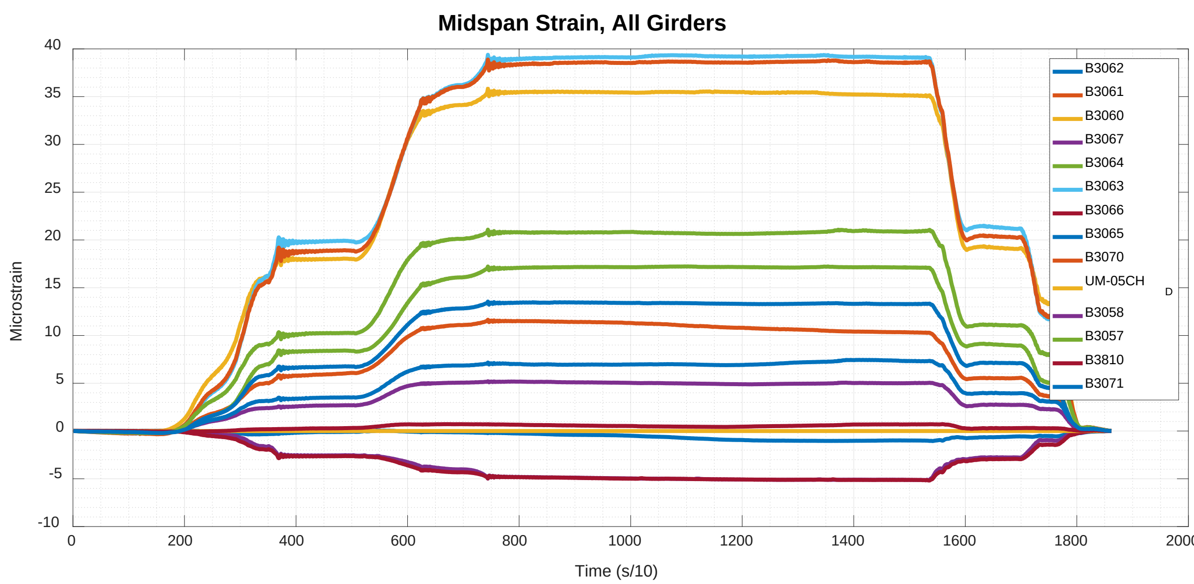

Figure 82: Franklin No. 3307 – BTB_2_1 strains - Midspan

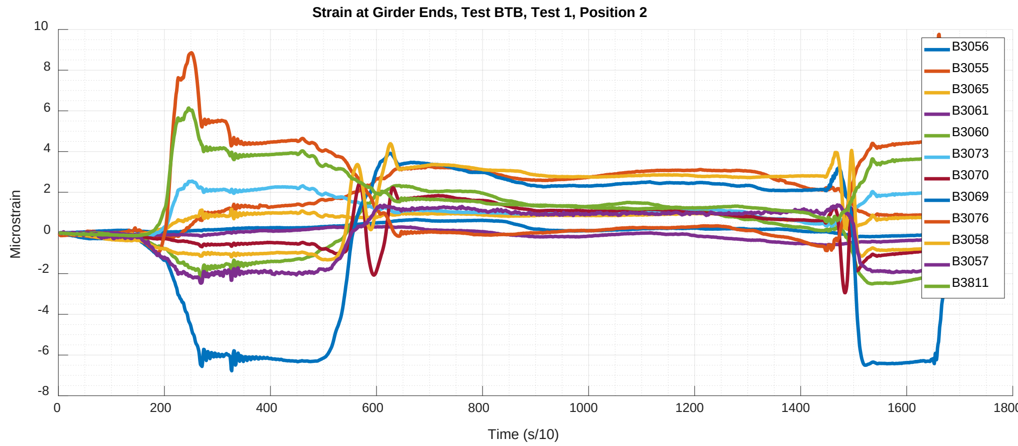

Figure 83: Franklin No. 3307 – BTB_2_1 strains - Ends

A.6.5 Rating Factor Calculations

Figure 84: Franklin No. 3307 Calculations

AASHTO Rating Calculations:

