16 State House Station
Augusta, Maine 04333

MaineDOT

Transportation Research Division

Technical Report 19-01

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

Technical Report Documentation Page

1. Report No. ME 19-01	2.	3. Recipient's Accession No.
4. Title and Subtitle Live Load Testing and Load Rating of Five Skewed Reinforced Concrete T-Beam Bridges		5. Report Date December 2018
		6.
7. Author(s) Andrew Schanck, E.I., Graduate Researcher Bill Davids, Ph.D., P.E.		8. Performing Organization Report No.
9. Performing Organization Name and Address University of Maine		10. Project/Task/Work Unit No. Project 23833.00
		11. Contract © or Grant (G) No.
12. Sponsoring Organization Name and Address Maine Department of Transportation 16 State House Station Augusta, Maine 04333		13. Type of Report and Period Covered
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract (Limit 200 words)

The five skewed reinforced T-beam bridges listed below were tested during the summer of 2018 by the University of Maine (UMaine) in partnership with the Maine Department of Transportation (MaineDOT):

1. Bridge No. 5489 in Levant, carrying Route 222 over Black Stream,
2. Bridge No. 5109 in Hampden, carrying Route 9 over Souadabscook Stream,
3. Bridge No. 2390 in Unity, carrying Town Farm Road over Sandy,
4. Bridge No. 2879 in Atkinson, carrying Stagecoach Road over Piscataquis River,
5. Bridge No. 3848 in Columbia, carrying Saco Road over Western Little River.

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 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.

17. Document Analysis/Descriptors Reinforced concrete T-beam bridges, load rating, live load testing		18. Availability Statement	
19. Security Class (this report)	20. Security Class (this page)	21. No. of Pages 120	22. Price

Live Load Testing and Load Rating of Five Skewed 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
Report Number: 19-09-1613

12-31-2018-Rev00

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

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

Document Log

Name/ Organization	Date	Version	Action
Andrew Schanck, Author William Davids, Author Scott Tomlinson, Reviewer	2018-12-31	Rev00	Initial release to MaineDOT.

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Executive Summary

The five skewed reinforced T-beam bridges listed below were tested during the summer of 2018 by the University of Maine (UMaine) in partnership with the Maine Department of Transportation (MaineDOT):

1. Bridge No. 5489 in Levant, carrying Route 222 over Black Stream,
2. Bridge No. 5109 in Hampden, carrying Route 9 over Souadabscook Stream,
3. Bridge No. 2390 in Unity, carrying Town Farm Road over Sandy,
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5. Bridge No. 3848 in Columbia, carrying Saco Road over Western Little River.

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. Levant No. 5489: On July 31, 2018, maximum applied loading produced 79.5% of HL-93 flexural service load with impact. The rating factors per AASHTO were 0.784 for interior, and 1.88 for exterior girders. Low flexural strains were recorded, which showed that both interior and exterior girders remained uncracked. Negative strains recorded at girder ends indicated a small amount of unintended support restraint. Using the provisions of the AASHTO *Manual for Bridge Evaluation* (AASHTO 2012), the rating factor for HL-93 was increased to 1.10 for the interior girders and 2.36 for the exterior girders.
2. Hampden No. 5109: On August 2, 2018 91.8% of HL-93 flexural service loading with impact was produced from maximum loading. The live load rating factors per AASHTO were 0.686 for the interior girders and 1.59 for the exterior girders. Measured strains indicated uncracked sections and negative strains at interior girder ends indicated unintended fixity. Rating factors were increased for this structure to 0.942 for interior girders and 3.78 for exterior girders, bringing this bridge to an acceptable operating flexural rating. These rating factors are valid when the newer curbs and wearing surface are assumed to be composite with the superstructure.
3. Unity No. 2390: On August 4, 2018 93.2% of HL-93 flexural service loading with impact was produced for this under maximum loading. The initial rating factors per AASHTO were 0.757 for interior girders and 1.05 for exterior girders. Live load testing results allowed the rating factors to be increased to 0.838 for interior girders and 1.15 for exterior girders, meaning that the operating rating factor was unable to be raised above 1.0. This is likely due to the thick asphalt wearing surface overlay.
4. Atkinson No. 2879: On August 9, 2018 92.4% of HL-93 service flexural loading with impact was achieved under maximum applied load. AASHTO calculated rating factors of 1.09 for the interior and 2.57 for the exterior girders. This structure was the only structure whose operating

rating factor began above 1.0. Uncracked section behavior was observed, and rating factors could be increased to 1.35 and 2.76 for interior and exterior girders, respectively.

5. Columbia No. 3848: On August 28, 2018 maximum applied loading produced 80.9% of HL-93 service flexural loading with impact. A small amount of unintended end fixity was observed from negative strains measured at interior girder ends. This bridge exhibited uncracked section behavior, justifying rating factor increases from 0.887 and 1.41 to 1.15 and 2.20 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 2018 as part of this program:

1. Bridge No. 5489 in Levant, carrying Route 222 over Black Stream,
2. Bridge No. 5109 in Hampden, carrying Route 9 over Souadabscook Stream,
3. Bridge No. 2390 in Unity, carrying Town Farm Road over Sandy,
4. Bridge No. 2879 in Atkinson, carrying Stagecoach Road over Piscataquis River,
5. Bridge No. 3848 in Columbia, carrying Saco Road over Western Little River.

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 1931 and 1952 and were originally designed as simply supported with a 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. When two numbers are listed, the first gives the value for interior girders and the second for exterior girders. When one value is listed, the interior and exterior girders are the same.

Table 1: Bridge Characteristics

Bridge	Levant	Hampden	Unity	Atkinson	Columbia
Number	5489	5109	2390	2879	3848
Year Built	1952	1951	1950	1931	1951*
Span - Center to Center of Bearings (feet)	47.0	47.0	37.0	50.0	34.0
Skew (Degrees)	15.0	35.0	30.0	30.0	30.0
Number of Girders	5	5	5	4	5
Girder Spacing (in)	82.0, 54.0	85.8, 57.3	73.5, 42.8	90.0,54.0	70.4, 45.2
Total depth (in)	36.0	39.8	31.3	50.0	29.8
Girder web thickness (in)	19.0	22.8	24.0, 15.0	22.0, 17.0	19.5, 16.0
Slab Thickness (in)	5.50	6.25	5.75	8.00	5.75

*Substructure built in 1943, superstructure built in 1951

1.1 Instrumentation

The strain measurement system used 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

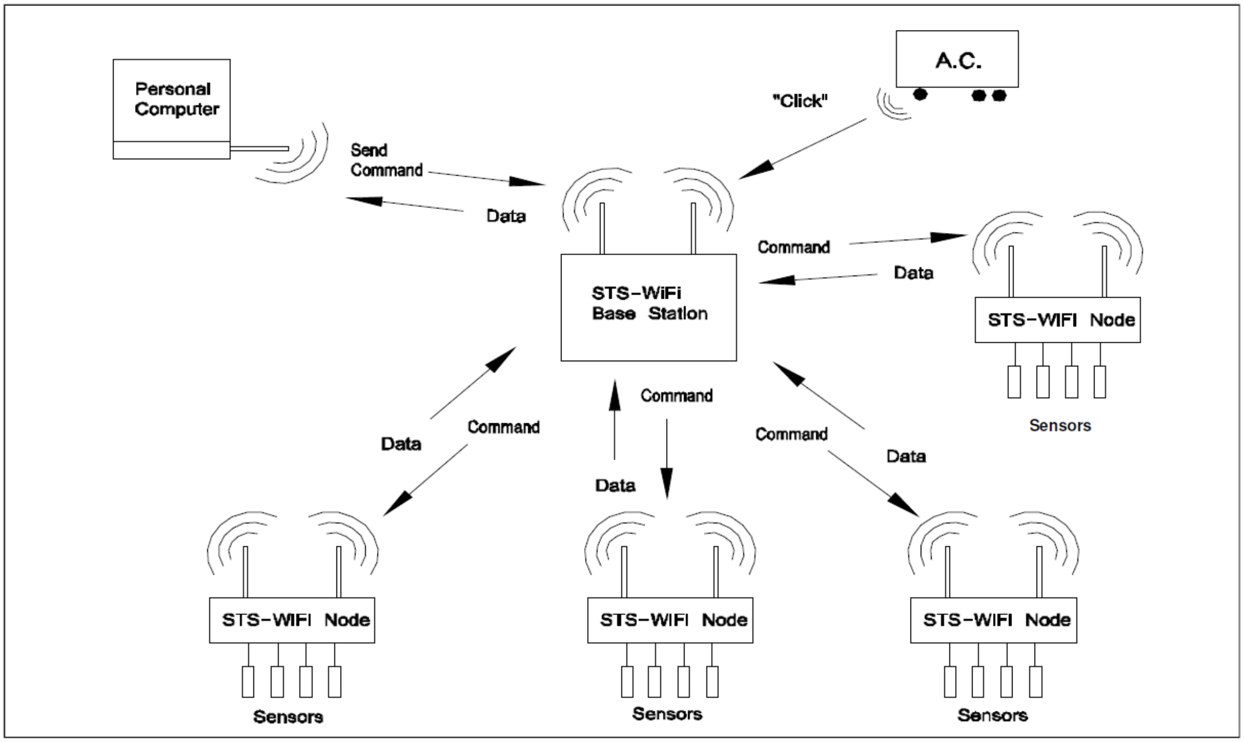

Figure 2: BDI STS-Wi-Fi network setup for bridge sensor setup.

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 measure load distribution and peak flexural strains in each girder. 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 Bridge 5489, A.3.2 for Bridge 5109, A.4.2 for Bridge 2390, A.5.2 for Bridge 2879, and A.6.2 for Bridge 3748.

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 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_Centerline_Test Position_Test Number". Test configurations included two trucks, one in each lane ("SBS"), 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"). Centerline refers to the longitudinal centerline by which truck positions were measured. It was not immediately obvious as to whether positioning trucks relative to the skewed centerline (Figure 6) or perpendicular centerline (Figure 7) would produce larger moments, so both centerline configurations were tested for all configurations. Centerline code "S" refers to tests relative to the skew centerline, and "U" refers to tests with trucks measured relative to the perpendicular centerline. 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. Not all bridges were subjected to all load cases.

Figure 4: Maine DOT three axle trucks used for loading

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

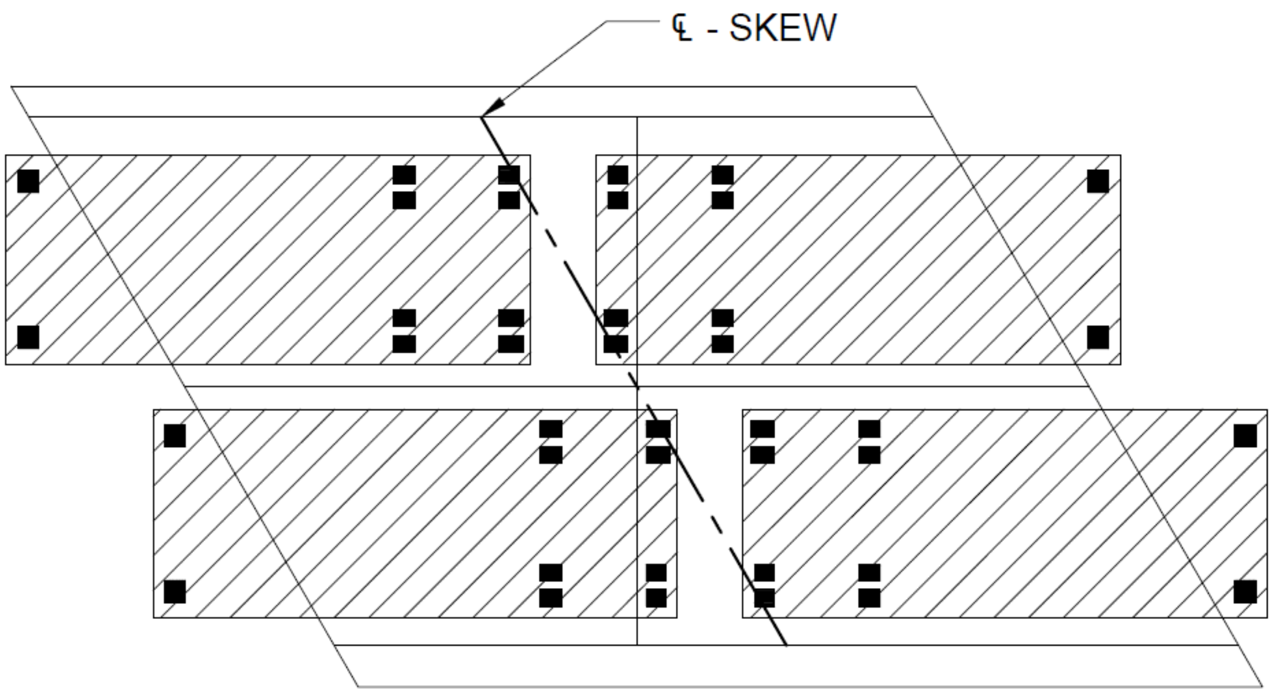

Figure 6: Truck positioning relative to skew centerline

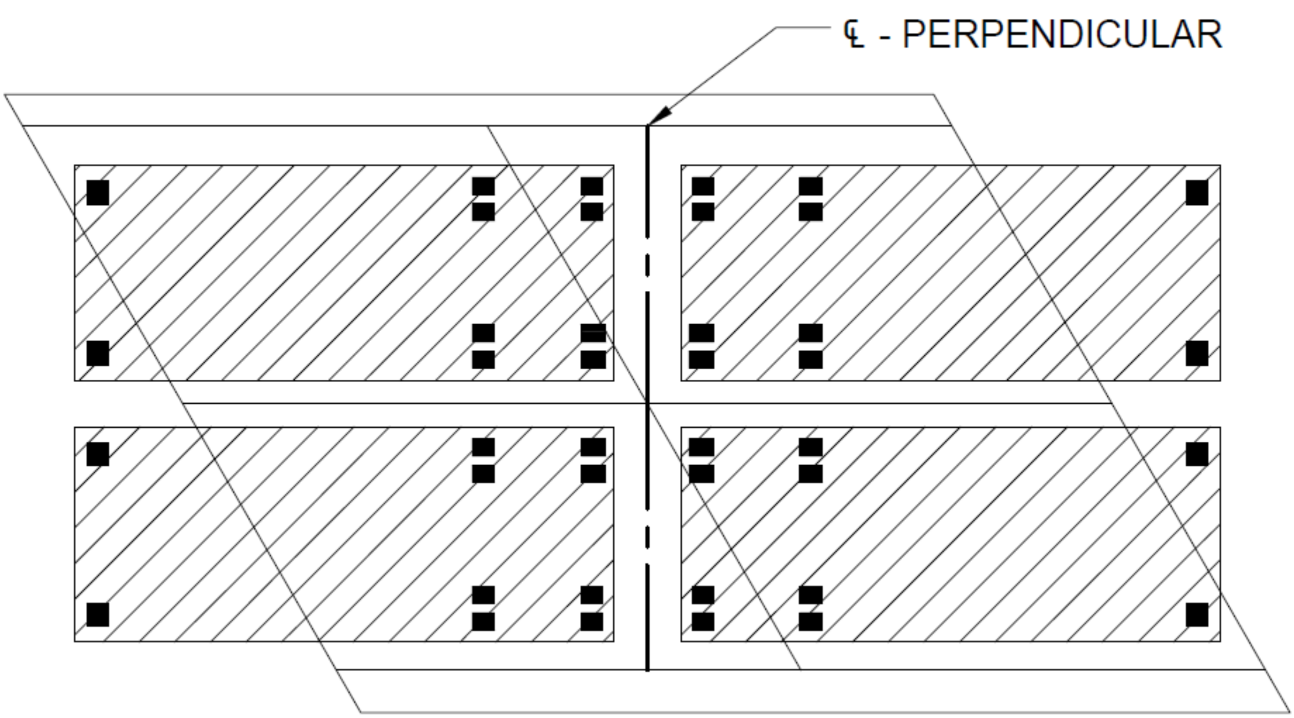

Figure 7: Truck positioning relative to perpendicular centerline

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. Bridge No. 3848 had typical geometry and results for all test configurations. Figure 8 shows a time history of the strains measured at midspan of the center girder during the MAX_S_2_1 test, and Figure 9 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 were positioned such that two trucks were arranged back to back in each lane with their rear tandem wheels spaced approximately symmetrically about the skew longitudinal centerline. All four trucks were equidistant the striped centerline. After 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 8 which demark a truck backing onto or pulling off the bridge.

In addition to showing the girder's response to sequential loading Figure 8 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 many of the sections behaved as uncracked under test loading and had not experienced significant flexural cracking due to prior loading. Figure 9 shows the typical behavior of girder ends. At both ends of the girder, the bottom of the section experienced small compressive strains throughout the section depth at one end, indicating that some unintended end restraint was present, but did not greatly affect overall bridge response. This was common of many of the bridges.

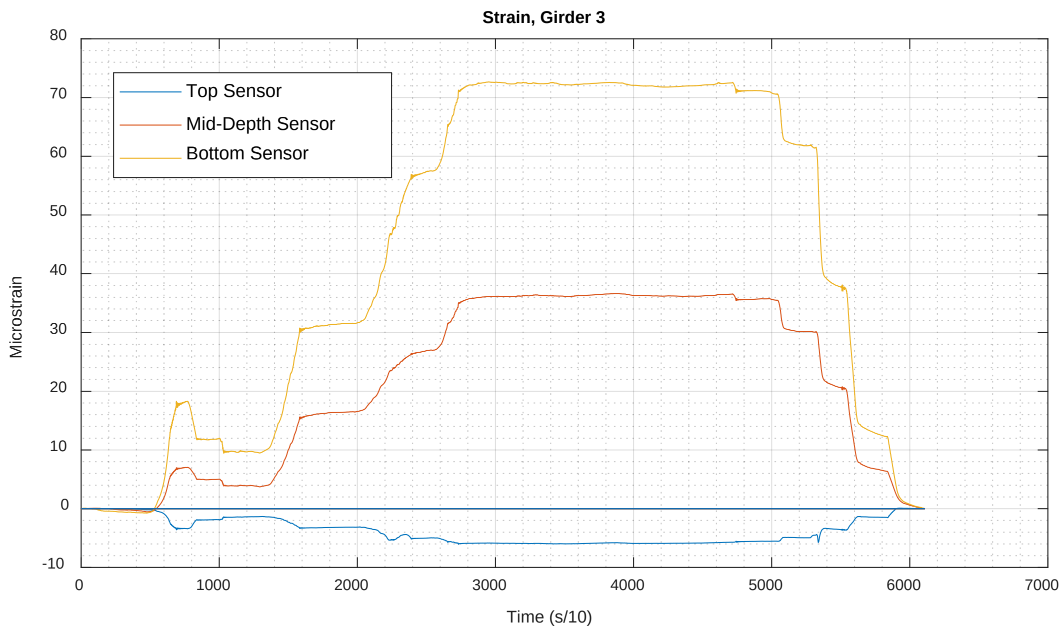

Figure 8: Bridge 3848 – MAX_S_2_1, center girder strains at midspan

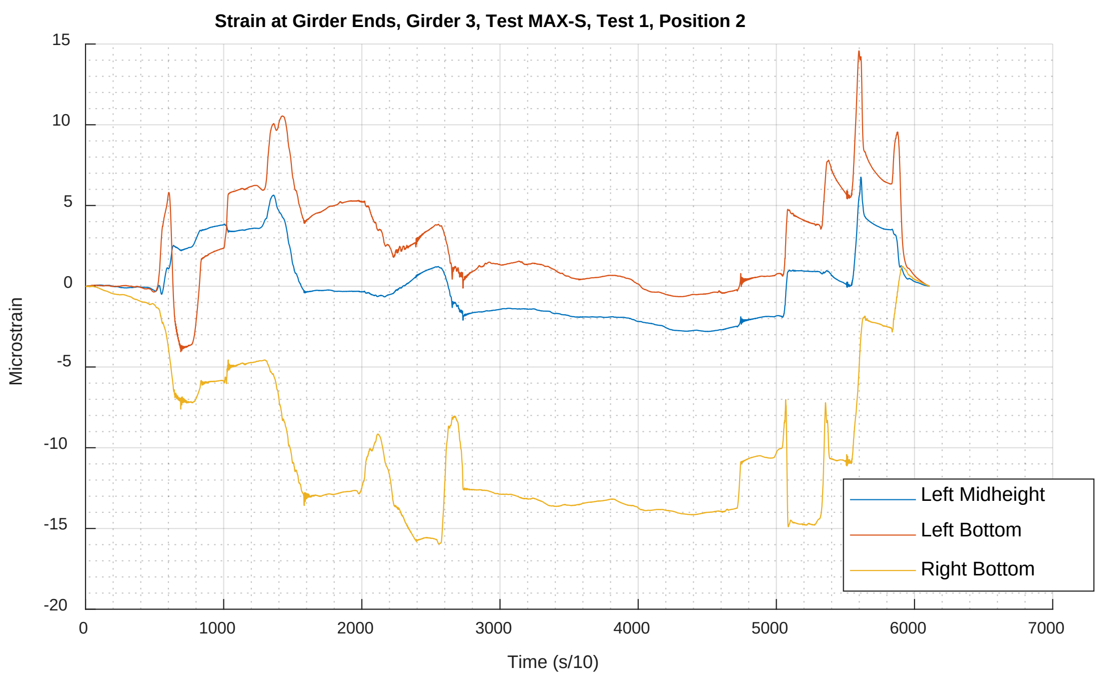

Figure 9: Bridge 3848 – MAX_S_2_1, center girder strains at ends

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 Bridge 5489, A.3.5 contains calculations for Bridge 5109, A.4.5 contains calculations for Bridge 2390, A.5.5 contains calculations for Bridge 2879. 2130, and A.6.5 contains calculations for Bridge 3848.

1.4.2 Bridge Characteristics

Material properties and general bridge geometry (i.e. span length, girder section properties, and reinforcement layout and geometry) were required for calculations. Geometric parameters were taken from each bridge's most recent available rating report and were verified in the field when accessible. Material properties were assumed based on the bridges' ages as 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 were 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

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 impact as applicable and including lane load. Girder moment was calculated based on this load and the AASHTO Distribution Factors calculated as described in section 1.4.3 of this report.

1.4.5 AASHTO Rating Factor

Flexural rating factors were independently 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. Where present, integral concrete wearing surfaces and integral curbs were assumed to contribute to interior and exterior girders' moment capacities respectively. 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

$\varphi_s = 0.85\ to\ 1.0\ inclusive$ 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 significant moment effects on the bridge. Continuous data recording was initiated, and then trucks were moved onto the bridge in a series. 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 the moment produced by the AASHTO HL-93 loading as described in section 1.4.4 of this report. Total moment applied during testing was determined based on the measured magnitude of truck wheel loads and the positions of wheels measured during testing.

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 of 2.5 ksi specified by the AASHTO *Manual for Bridge Evaluation*. 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. Several studies of cast-in-place concrete structures of similar age have shown that concrete strengths can approach 8 ksi in older structures (Buckle et al. 1984, Saraf 1998, Alkhrdaji et al. 2001). 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. The assumption of a higher-than-nominal strength of 5 ksi is conservative, since it leads to a higher-than-nominal elastic modulus and therefore lower predicted strains.

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 when the recorded strains were reliable. The strains measured at the bottom of the slab were generally not used per BDI's recommendation against relying on very small measured strains, but were used when recorded strains in another sensor were deemed unreliable. In general, measured neutral axis locations tended to be consistent with either uncracked section behavior or fell between cracked and uncracked behavior ("partially cracked"). In only one case (an exterior girder from Bridge 2390) did a neutral axis depth inferred through recorded strains seem to indicate a cracked section. However, the strains recorded at the section's bottom were still significantly less than the strains predicted for an uncracked section and so the girder was assumed uncracked.

1.4.8 Distribution Factors Determined from Live Load Testing

The moment carried by each girder was then calculated as per Equation 3 assuming an uncracked section.

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

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

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 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. In all cases, the strains used corresponded to the test causing the greatest applied moment. Although the "MAX_2" tests were designed ideally apply the greatest moment of all of the test series, in some cases other tests caused greater moments to be applied and so those moments and strains were used.

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

2 Live Load Test Results

2.1 Levant No. 5489

The bridge in Levant, No. 3356 over Black Stream, is shown in Figure 10. Testing was conducted on July 31, 2018 with a maximum applied moment producing 79.5% of HL-93 moment loading with impact. The moment rating factors based on the AASHTO *LRFD Design Manual* and *Manual for Bridge Evaluation* are 0.784 and 1.88 for the interior and exterior girders respectively. Table 2 shows the maximum measured strains for this bridge under typical two-truck and four-truck loading cases. The strains recorded with trucks positioned relative to the skew centerline consistently resulted in higher values of recorded strain than for load cases positioned relative to the perpendicular centerline. For this reason, these values were reported for Bridge 5489 along with all other bridges. Where two values of strain are reported, the first value is the recorded strain, which was determined to be unreliable and inaccurate due to its magnitude being grossly inconsistent with that of other similarly loaded girders and other strains measured over the section depth. The second value of strain was calculated using the other strains recorded in the same section and assuming linear strain distribution.

Assuming the conservative concrete compressive strength of 5 ksi, the strains recorded indicate the sections remained uncracked. This is supported by the observed neutral axis depths, which are consistently lower in the section than would be predicted for an uncracked section, as can be seen in Table 3. The high level of applied load and low recorded strains allowed interior and exterior girder rating factors to be increased to 1.10 and 2.36 respectively.

The live load distribution factors determined from the measured strains and those calculated per AASHTO are shown in in Table 4, and indicate that the AASHTO distribution factors are quite conservative. The distribution factor inferred for each girder was reduced by a minimum of 27% with respect to AASHTO calculated distribution factors for both two-truck and four-truck load cases. As shown in Table 2, strain measured at the ends of the girders indicate that the central girder and one of the exterior girders experienced a small amount of unintended fixity as evidenced by the negative strains recorded near the abutments. Original design drawings indicate the presence of dowel bars attaching one abutment to the superstructure. These, along with friction between the superstructure and opposing abutment may contribute to this small, apparent fixity.