Bridge 3307 - Franklin, Maine

Material Parameters:

Concrete Compressive Strength $f'_c := 5\ ksi$

Reinforcement Yield Strength $F_y := 33\ ksi$

Unit Weights: Reinforced Concrete, Wearing Surface $\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$ $\gamma_{ws} := 0.144\ \frac{kip}{ft^3}$

Concrte Elastic Modulus - LRFD Design C5.4.2.4-1

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Geometric Properties:

Span Length $L := 40\ ft$

Girder Spacing - Interior and Exterior $S := 68.75\ in$ $S_x := 46.875\ in$

Number of Girders $NG := 5$

Skew Angle $skew := 0\ °$

Width and Number of Lanes $lanewidth := 11\ ft$ $Nlane := 2$

Wearing Surface Thickness $ws := 4\ in$

Girder Height - Interior, Exterior $h := 31.5\ in$ $h_x := 31.5\ in$

Deck Thickness $d_s := 5.75\ in$

Web Height - Interior, Exterior $d_g := h - d_s$ $d_{gx} := h_x - d_s$

Web Thickness - Interior, Exterior $b_w := 19\ in$ $b_{wx} := 19\ in$

Distance from Centerline of Girder to Edge of Curb $d_e := -15\ in$

Height of Curb Above Deck $h_{curb} := 12\ in$

Width of Curb $b_{curb} := 18\ in$

Height to Centroid of Reinforcement - Interior, Exterior

$$y_{bar} := \begin{bmatrix} 4.42 \\ 5.125 \\ 6.4 \\ 7.25 \\ 6.4 \\ 5.125 \\ 4.42 \end{bmatrix} \textbf{in} \qquad y_{barx} := \begin{bmatrix} 4.42 \\ 5.125 \\ 6.4 \\ 7.25 \\ 6.4 \\ 5.125 \\ 4.42 \end{bmatrix} \textbf{in}$$

Dictance Between Centroids of Deck and Web

$$e_g := \frac{d_g + d_s}{2} = 15.75 \ \textbf{in}$$

Depth of Reinforcement - Interior, Exterior

$$d := h - y_{bar} \qquad d_x := h_x - y_{barx} + h_{curb}$$

Area of Reinforcement at All Changes - Interior, Exterior

$$A_s := \begin{bmatrix} 7.594 \\ 10.125 \\ 12.656 \\ 15.188 \\ 12.656 \\ 10.125 \\ 7.594 \end{bmatrix} \textbf{in} \ A_{sx} := \begin{bmatrix} 7.594 \\ 10.125 \\ 12.656 \\ 15.188 \\ 12.656 \\ 10.125 \\ 7.594 \end{bmatrix} \textbf{in}^2$$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder

$$P_{dint} := 0 \ \textbf{kip}$$

Resistance Factors - Structural Dead load, Weading Surface, Live Load

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.25 \quad \gamma_{LL} := 1.35$$

Impact Factor

$$IM := 0.33$$

Resistance Factor

$$\phi := .9$$

System Factor

$$\phi_s := 1.0$$

Condition Factor

$$\phi_c := 1.0$$

Distributed Load Due to Rail

$$w_{rail} := 0.1 \ \frac{\textbf{kip}}{\textbf{ft}}$$

Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway

$$exc := 0 \ \textbf{in}$$

$$P_{dext} := \frac{P_{dint}}{2}$$

Distribution Factors

Area and Moment of Inertia of Web $A := d_g \cdot b_w = 489.25\ in^2$ $I := \frac{b_w \cdot d_g^{\ 3}}{12} = \left(2.703 \cdot 10^4\right)\ in^4$

$n := 1$

Modular Ratio - Deck and Web

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(1.484 \cdot 10^5\right)\ in^4$