Figure 10: Bridge 5489 general condition

Table 2: Bridge 5489 strains recorded from tests SBS_S_2_1 and MAX_S_2_1 with corrections noted

Girder	*Location*	SBS_S_2_1			MAX_S_2_1		
		Midspan	*Abutment 1*	*Abutment 2*	*Midspan*	*Abutment 1*	*Abutment 2*
		με	*με*	*με*	*με*	*με*	*με*
1	**Top**	-3.61	-	-	-5.50	-	-
	Center	9.85	-	-	16.8	-	-
	Bottom	11.5 / 23.3	-1.75	2.69	17.2 / 39.0	-3.64	5.10
2	**Top**	-6.90	-	-	-10.3	-	-
	Center	6.05 / 21.3	-	-	8.38 / 30.2	-	-
	Bottom	45.6	-	-	70.7	-	-
3	**Top**	-7.22	-	-	-16.9	-	-
	Center	20.1	-3.66	-	32.5	-7.22	-
	Bottom	21.9 / 47.4	0.537	-14.0	34.5 / 81.8	-5.92	-17.3
4	**Top**	-1.48	-	-	-12.3	-	-
	Center	21.1	-	-	34.2	-	-
	Bottom	14.7 / 43.6	-	-	24.5 / 80.6	-	-
5	**Top**	-5.30	-	-	-8.64	-	-
	Center	10.9	0.167	-	18.8	-0.892	-
	Bottom	-0.00 / 27.0	0.428	-2.68	-0.00 / 46.2	-17.3	-1.13

Table 3: Bridge 5489 neutral axis depths

Girder	Uncracked NA Depth (in)	Cracked NA Depth (in)	Measured NA Depth (in)
1	23.8	17.2	26.7
2	15.7	13.0	35.7
3	15.7	13.0	25.3
4	15.7	13.0	26.5
5	23.8	17.2	25.7

Table 4: Bridge 5489 distribution factors

Girder	AASHTO DF	SBS_S_2_1		MAX_S_2_1	
		Measured DF	% Difference	Measured DF	% Difference
1	0.483	0.271	-55.1%	0.272	-43.7%
2	0.685	0.498	-27.3%	0.426	-37.8%
3	0.685	0.477	-30.4%	0.493	-28.0%
4	0.685	0.438	-36.1%	0.486	-29.1%
5	0.483	0.314	-35.0%	0.322	-33.3%

2.2 Hampden No. 5109

The bridge in Hampden, No. 5109 over Souadabscook Stream, is shown in Figure 11. Testing was conducted on August 2, 2018 with maximum applied moment producing 91.8% of HL-93 flexural load with impact. Strains recorded during testing are presented in Table 5. Where two values of strain are reported, the first value is the recorded strain, which was determined to be unreliable and inaccurate due to its magnitude being grossly inconsistent with that of other similarly loaded girders and other strains measured over the section depth. The second value of strain was calculated using the other strains recorded in the same section and assuming linear strain distribution. By comparing the recorded strains at the bottom of the girders it was determined that none of the girders had experienced significant flexural cracking throughout their service live and did not crack during testing. Further evidence for uncracked behavior is provided by the measured neutral axis depths presented in Table 5 which show that for each of the girders the inferred neutral axis depths were well below those expected for an uncracked section.

The rating factors computed based on the AASHTO *LRFD Design Manual* and *Manual for Bridge Evaluation* are 0.686 and 1.59 for the interior and exterior girders respectively. Through testing, the interior and exterior rating factors were able to be increased to 0.942 and 3.78. It should be noted that in the initial calculation of girder capacity, the wearing surface and curbs were included despite their replacement. Design drawings for the replacement indicated that the new curbs would be anchored to the exterior girders with grouted rebar and that the new concrete wearing surface would be bonded to the deck. These specifications justified the assumption of composite action.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 7. As is apparent, the AASHTO predicted distribution factors are conservative. This conservatism is greatest for the exterior girders with decreasing conservatism as toward the center girder. From the strains reported in Table 5 near the girder ends, it can be seen that some unintended fixity was experienced in the central girder. This is evidenced by the negative strains recorded at the girder's bottom. Original design drawings indicate that dowel bars were specified to connect interior girders with the Western abutment. These dowel bars are likely the source of some of this apparent fixity.

Figure 11: Bridge 5109 general condition

Table 5: Bridge 5109 strains from tests SBS_S_2_1 and MAX_S_2_1 with corrections

Girder	***Location***	**SBS_S_2_1**			**MAX_S_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
		με	***με***	***με***	***με***	***με***	***με***
1	**Top**	-0.083	-	-	-0.638	-	-
	Center	8.04	-	-	14.3	-	-
	Bottom	20.7	-4.10	-0.01	34.5	-3.92	1.57
2	**Top**	7.09	-	-	-1.59	-	-
	Center	7.80 / 27.7	-	-	11.2 / 33.7	-	-
	Bottom	48.2	-	-	68.9	-	-
3	**Top**	-6.78	-	-	-3.75	-	-
	Center	21.9	-4.79	-	30.9	-12.6	-
	Bottom	57.1	-10.7	-14.4	90.5	-22.8	-19.3
4	**Top**	-1.57	-	-	0.270	-	-
	Center	0 / 20.8	-	-	0 / 35.8	-	-
	Bottom	43.2	-	-	71.3	-	-
5	**Top**	-0.588	-	-	-0.361	-	-
	Center	2.52	0.603	-	4.10	-3.57	-
	Bottom	7.10	0.207	-	11.2	-5.40	-

Table 6: Bridge 5109 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	24.1	15.5	28.7
2	19.5	11.8	34.3
3	19.5	11.8	25.4
4	19.5	11.8	33.4
5	24.1	15.5	26.4

Table 7: Bridge 5109 distribution factors from recorded strains

Girder	**AASHTO DF**	**Two Trucks**		**Four Trucks**	
		Measured DF	**% Difference**	**Measured DF**	**% Difference**
1	0.506	0.283	-44.1%	0.300	-40.7%
2	0.686	0.526	-23.3%	0.479	-30.2%
3	0.686	0.623	-9.18%	0.629	-8.31%
4	0.686	0.471	-31.3%	0.495	-27.8%
5	0.506	0.093	-81.6%	0.097	-80.8%

2.3 Unity No. 2390

The bridge in Unity, No. 2390 over the Sandy Stream, is shown in Figure 12. Testing was conducted on August 7, 2018 with maximum applied moment producing 93.2% of HL-93 moment with impact. This was the largest percentage of HL-93 moment with impact applied to any of the bridges tested. This led to relatively large recorded strains, as shown in Table 8. Where two values of strain are reported, the first value is the recorded strain, which was determined to be unreliable and inaccurate due to its magnitude being grossly inconsistent with that of other similarly loaded girders and other strains measured over the section depth. The second value of strain was calculated using the other strains recorded in the same section and assuming linear strain distribution. Rating factors determined per AASHTO equaled 0.757 and 1.05 for interior and exterior girders respectively.

In contrast to other bridges investigated, some of the neutral axis depths inferred from recorded strains indicate either partially or fully cracked behavior, as seen in Table 9. However, the strains recorded at midspan at the sections' bottoms were still below those expected for an uncracked section, suggesting that the sections indeed behaved as though they remained uncracked. Because of this behavior, the interior and exterior rating factors could be increased to 0.838 and 1.15 respectively. A contributing factor to this bridge's low rating factors is the very thick (~5 in.) asphalt overlay. The thickness of this overlay is seen in Figure 13 which shows a drainage opening. This layer could not be assumed to add to the section's capacity and so only added additional dead load.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 10. These results suggest that AASHTO's distribution factors are conservative for exterior girders and non-central interior girders, but are relatively accurate for the central girder. This is true for both two-truck and four-truck load cases. The strains recorded in Table 8 indicate that significant fixity was experienced in the central girder. This is evidenced by the relatively large negative strains recorded at the bottom of this girder near the abutments. This unintended fixity is likely due in part to dowel bars specified in the original design drawings which attach the interior girders to the West abutment.

Figure 12: Bridge 2390 general condition

Figure 13: Thick asphalt overlay

Table 8: Bridge 2390 strains from tests SBS_S_2_1 and MAX_S_2_1 with corrections

Girder	***Location***	**SBS_S_2_1**			**MAX_2_S_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
		με	*με*	*με*	*με*	*με*	*με*
1	**Top**	0.920	-	-	-1.98	-	-
	Center	8.89	-	-	15.4	-	-
	Bottom	29.9	0.355	-4.11	50.4	-10.9	-19.6
2	**Top**	-5.77	-	-	-12.0	-	-
	Center	19.3	-	-	24.7	-	-
	Bottom	66.1	-	-	83.1	-	-
3	**Top**	-3.76	-	-	-4.65	-	-
	Center	34.5	-4.68	-	41.3	-5.96	-
	Bottom	97.5	-20.9	-29.2	117	-26.7	-36.8
4	**Top**	-9.21	-	-	-8.96	-	-
	Center	32.5	-	-	40.8	-	-
	Bottom	22.9 / 74.2	-	-	30.3 / 90.6	-	-
5	**Top**	-0.067	-	-	-0.923	-	-
	Center	14.4	-	-	25.6	-	-
	Bottom	9.31 / 28.9	-0.009	-	13.4 / 52.2	4.54	-

Table 9: Bridge 2390 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	22.1	13.9	16.9
2	14.5	8.60	17.1
3	14.5	8.60	15.5
4	14.5	8.60	12.0
5	22.1	13.9	10.2

Table 10: Bridge 2390 distribution factors

Girder	**AASHTO DF**	**SBS_S_2_1**		**MAX_S_2_1**	
		Measured DF	**% Difference**	**Measured DF**	**% Difference**
1	0.428	0.196	-54.2%	0.250	-41.9%
2	0.635	0.449	-29.3%	0.427	-32.8%
3	0.635	0.662	4.25%	0.599	-5.67%
4	0.635	0.504	-20.6%	0.465	-26.8%
5	0.428	0.190	-55.6%	0.259	-39.5%

2.4 Atkinson No. 2879

The bridge in Atkinson, No. 2879 over the Piscataquis River, is shown in Figure 14. Testing occurred on August 9, 2018 with maximum applied moment producing 92.4% of HL-93 live load with impact. This bridge was unique in that it had only four girders and four simple spans. Only the Eastern, interior span was tested and so results may or may not be applicable to other spans. Rating factors determined per AASHTO were 1.09 and 2.57 for interior and exterior girders respectively, making it the only bridge investigated with an operating rating factor above 1.0. The strains recorded during testing are presented in Table 11 for two-truck and four-truck loadings. Intuitively, it would appear that the strains presented at midspan at the bottom of girders 3 and 4 have been switched, with the reading of one being valid for the other and vice-versa. However, no definitive evidence was found to support this and so it was assumed that the recorded strains were correct. Regardless, recorded strains were consistently lower than predicted for an uncracked section, suggesting that the section behaved as uncracked. This is further evidenced by the inferred neutral axis depths shown in Table 12, which show that inferred neutral axis depths were close to or below predicted neutral axis locations for uncracked sections. These conditions allowed for interior and exterior rating factors to be increased to 1.35 and 2.76 respectively.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 13. These distribution factors were lower than those predicted by AASHTO, but to a smaller degree than was seen on other bridges. This suggests that AASHTO may be less conservative for bridges with four girders rather than five. From the consistently positive girder end strains reported in Table 11, no unintended fixity was measured during testing for this particular span.

Figure 14: Bridge 2879 general condition

Table 11: Bridge 2879 strains from tests SBS_S_2_1 and MAX_S_2_1

		SBS_S_2_1			**MAX_S_2_1**		
		Midspan	***Abutment 1***	***Abutment 2***	***Midspan***	***Abutment 1***	***Abutment 2***
Girder	***Location***	***με***	***με***	***με***	***με***	***με***	***με***
1	**Top**	-6.40	-	-	-8.76	-	-
	Center	7.38	-	-	14.4	-	-
	Bottom	34.6	6.28	3.46	56.4	8.77	7.67
2	**Top**	-8.84	-	-	-11.6	-	-
	Center	16.2	0.136	-	24.5	-0.858	-
	Bottom	45.5	9.19	1.49	65.5	8.16	8.37
3	**Top**	-5.32	-	-	-9.65	-	-
	Center	17.9	4.73	-	27.6	2.70	-
	Bottom	35.8	6.89	-0.103	54.9	5.69	7.73
4	**Top**	-4.04	-	-	-7.29	-	-
	Center	8.20	-	-	15.0	-	-
	Bottom	39.9	7.96	-	65.5	11.0	-

Table 12: Bridge 2879 neutral axis depths

Girder	**Uncracked NA Depth (in)**	**Cracked NA Depth (in)**	**Measured NA Depth (in)**
1	28.5	18.3	28.2
2	21.4	13.2	33.5
3	21.4	13.2	42.2
4	28.5	18.3	27.2

Table 13: Bridge 2879 distribution factors

		SBS_S_S_2_1		**MAX_S_S_2_1**	
Girder	**AASHTO DF**	**Measured DF**	**% Difference**	**Measured DF**	**% Difference**
1	0.498	0.377	-24.3%	0.397	-20.3%
2	0.701	0.662	-5.56%	0.621	-11.4%
3	0.701	0.526	-25.0%	0.521	-25.7%
4	0.498	0.435	-12.7%	0.461	-7.43%

2.5 Columbia No. 3848

The bridge in Columbia, No. 3848 over Western Little Stream, is shown in Figure 15. Testing occurred on August 28, 2018 with maximum applied moment producing 80.9% of HL-93 load with impact. Rating factors determined per AASHTO equaled 0.887 and 1.41 for interior and exterior girders respectively. Strains measured during two-truck and four-truck load cases are given in Table 14. Where two values of strain are reported, the first value is the recorded strain, which was determined to be unreliable and inaccurate due to its magnitude being grossly inconsistent with that of other similarly loaded girders and other strains measured over the section depth. The second value of strain was calculated using the other strains recorded in the same section and assuming linear strain distribution. Strains measured at girder bottoms were consistently smaller than would be predicted with an uncracked section, suggesting the girders behaved as uncracked. This behavior is supported by the inferred neutral axis depths, which indicate uncracked behavior for all girders as seen in Table 15. Based on these conditions, the interior and exterior flexural rating factors could be increased to 1.15 and 2.20 respectively.

The live load distribution factors determined per AASHTO as well as those experimentally determined from measured strains are given in Table 16. Unexpectedly, significantly more load was distributed to one of the non-central interior girders (girder 4) than to other interior girders. The reason for this anomaly is not immediately apparent, however the strain recorded was still smaller than was expected for an uncracked section and so can be neglected. A small amount of fixity was experienced in the central girder, as is shown by the negative strains reported in Table 14. This is likely due to dowel bars, which were designed to connect the interior girders with one of the abutments.

Figure 15: Bridge 3848 general condition

Table 14: Bridge 3848 strains from tests SBS_S_2_1 and MAX_S_2_1

Girder	Location	SBS_S_2_1			MAX_S_2_1		
		Midspan	Abutment 1	Abutment 2	Midspan	Abutment 1	Abutment 2
		$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$	$\mu\varepsilon$
1	Top	0.503	-	-	-2.54	-	-
	Center	9.94	-	-	22.1	-	-
	Bottom	28.4	1.22	5.96	53.6	6.16	8.78
2	Top	-7.58	-	-	-7.66	-	-
	Center	21.5	-	-	31.2	-	-
	Bottom	55.9	-	-	74.2	-	-
3	Top	-5.67	-	-	-5.98	-	-
	Center	27.5	-3.19	-	36.2	-1.83	-
	Bottom	53.0	-8.60	-14.7	72.2	0.600	-13.6
4	Top	-5.51	-	-	-4.76	-	-
	Center	-0.00 / 32.5	-	-	-0.00 / 36.5	-	-
	Bottom	70.6	-	-	77.8	-	-
5	Top	-0.132	-	-	-0.262	-	-
	Center	12.5	-	-	16.0	-	-
	Bottom	30.3	12.9	-	39.0	19.5	-

Table 15: Bridge 3848 neutral axis depths

Girder	Uncracked NA Depth (in)	Cracked NA Depth (in)	Measured NA Depth (in)
1	19.8	12.7	20.9
2	12.9	7.6	20.4
3	12.9	7.6	25.9
4	12.9	7.6	25.4
5	19.8	12.7	20.8

Table 16: Franklin No. 3307 distribution factors

Girder	AASHTO DF	SBS_S_2_1		MAX_S_2_1	
		Measured DF	% Difference	Measured DF	% Difference
1	0.431	0.239	-44.5%	0.339	-21.3%
2	0.611	0.469	-23.2%	0.468	-23.4%
3	0.611	0.444	-27.3%	0.455	-25.5%
4	0.611	0.593	-2.95%	0.491	-19.6%
5	0.431	0.255	-40.8%	0.247	-42.7%

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 above 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 nearly always conservative. The maximum differences between values inferred from the tests and values computed per AASHTO were seen for exterior girders in general for each of the bridges. Assuming a conservative concrete compressive strength of 5 ksi, all bridges exhibited uncracked behavior under maximum applied moment. This was generally supported by the calculated neutral axis depths which were often much lower in the section than computed for an uncracked section.

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 28.3%, with minimum and maximum increases of 10.7% and 40.2% 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. The controlling operating flexural rating factor could be increased to 1.0 or greater for HL-93 loading with impact for Bridges 5489, 2879, and 3848, indicating that they are sufficient for such loading. The controlling rating factors for Bridges 5109 and 2390 were unable to be raised above 1.0, using the noted conservative assumptions.

4 References

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4. Buckle, I.G., Dickson, A.R., & Phillips, M.H. (1984). Ultimate strength of three concrete highway bridges. *Canadian Journal of Civil Engineering.* 12(63-72).

5. Saraf, V.K. (1998). Evaluation of existing RC slab bridges. *Journal of Evaluation of Constructed Facilities*. 12(1).

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

A Comma Separated Variable (.csv) file is provided for each bridge 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 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 Levant No. 5489

A.2.1 Experimental Configuration and Experimental Data Collected

Table 17: Bridge 5489 experimental configuration and experimental data collected

File Contents	File Name	File Type
Sensors	Br5489 _Sensors.csv	CSV Format
Sensor Layout	Br5489 _SensorLayout.mat	MATLAB Data File
Sensor Data	Br5489_ALT_S_2_1_Strain.mat	MATLAB Data File
	Br5489_ALT_U_2_1_Strain.mat	MATLAB Data File
	Br5489_MAX_S_1_1_Strain.mat	MATLAB Data File
	Br5489_MAX_S_2_1_Strain.mat	MATLAB Data File
	Br5489_MAX_S_3_1_Strain.mat	MATLAB Data File
	Br5489_MAX_U_2_1_Strain.mat	MATLAB Data File
	Br5489_SBS_S_2_1_Strain.mat	MATLAB Data File
	Br5489_SBS_U_2_2_Strain.mat	MATLAB Data File