Interior Moment Distribution Factor

$$g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.47$$

$$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.6$$

$$g_m := \max\left(g_{m1}, g_{m2}\right)$$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From Center of Gravity of Girders $e_1 := \frac{W_r}{2} - 5\ ft + exc = 6\ ft$

Eccentricity of Exterior Girder From Center of Gravity of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 11.458\ ft$

Eccentricity of Each Girder

$$x_1 := X_{ext}$$
$$x_2 := X_{ext} - S$$
$$x_3 := X_{ext} - 2 \cdot S$$

$$x_4 := \text{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ ft\right)$$
$$x_5 := \text{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ ft\right)$$
$$x_6 := \text{if}\left(NG > 5, X_{ext} - 5 \cdot S, 0\ ft\right)$$
$$x_7 := \text{if}\left(NG > 6, X_{ext} - 6 \cdot S, 0\ ft\right)$$

Lever Rule - One Design Lane

$$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2} + x_6^{\ 2} + x_7^{\ 2}} = 0.409$$

$$g_{mR1} := \text{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0$$

Lever Rule - Two Design Lanes $$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot \left(e_1 - 5\ ft\right)}{{x_1}^2 + {x_2}^2 + {x_3}^2 + {x_4}^2 + {x_5}^2 + {x_6}^2 + {x_7}^2} = 0.435$$

$$g_{mR2} := \text{if}\left(P_{dint} > 0, R_2, 0\right) = 0$$

Exterior Moment Distribution Factor $$g_{mx1} := \frac{1.2\ \left(S + d_e - 2\ ft\right)}{2 \cdot S} = 0.26$$

$$ee := 0.77 + \frac{d_e}{9\ ft} = 0.631$$

$$g_{mx2} := g_{m2} \cdot ee = 0.379$$

$$g_{mx} := \max\left(g_{mx1}, g_{mx2}\right)$$

<u>Interior</u>	<u>Exterior</u>
$g_m = 0.6$	$g_{mx} = 0.379$

Loading

Interior Girder Dead Load $$w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.51\ \frac{kip}{ft}$$

Deck Dead Load $$w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.412\ \frac{kip}{ft}$$

Curb Dead Load $$w_{curb} := \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.225\ \frac{kip}{ft}$$

Dead Load from Nonstructural Components $$w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.145\ \frac{kip}{ft}$$

Total Dead Load $$DC := w_{girder} + w_{deck} + w_{ns} = 1.066\ \frac{kip}{ft}$$

Exterior Girder Dead Load $$w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.51\ \frac{kip}{ft}$$

Exterior Deck Dead Load $$w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.281\ \frac{kip}{ft}$$

Total Exterior Dead Load $$DC_x := w_{girderx} + w_{deckx} + w_{ns} = 0.935\ \frac{kip}{ft}$$

Wearing Surface Dead Load $$DW := \gamma_{ws} \cdot ws \cdot S = 0.275\ \frac{kip}{ft}$$

Exterior Wearing Surface Dead Load $$DW_x := \gamma_{ws} \cdot ws \cdot S_x = 0.188\ \frac{kip}{ft}$$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} = 213.284\ ft \cdot kip \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} = 187.079\ ft \cdot kip$$

$$M_{DW} := \frac{DW \cdot L^2}{8} = 55\ ft \cdot kip \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8} = 37.5\ ft \cdot kip$$

Live Load Moment - Truck Load

$$M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 440\ ft \cdot kip$$

Live Load Moment - Tandem

$$M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 450\ ft \cdot kip$$

Live Load Moment - Lane

$$M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 128\ ft \cdot kip$$

Total HL-93 Live Load

$$M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right) = 726.5\ ft \cdot kip$$

Nominal Resistance

Depth of Interior Whitney Stress Block

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 0.858 \\ 1.144 \\ 1.429 \\ 1.715 \\ 1.429 \\ 1.144 \\ 0.858 \end{bmatrix} \, \mathbf{in}$$

Depth of Exterior Whitney Stress Block

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 1.258 \\ 1.677 \\ 2.096 \\ 2.516 \\ 2.096 \\ 1.677 \\ 1.258 \end{bmatrix} \, \mathbf{in}$$