A.2.2 Instrumentation

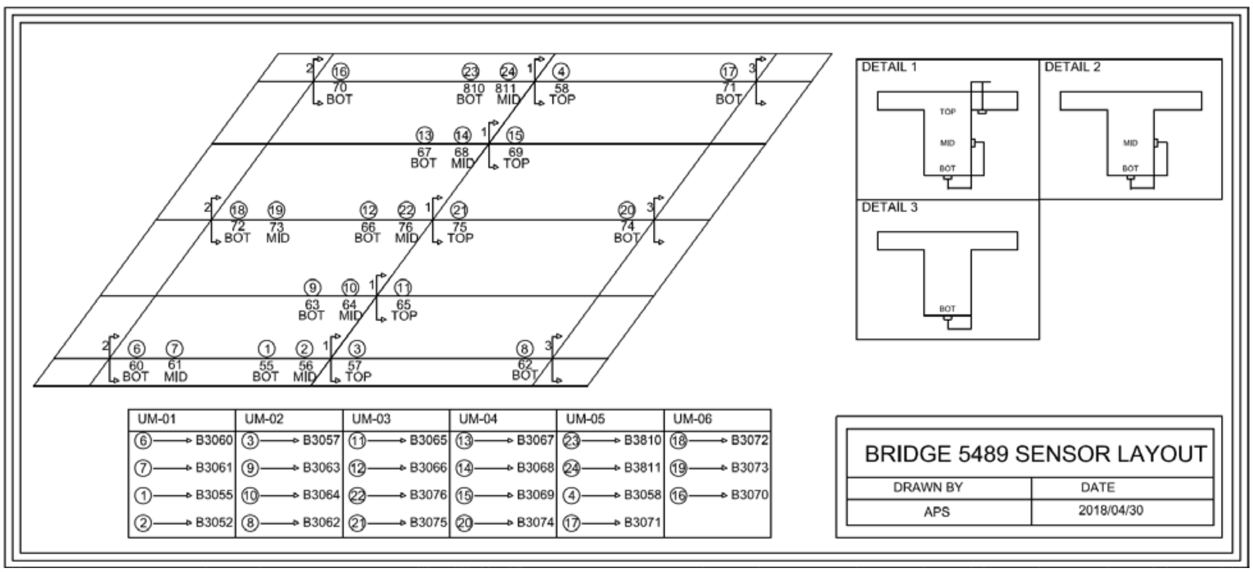

Figure 16: Bridge 5489 sensor layout

A.2.3 Loading

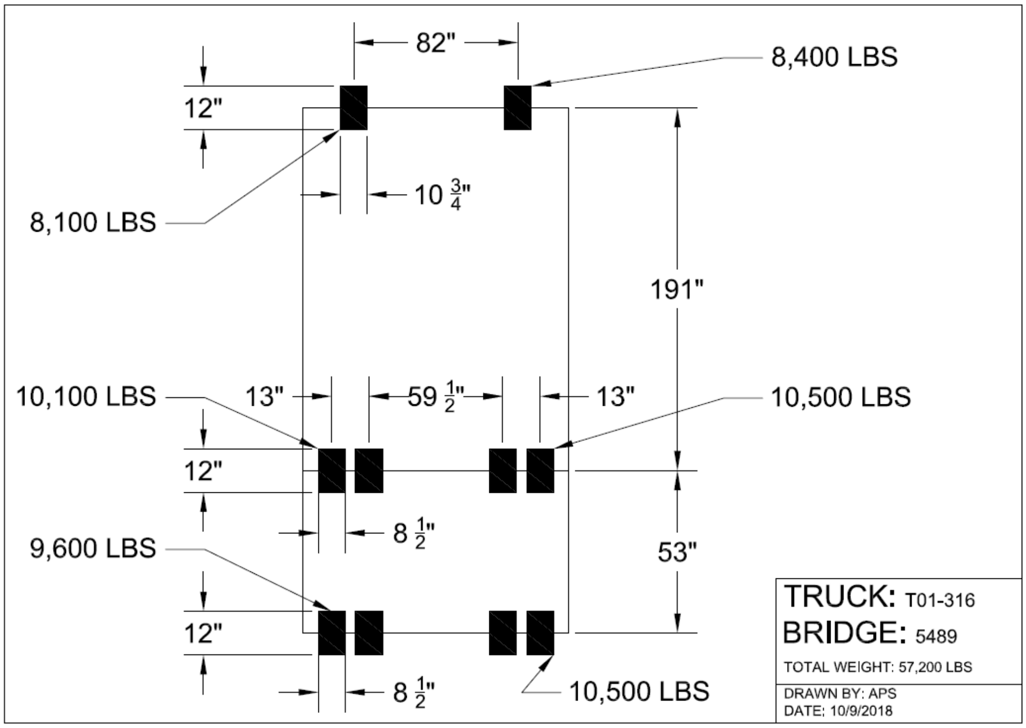

Figure 17: Bridge 5489 Truck T01-316 loading

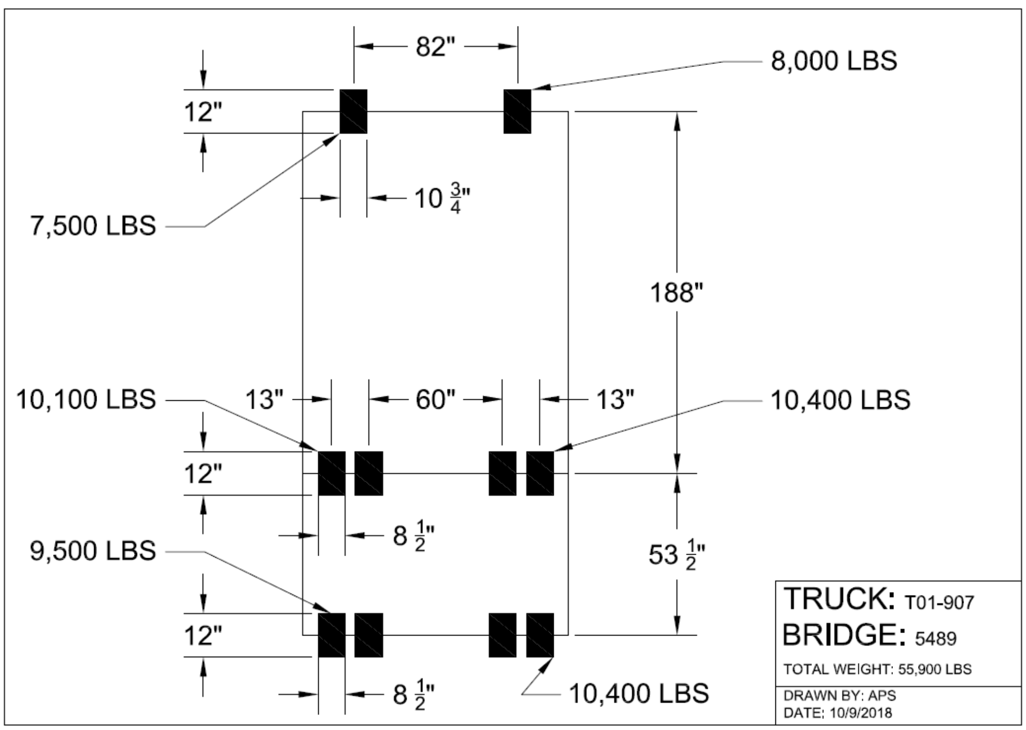

Figure 18: Bridge 5489 Truck T01-907 loading

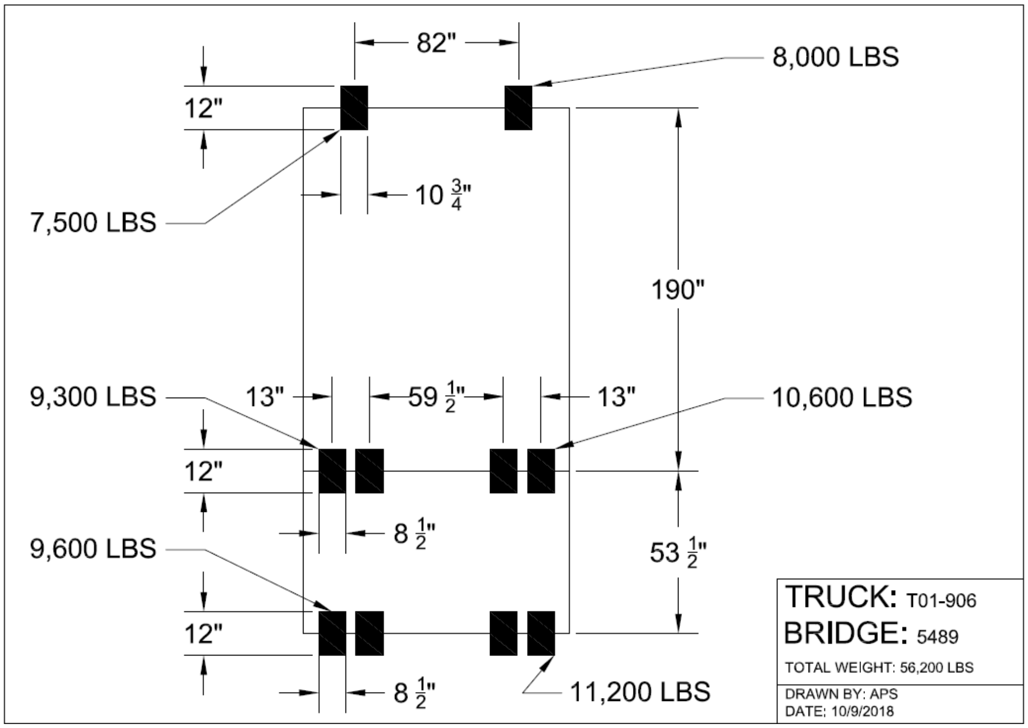

Figure 19: Bridge 5489 Truck T01-906 loading

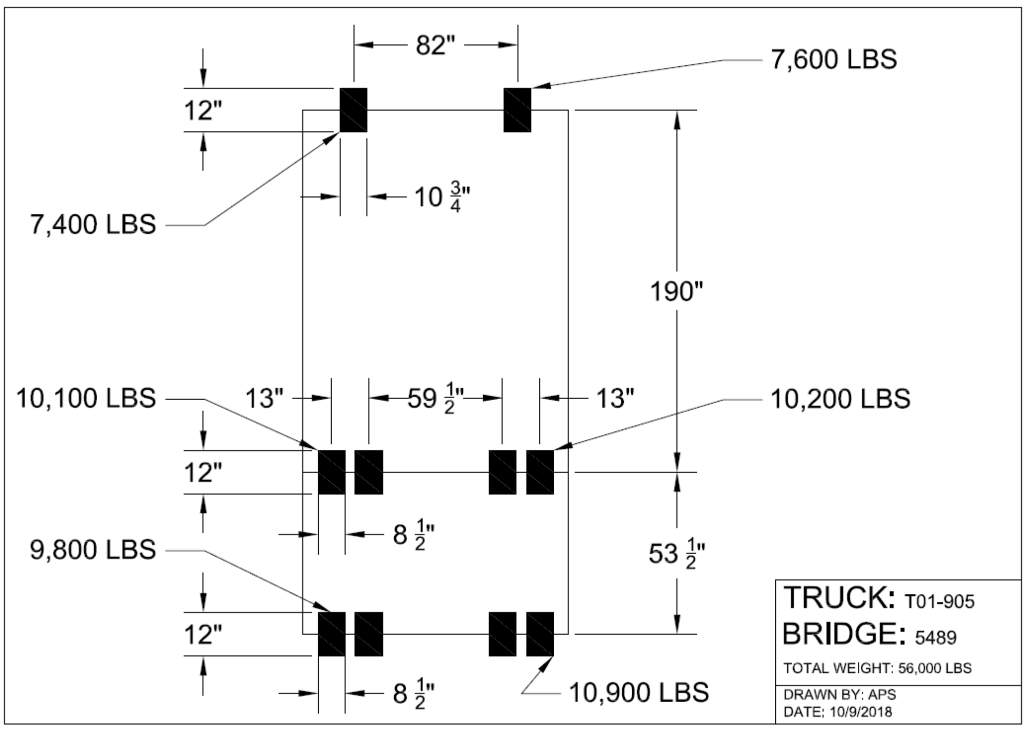

Figure 20: Bridge 5489 Truck T01-904 loading

A.2.4 Representative Data Plots

Figure 21: Bridge 5489 SBS_S_2_1 strains - Midspan

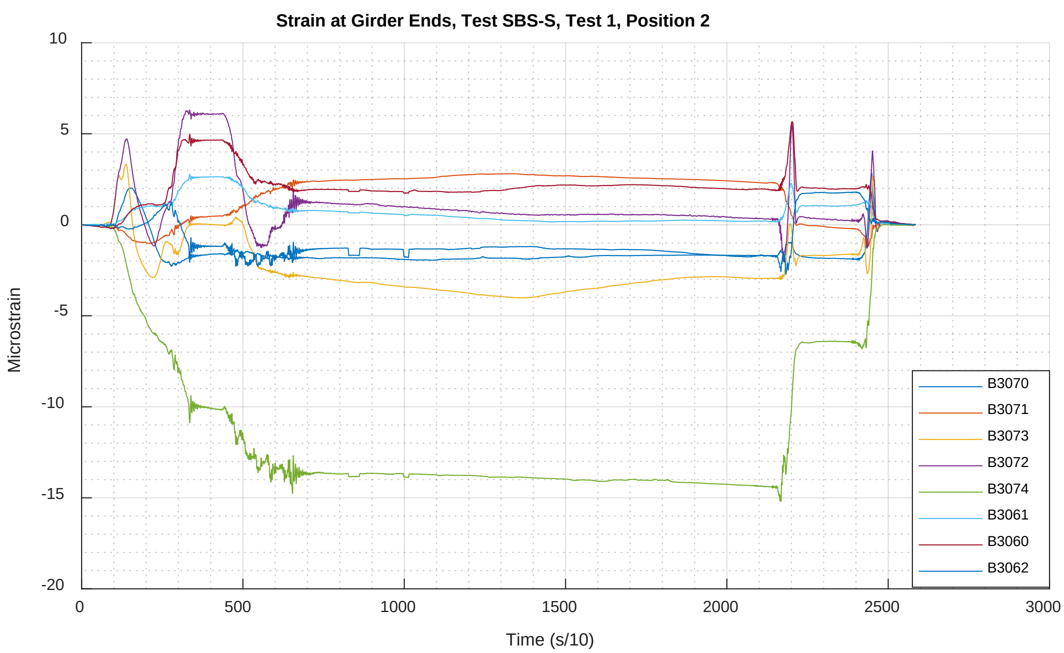

Figure 22: Bridge 5489 SBS_S_2_1 strains - Ends

Figure 23: Bridge 5489 SBS_U_2_2 strains - Midspan

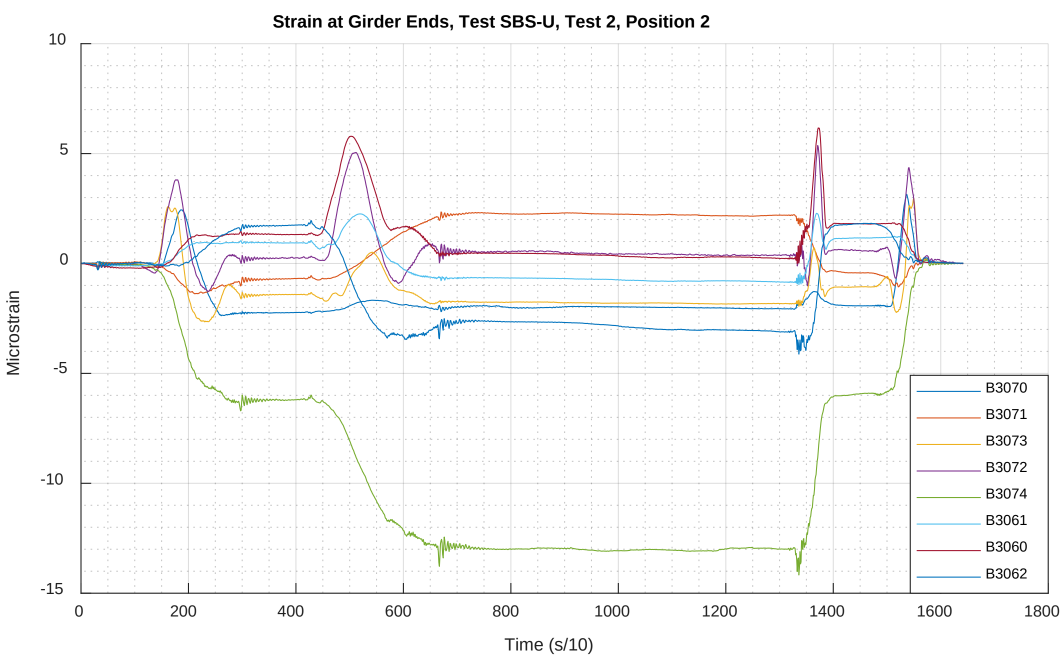

Figure 24: Bridge 5489 SBS_U_2_2 strains - Ends

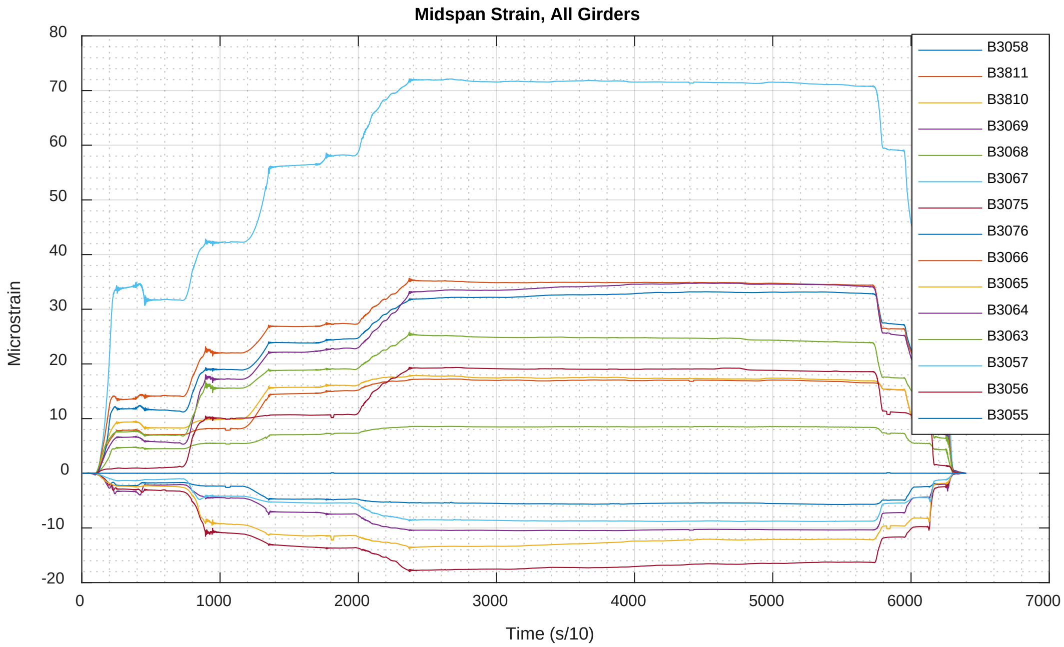

Figure 25: Bridge 5489 MAX_S_2_1 strains - Midspan

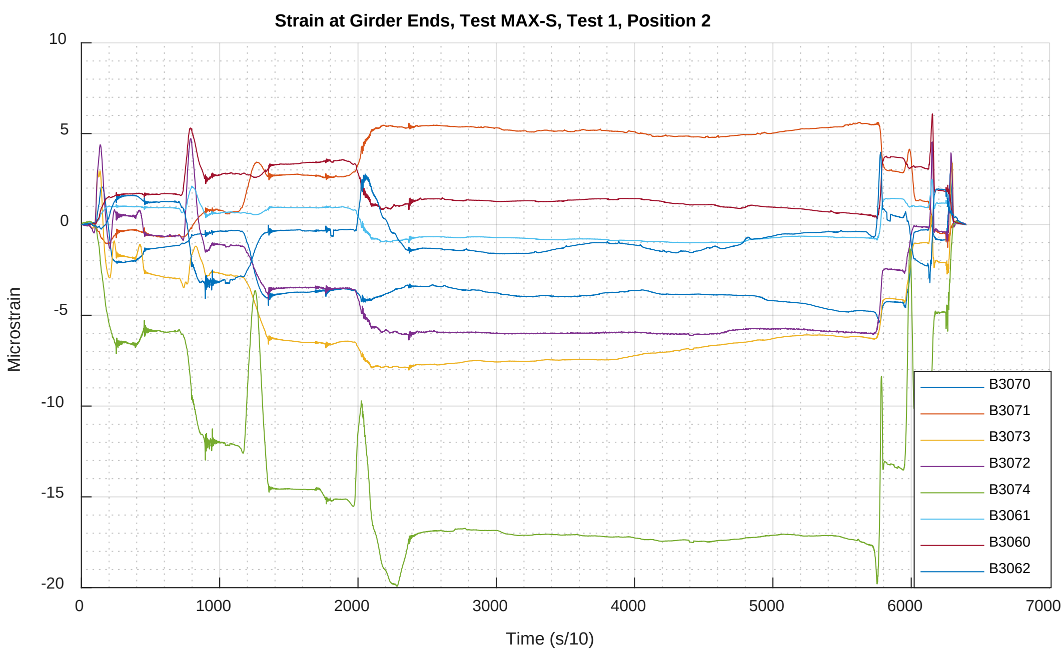

Figure 26: Bridge 5489 MAX_2_1 strains – Ends

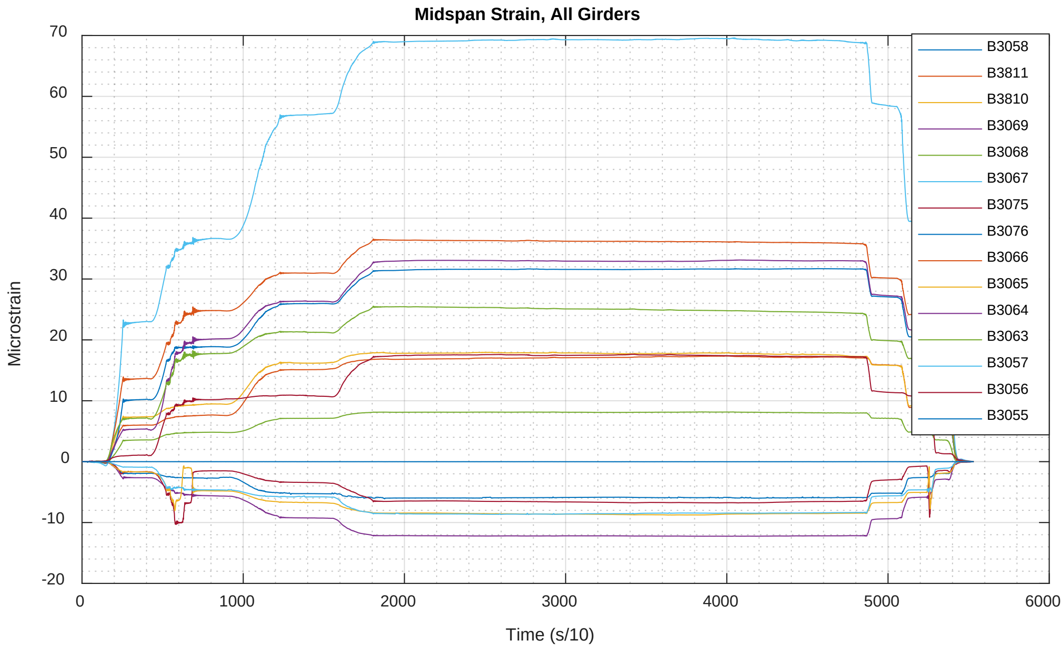

Figure 27: Bridge 5489 MAX_U_2_1 strains – Midspan

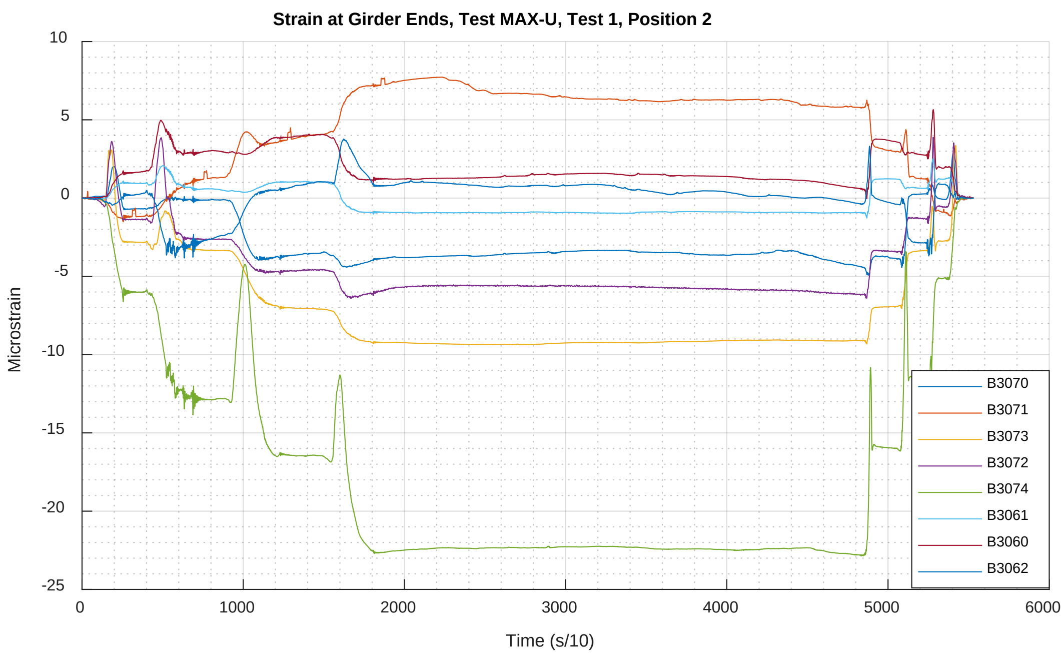

Figure 28: Bridge 5489 MAX_U_2_1 strains – Ends

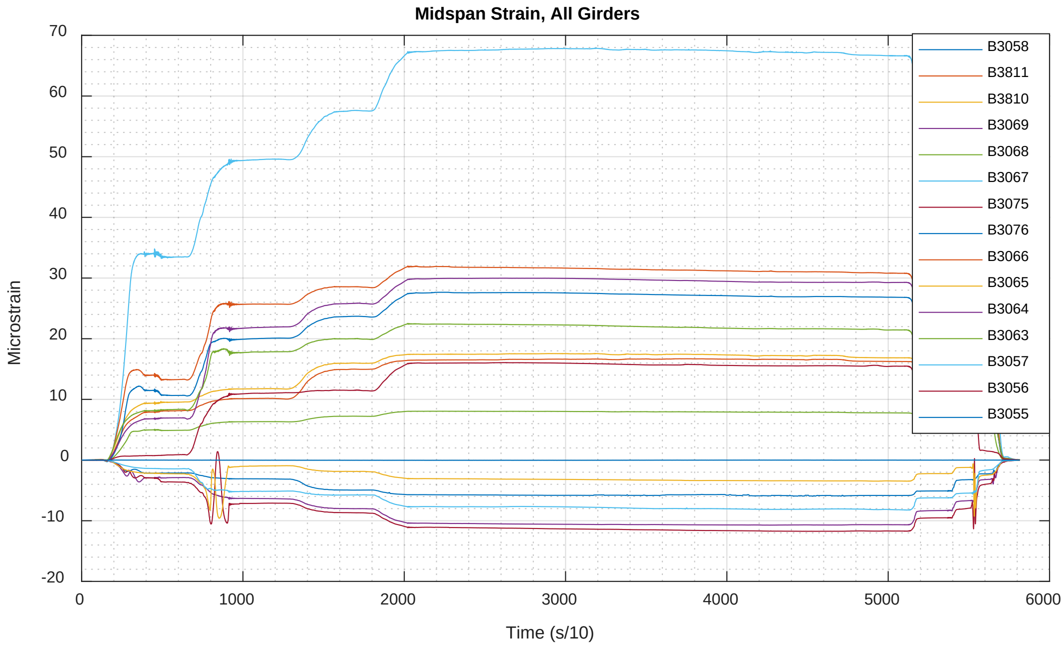

Figure 29: Bridge 5489 ALT_S _2_1 strains – Midspan

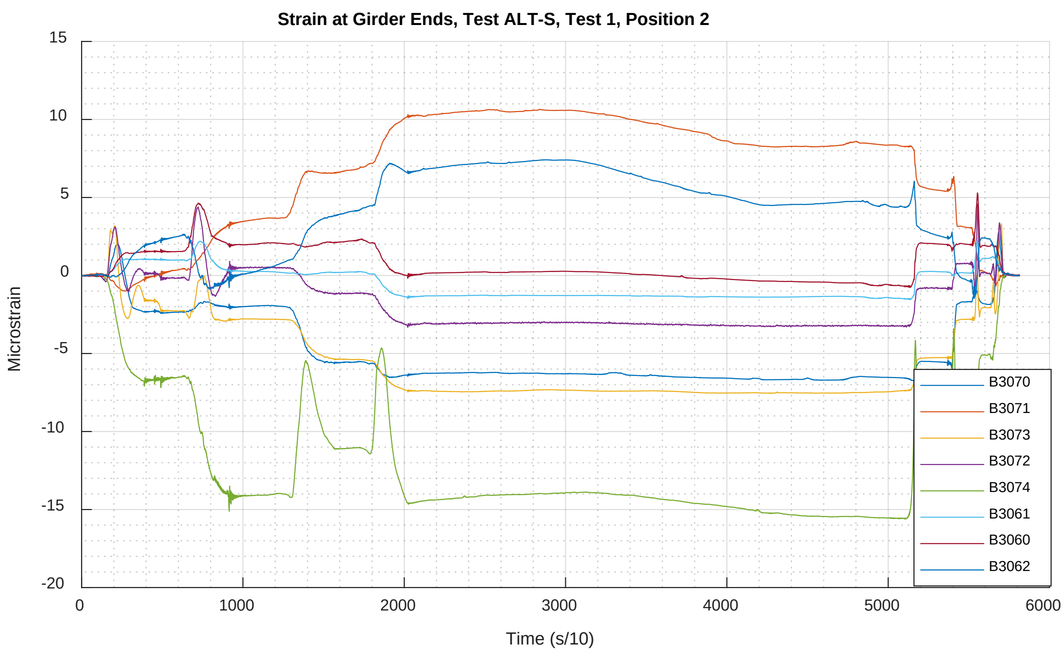

Figure 30: Bridge 5489 ALT_S_2_1 strains – Ends

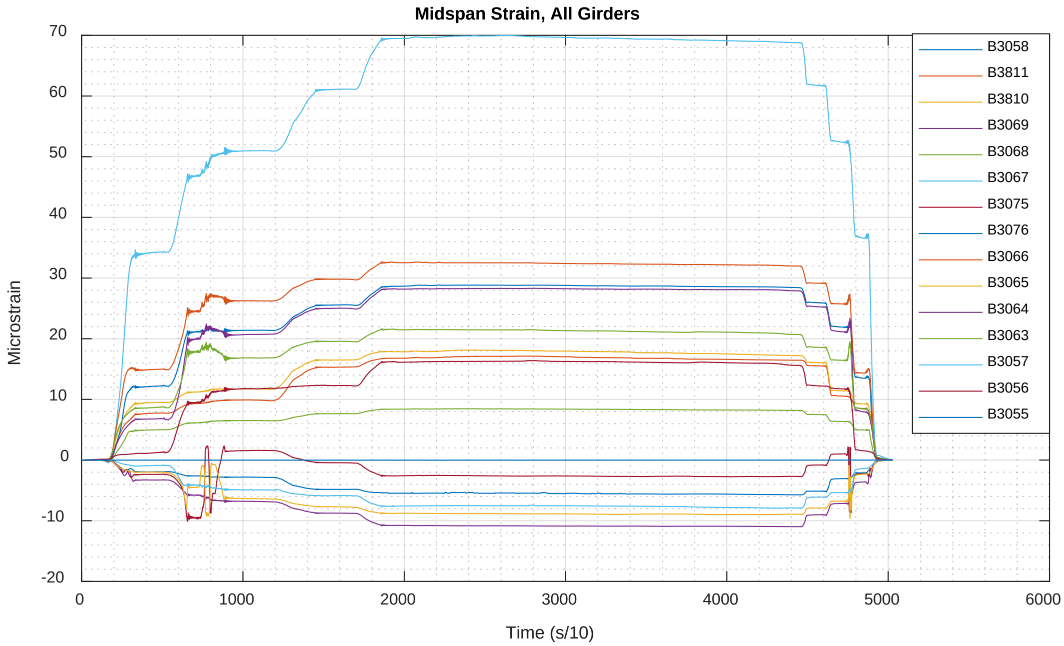

Figure 31: Bridge 5489 ALT_U_2_1 strains – Midspan

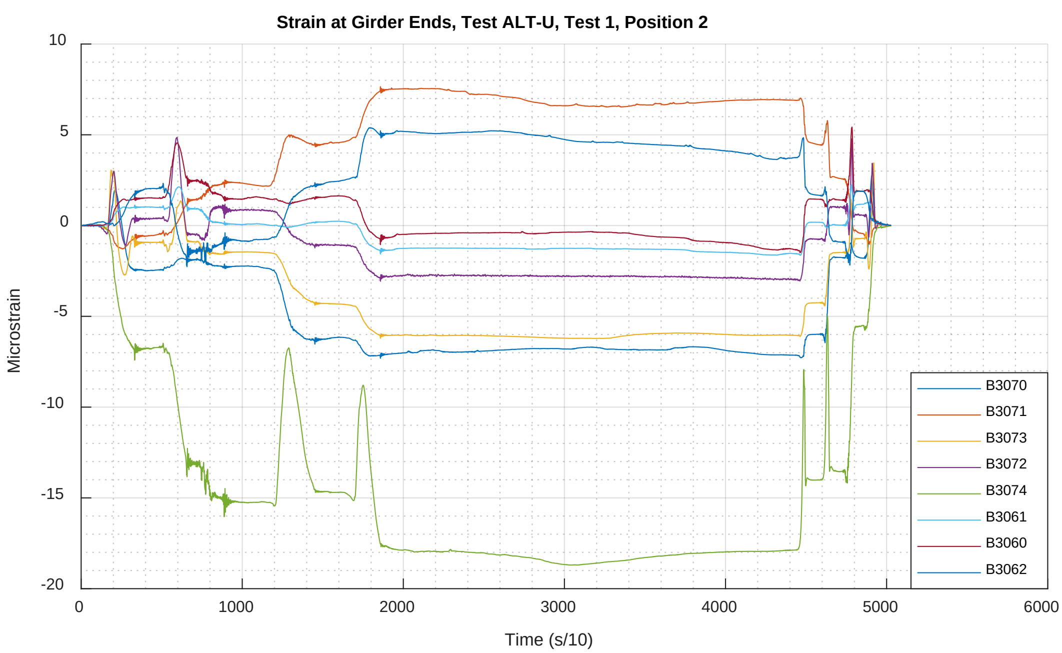

Figure 32: Bridge 5489 ALT_U_2_1 strains – Ends

A.2.5 Rating Factor Calculations

AASHTO Rating Calculations:

Bridge 5489 - Levant, Maine

Material Parameters:

Concrete Compressive Strength	$f'_c := 2.5\ ksi$
Reinforcement Yield Strength	$F_y := 33\ ksi$
Unit Weight: Reinforced Concrete	$\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$
Unit Weight: Wearing Surface	$\gamma_{ws} := 0.150\ \frac{kip}{ft^3}$

Geometric Properties:

Span Length	$L := 47\ ft$
Girder Spacing - Interior	$S := 82\ in$
Girder Spacing - Exterior	$S_x := 54\ in$
Number of Girders	$NG := 5$
Skew Angle	$skew := 15\ °$
Lane Width	$lanewidth := 13\ ft$
Number of Lanes	$Nlane := 2$
Wearing Surface Thickness	$ws := 3\ in$
Thickness of Pavement Overlay	$ws_2 := 0\ in$
Girder Height - Interior	$h := 36\ in$
Girder Height - Exterior	$h_x := 36\ in$
Deck Thickness	$d_s := 5.5\ in$
Web Width - Interior	$b_w := 19\ in$
Web Width - Exterior	$b_{wx} := 19\ in$
Curb Depth	$h_{curb} := 12\ in$
Curb Width	$b_{curb} := 20\ in$
Height to Centroid of Reinforcement - Interior	$y_{bar} := \begin{bmatrix} 4.893 \\ 6.077 \\ 8.27 \\ 6.077 \\ 4.893 \end{bmatrix} in$
Height to Centroid of Reinforcement - Exterior	$y_{barx} := \begin{bmatrix} 4.893 \\ 6.077 \\ 8.27 \\ 6.077 \\ 4.893 \end{bmatrix} in$
Area of Reinforcement - Interior	$A_s := \begin{bmatrix} 12.016 \\ 15.142 \\ 18.268 \\ 15.142 \\ 12.016 \end{bmatrix} in^2$