Nominal Interior Moment Resistance

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 556.57 \\ 718.459 \\ 848.706 \\ 977.027 \\ 848.706 \\ 718.459 \\ 556.57 \end{bmatrix} \, \mathbf{ft} \cdot \mathbf{kip}$$

Midspan Moment Capacity

$$M_{capacity} := \max\left(M_n\right) = 977.027 \, \mathbf{ft} \cdot \mathbf{kip}$$

Nominal Exterior Moment Resistance

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 802.992 \\ 1.045 \cdot 10^3 \\ 1.255 \cdot 10^3 \\ 1.462 \cdot 10^3 \\ 1.255 \cdot 10^3 \\ 1.045 \cdot 10^3 \\ 802.992 \end{bmatrix} \, (\mathbf{ft} \cdot \mathbf{kip})$$

Midspan Moment Capacity

$$M_{capacityx} := \max\left(M_{nx}\right) = \left(1.462 \cdot 10^3\right) \, \mathbf{ft} \cdot \mathbf{kip}$$

Rating Factors

$$RF_{Interior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacity} - \gamma_{DC} \cdot M_{DC} - \gamma_{DW} \cdot M_{DW}}{\gamma_{LL} \cdot M_{LL} \cdot g_m}$$

$$RF_{Exterior} := \frac{\phi \cdot \phi_s \cdot \phi_c \cdot M_{capacityx} - \gamma_{DC} \cdot M_{DCx} - \gamma_{DW} \cdot M_{DWx}}{\gamma_{LL} \cdot M_{LL} \cdot g_{mx}}$$

Interior	Exterior
$RF_{Interior} = 0.924$	$RF_{Exterior} = 2.784$

Rating Factor Improvements

Interior

Maximum Recorded Strain $\varepsilon_T := 76.16 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 609.88\ \boldsymbol{ft \cdot kip}$

Uncracked Section Modulus $S_{unc} := 5712\ \boldsymbol{in}^3$

Cracked Section Modulus $S_{cr} := 2250\ \boldsymbol{in}^3$

Section Behavior $Behavior := \text{"Uncracked"}$

Section Modulus Effective for Behavior

$$S_e := \mathbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right) = \left(5.712 \cdot 10^3\right)\ \boldsymbol{in}^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.89 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 1.482$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.839$

Test Understanding Factor $k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.741$

Improved Rating Factor $RF_{Improved} := RF_{Interior} \cdot k = 1.608$

Exterior

Maximum Recorded Strain $\varepsilon_T := 44.3 \cdot 10^{-6}$

Maximum Applied Moment per Lane $M_{Max} := 609.88\ ft \cdot kip$

Uncracked Section Modulus $S_{unc} := 5894\ in^3$

Cracked Section Modulus $S_{cr} := 1832\ in^3$

Section Behavior $Behavior := \text{“Uncracked”}$

Section Modulus Effective for Behavior

$$S_e := \text{if}\left(Behavior = \text{“Uncracked”}, S_{unc}, S_{cr}\right) = \left(5.894 \cdot 10^3\right)\ in^3$$

Calculated Strain $\varepsilon_c := \dfrac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 1.156 \cdot 10^{-4}$

Test Benefit Factor $k_a := \dfrac{\varepsilon_c}{\varepsilon_T} - 1 = 1.61$

Ratio of Applied to HL-93 Moment $r_M := \dfrac{M_{Max}}{M_{LL}} = 0.839$

Test Understanding Factor $k_b := \text{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.805$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 5.025$

Summary

Interior	Exterior
$RF_{Improved} = 1.608$	$RF_{ImprovedExt} = 5.025$

Bridge 3307

Interior Section Properties

$y_{unc} := 18.1\ in$ $\quad I_{unc} := 103148\ in^4$ $\quad S_{unc} := 5712\ in^3$ $\quad N_i := 3$