Area of Reinforcement - Exterior	$A_{sx} := \begin{bmatrix} 12.016 \\ 15.142 \\ 18.268 \\ 15.142 \\ 12.016 \end{bmatrix} in^2$
Distance from Centerline of Girder to Edge of Curb	$d_e := -7\ in$
Eccentricity of Centerline of Girders w.r.t. Centerline of Roadway	$exc := 0\ in$

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder	$P_{dint} := 1.99\ kip$
Location of Intermediate Diaphragm (Half, Third, Quarter)	$loc_d :=$ "Half"
Distributed Load Due to Rail	$w_{rail} := 0.328\ \frac{kip}{ft}$
Structural Dead Load Factor	$\gamma_{DC} := 1.25$
Wearing Surface Dead Load Factor	$\gamma_{DW} := 1.25$
Live Load Factor	$\gamma_{LL} := 1.35$
Live Load Impact Factor	$IM := 0.33$
Flexural Resistance Factor	$\phi := .9$
System Factor	$\phi_s := 1.0$
Condition Factor	$\phi_c := 1.0$

Initial Calculations

Web Height - Interior $\quad d_g := h - d_s$

Web Height - Exterior $\quad d_{gx} := h_x - d_s$

Include Wearing Surface in Section Height $\quad h := h + \mathbf{if}\left(\gamma_{ws} = 0.15\ \frac{kip}{ft^3}, ws, 0\right) = 39\ in$

Depth to Centroid of Reinforcement - Interior $\quad d := h - y_{bar}$

Depth to Centroid of Reinforcement - Exterior $\quad d_x := h_x - y_{barx} + h_{curb}$

Moment Applied to Interor Girders from Diaphragm

$$M_d := \begin{cases} P_{dint} \cdot \frac{L}{4} & \text{if } loc_d = \text{"Half"} \\ P_{dint} \cdot \frac{L}{3} & \text{else if } loc_d = \text{"Third"} \\ P_{dint} \cdot \frac{L}{4} + P_{dint} \cdot \frac{L}{4} & \text{else if } loc_d = \text{"Quarter"} \end{cases} = 23.383\ ft \cdot kip$$

Moment Applied to Exterior Girders from Diaphragm $\quad M_{dx} := \frac{M_d}{2} = 11.691\ ft \cdot kip$

Distribution Factors

Dictance Between Centroids of Deck and Web $e_g := \frac{d_g + d_s}{2} = 18\ in$

Area of Web $A := d_g \cdot b_w = 579.5\ in^2$

Moment of Inertia of Web $I := \frac{b_w \cdot d_g^{\ 3}}{12} = \left(4.492 \cdot 10^4\right)\ in^4$

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

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(2.327 \cdot 10^5\right)\ in^4$

Interior Moment Distribution Factor - 1 Lane $g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.521$

Interior Moment Distribution Factor - 2 Lane $g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.686$

Controling Interior Moment Distribution Factor $g_m := \max\left(g_{m1}, g_{m2}\right)$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From C.G. of Girders $e_1 := \frac{W_r}{2} - 5\ ft + exc = 8\ ft$

Eccentricity of Exterior Girder From C.G. of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 13.667\ ft$

Eccentricity of Each Girder

$x_1 := X_{ext}$

$x_2 := X_{ext} - S$

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

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

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

Lever Rule Distribution Factor - One Lane $R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.434$

$g_{mR1} := \mathbf{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0.521$

Lever Rule Distribution Factor - Two Lanes $R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot \left(e_1 - 5\ ft\right)}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.488$

$g_{mR2} := \mathbf{if}\left(P_{dint} > 0, R_2, 0\right) = 0.488$

Exterior Moment Distribution Factor $g_{mx1} := \frac{1.2\ \left(S + d_e - 2\ ft\right)}{2 \cdot S} = 0.373$

$ee := 0.77 + \frac{d_e}{9.1\ ft} = 0.706$

$g_{mx2} := g_{m2} \cdot ee = 0.484$

$g_{mx} := \max\left(g_{mx1}, g_{mx2}\right) = 0.484$

Skew Correction Factor

$c_1 := 0.25 \cdot \left(\frac{K_g}{12 \cdot L \cdot d_s^{\,3}}\right)^{0.25} \cdot \left(\frac{S}{L}\right)^{.5} = 0.064$

$\theta := \text{if}\left(skew \geq 30°, skew, 0°\right)$

$C_\theta := 1 - c_1 \cdot \left(\tan(\theta)\right)^{1.5} = 1$

$g_m := g_m \cdot C_\theta = 0.686$

$g_{mx} := g_{mx} \cdot C_\theta = 0.484$

Interior DF	Exterior DF
$g_m = 0.686$	$g_{mx} = 0.484$

Loading

Interior Girder Dead Load	$w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.604\ \frac{kip}{ft}$
Deck Dead Load	$w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.47\ \frac{kip}{ft}$
Curb Dead Load	$w_{curb} := 2 \cdot \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.5\ \frac{kip}{ft}$
Dead Load from Nonstructural Components	$w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.428\ \frac{kip}{ft}$
Total Structural Dead Load on Interior Girders	$DC := w_{girder} + w_{deck} + w_{ns} = 1.501\ \frac{kip}{ft}$
Exterior Girder Dead Load	$w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.604\ \frac{kip}{ft}$
Exterior Deck Dead Load	$w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.309\ \frac{kip}{ft}$
Total Structural Dead Load on Exterior Girders	$DC_x := w_{girderx} + w_{deckx} + w_{ns} = 1.341\ \frac{kip}{ft}$
Wearing Surface Dead Load on Interior Girders	$DW := \gamma_{ws} \cdot \left(ws + ws_2\right) \cdot S = 0.256\ \frac{kip}{ft}$
Wearing Surface Dead Load on Exterior Girders	$DW_x := \gamma_{ws} \cdot \left(ws + ws_2\right) \cdot S_x = 0.169\ \frac{kip}{ft}$
Dead Load Moments	$M_{DC} := \frac{DC \cdot L^2}{8} + M_d \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} + M_{dx}$
	$M_{DW} := \frac{DW \cdot L^2}{8} \qquad M_{DWx} := \frac{DW_x \cdot L^2}{8}$

$M_{DC} = 437.967\ ft \cdot kip$ $M_{DCx} = 381.981\ ft \cdot kip$

$M_{DW} = 70.757\ ft \cdot kip$ $M_{DWx} = 46.596\ ft \cdot kip$

Live Load Moment - Truck Load $M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 566\ ft \cdot kip$

Live Load Moment - Tandem $M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 537.5\ ft \cdot kip$

Live Load Moment - Lane $M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 176.72\ ft \cdot kip$

Total HL-93 Live Load $M_{LL} := M_{Lane} + (1 + IM) \cdot \max(M_{Truck}, M_{Tandem})$

$M_{LL} = 929.5\ ft \cdot kip$

Nominal Resistance

Depth Whitney Stress Block - Interior $a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 2.276 \\ 2.868 \\ 3.46 \\ 2.868 \\ 2.276 \end{bmatrix} in$

Nominal Moment Resistance - Interior $M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 1.089 \cdot 10^3 \\ 1.311 \cdot 10^3 \\ 1.457 \cdot 10^3 \\ 1.311 \cdot 10^3 \\ 1.089 \cdot 10^3 \end{bmatrix} ft \cdot kip$

Interior Nominal Moment Capacity

$M_{capacity} := \max(M_n) = (1.457 \cdot 10^3)\ ft \cdot kip$

Depth Whitney Stress Block - Exterior $a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 3.456 \\ 4.355 \\ 5.254 \\ 4.355 \\ 3.456 \end{bmatrix} in$

Nominal Moment Resistance - Exterior $M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 1.367 \cdot 10^3 \\ 1.655 \cdot 10^3 \\ 1.864 \cdot 10^3 \\ 1.655 \cdot 10^3 \\ 1.367 \cdot 10^3 \end{bmatrix} (ft \cdot kip)$

Exterior Nominal Moment Capacity

$M_{capacityx} := \max(M_{nx}) = (1.864 \cdot 10^3)\ ft \cdot kip$

Rating Factors

Interior Moment Rating Factor

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

Exterior Moment Rating Factor

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

Interior	Exterior
$RF_{Interior} = 0.784$	$RF_{Exterior} = 1.879$

Rating Factor Improvements

Concrete Compressive Strength - Larger is More Conservative	$f'_c := 5\ ksi$
Concrte Elastic Modulus	$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = (4.07 \cdot 10^3)\ ksi$

Interior Girders

Maximum Recorded Strain	$\varepsilon_T := 87.2 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 738.7\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 9507\ in^3$
Cracked Section Modulus	$S_{cr} := 4565\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \mathbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right)$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.572 \cdot 10^{-4}$
Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.803$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.795$
Test Understanding Factor	$k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.401$
Improved Rating Factor	$RF_{Improved} := RF_{Interior} \cdot k = 1.099$

Exterior Girders

Maximum Recorded Strain	$\varepsilon_T := 63.5 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 738.7\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 11008\ in^3$
Cracked Section Modulus	$S_{cr} := 3469\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \mathbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right)$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 9.583 \cdot 10^{-5}$

Test Benefit Factor $k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.509$

Ratio of Applied to HL-93 Moment $r_M := \frac{M_{Max}}{M_{LL}} = 0.795$

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

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.255$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 2.357$

Improved Rating Factors

Interior	Exterior
$RF_{Improved} = 1.099$	$RF_{ImprovedExt} = 2.357$

Figure 33: Bridge 5489 calculations

A.3 Hampden No. 5109

A.3.1 Experimental Configuration and Experimental Data Collected

Table 18: Bridge 5109 experimental configuration and experimental data collected

File Contents	File Name	File Type
Sensors	Br5109_Sensors.csv	CSV Format
Sensor Layout	Br5109_SensorLayout.mat	MATLAB Data File
Sensor Data	Br5109_ALT_S_2_1_Strain.mat	MATLAB Data File
	Br5109_ALT_U_2_1_Strain.mat	MATLAB Data File
	Br5109_MAX_S_1_1_Strain.mat	MATLAB Data File
	Br5109_MAX_S_2_1_Strain.mat	MATLAB Data File
	Br5109_MAX_S_3_1_Strain.mat	MATLAB Data File
	Br5109_MAX_U_2_1_Strain.mat	MATLAB Data File
	Br5109_SBS_U_2_1_Strain.mat	MATLAB Data File
	Br_5432_MAX_1_2_Strain.mat	MATLAB Data File