$y_{cr} := 22.8\ in$ $\quad I_{cr} := 51388\ in^4$ $\quad S_{cr} := 2250\ in^3$

Exterior Section Properties

$y_{uncx} := 20.6\ in$ $\quad I_{uncx} := 165682\ in^4$ $\quad S_{uncx} := 8041\ in^3$ $\quad N_x := 2$

$y_{crx} := 26.5\ in$ $\quad I_{crx} := 86963\ in^4$ $\quad S_{crx} := 3276\ in^3$

Test Parameters

Maximum Applied Moment
$M := 1219.76\ ft \cdot kip$

Maximum Measured Strain
$\varepsilon_m := 76.16 \cdot 10^{-6}$

Using Nominal Compressive Strength - 2.5 ksi

$f'_c := 2.5\ ksi$ $\qquad E_c := 1820\ ksi \cdot \sqrt{\dfrac{f'_c}{ksi}} = \left(2.878 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 1.531 \cdot 10^{-4}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 3.824 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  ‖ "Uncracked"
else
  ‖ "Cracked"

Using More Realistic Compressive Strength - 5 ksi

$f'_c := 5\ ksi$ $\qquad E_c := 1820\ ksi \cdot \sqrt{\dfrac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$

$$\varepsilon_{uncracked} := \frac{M}{N_i \cdot S_{unc} \cdot E_c + N_x \cdot S_{uncx} \cdot E_c} = 1.083 \cdot 10^{-4}$$

$$\varepsilon_{cracked} := \frac{M}{N_i \cdot S_{cr} \cdot E_c + N_x \cdot S_{crx} \cdot E_c} = 2.704 \cdot 10^{-4}$$

$Behavior :=$ if $\varepsilon_m < \varepsilon_{uncracked}$ = "Uncracked"
  ‖ "Uncracked"
else
  ‖ "Cracked"

Distributiuon Factors from Testing
Bridge 3307

Compressive Strength

$$f'_c := 5\ ksi$$

Section Modulus for Each Girder

$$S := \begin{bmatrix} 5894 \\ 5712 \\ 5712 \\ 5712 \\ 5894 \end{bmatrix} in^3$$

Strains Recorded During SBS & MAX Tests

$$\varepsilon_2 := \begin{bmatrix} 30.9 \\ 41.3 \\ 55.65 \\ 45.29 \\ 25.24 \end{bmatrix} \cdot 10^{-6} \qquad \varepsilon_4 := \begin{bmatrix} 44.31 \\ 58.21 \\ 76.16 \\ 62.48 \\ 38.65 \end{bmatrix} \cdot 10^{-6}$$

Modulus of Elasticity

$$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = \left(4.07 \cdot 10^3\right)\ ksi$$

Moment Produced in Each Girder

$$M_2 := \overrightarrow{E_c \cdot S \cdot \varepsilon_2} = \begin{bmatrix} 61.765 \\ 80.004 \\ 107.802 \\ 87.734 \\ 50.452 \end{bmatrix} ft \cdot kip \qquad M_4 := \overrightarrow{E_c \cdot S \cdot \varepsilon_4} = \begin{bmatrix} 88.57 \\ 112.762 \\ 147.533 \\ 121.033 \\ 77.256 \end{bmatrix} ft \cdot kip$$

Total Moment

$$M_{sum2} := \sum M_2 = 387.757\ ft \cdot kip \qquad M_{sum4} := \sum M_4 = 547.155\ ft \cdot kip$$

Distribution Factors

$$DF_2 := \frac{2 \cdot M_2}{M_{sum2}} \qquad DF_4 := \frac{2 \cdot M_4}{M_{sum4}}$$

$$DF_2 = \begin{bmatrix} 0.319 \\ 0.413 \\ 0.556 \\ 0.453 \\ 0.26 \end{bmatrix} \qquad DF_4 = \begin{bmatrix} 0.324 \\ 0.412 \\ 0.539 \\ 0.442 \\ 0.282 \end{bmatrix}$$