A.3.2 Instrumentation

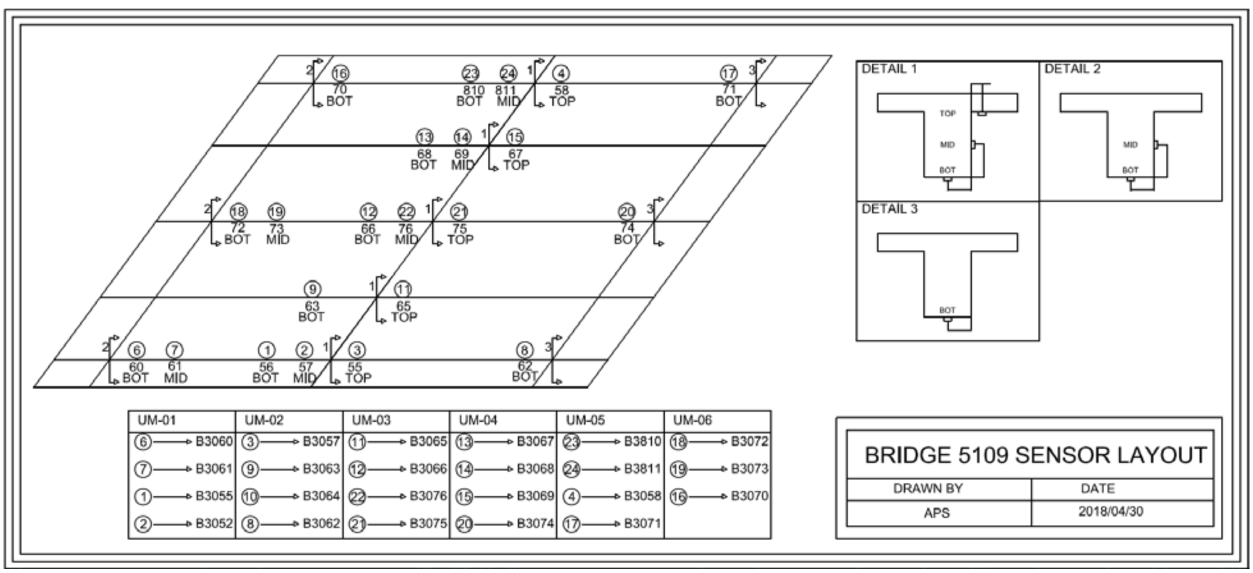

Figure 34: Bridge 5109 sensor layout

A.3.3 Loading

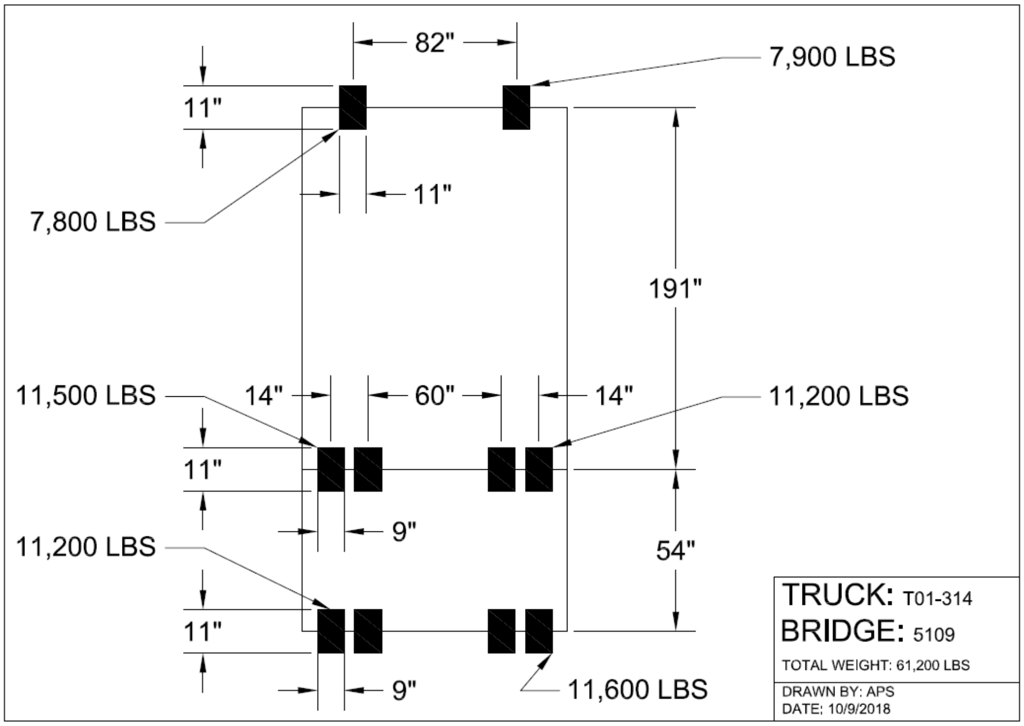

Figure 35: Bridge 5109 Truck T01-314 loading

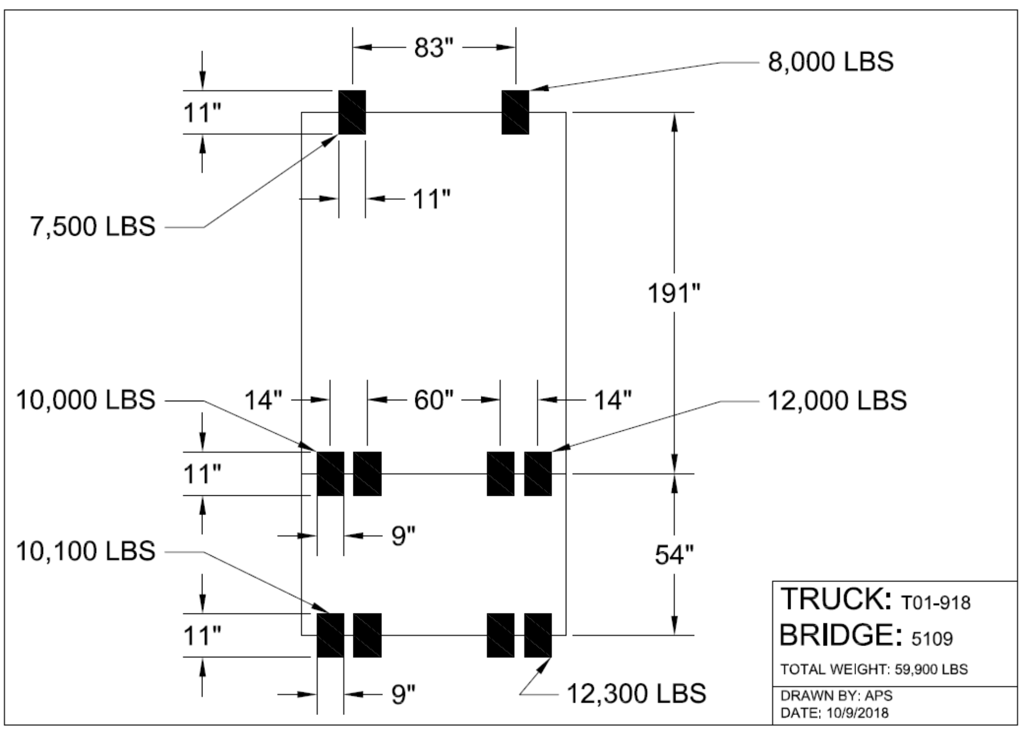

Figure 36: Bridge 5109 Truck T01-918 loading

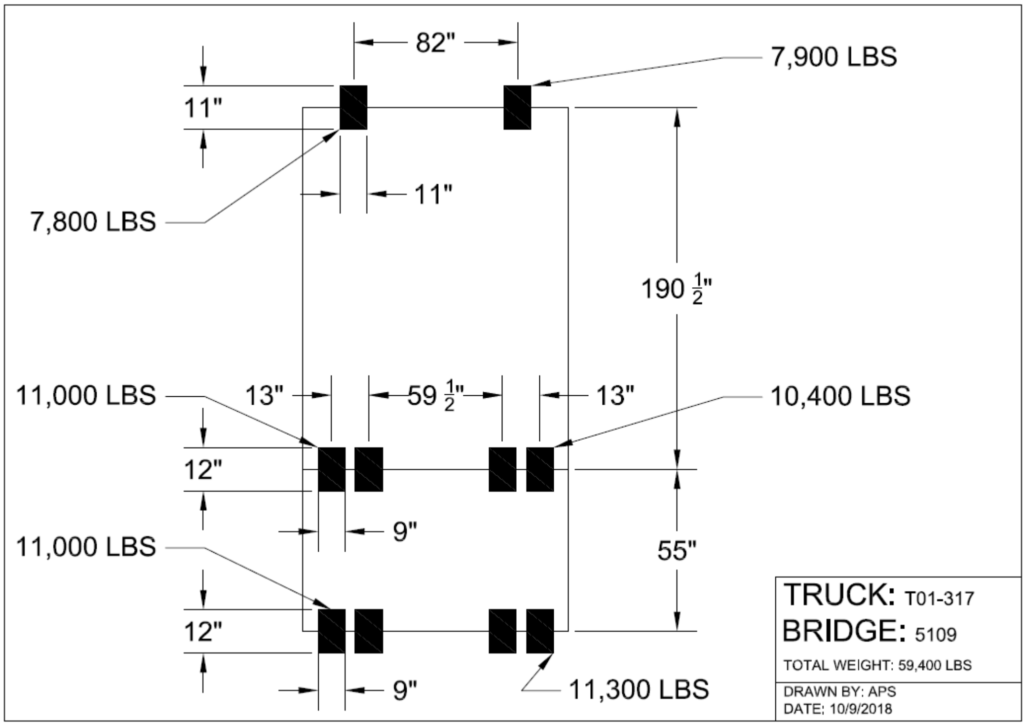

Figure 37: Bridge 5109 Truck T01-317 loading

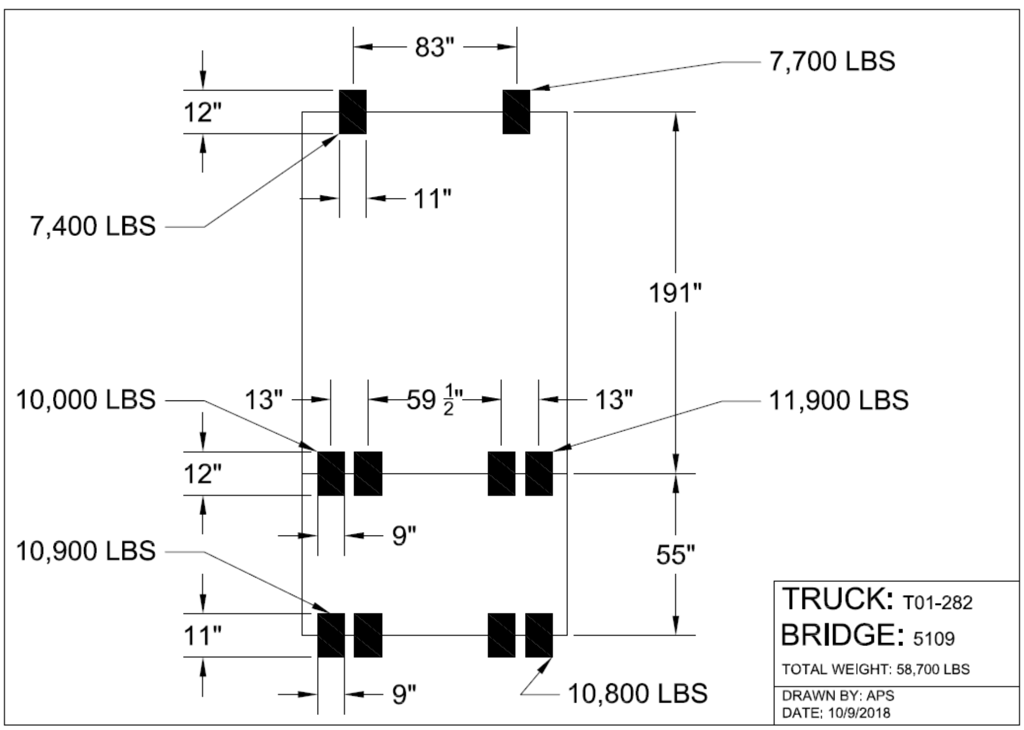

Figure 38: Bridge 5109 Truck T01-282 loading

A.3.4 Representative Data Plots

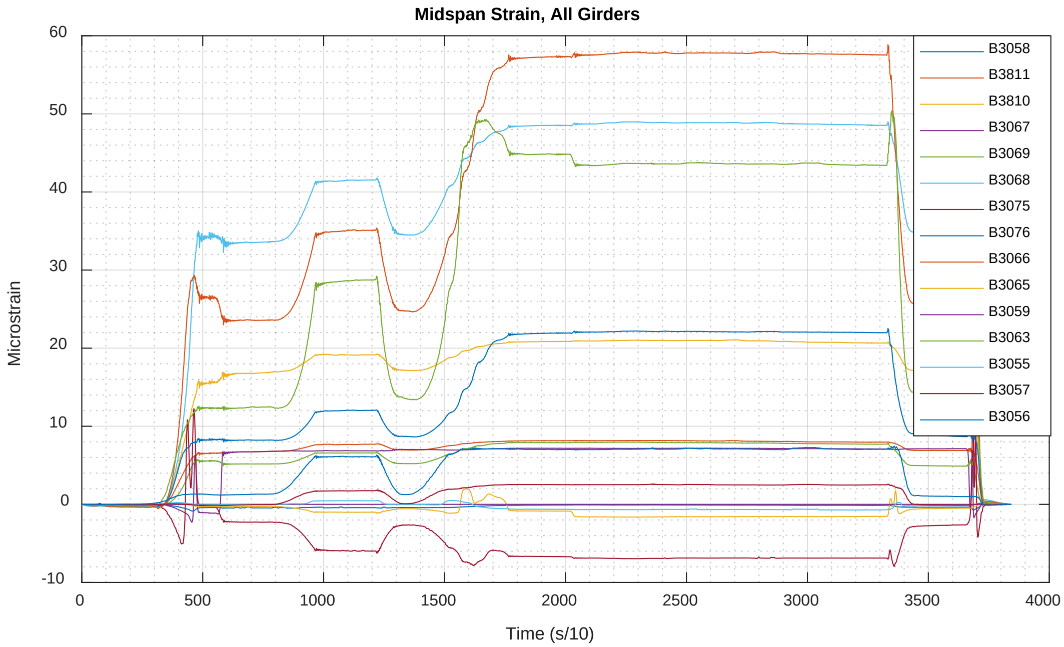

Figure 39: Bridge 5109 SBS_S_2_1 strains - Midspan

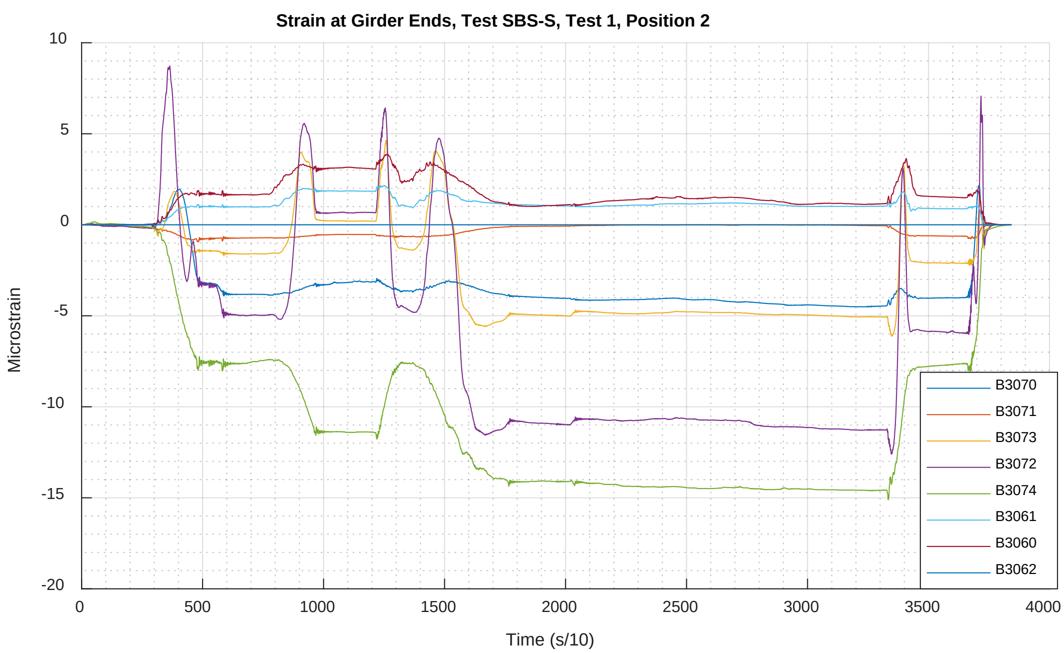

Figure 40: Bridge 5109 SBS_S_2_1 strains - Ends

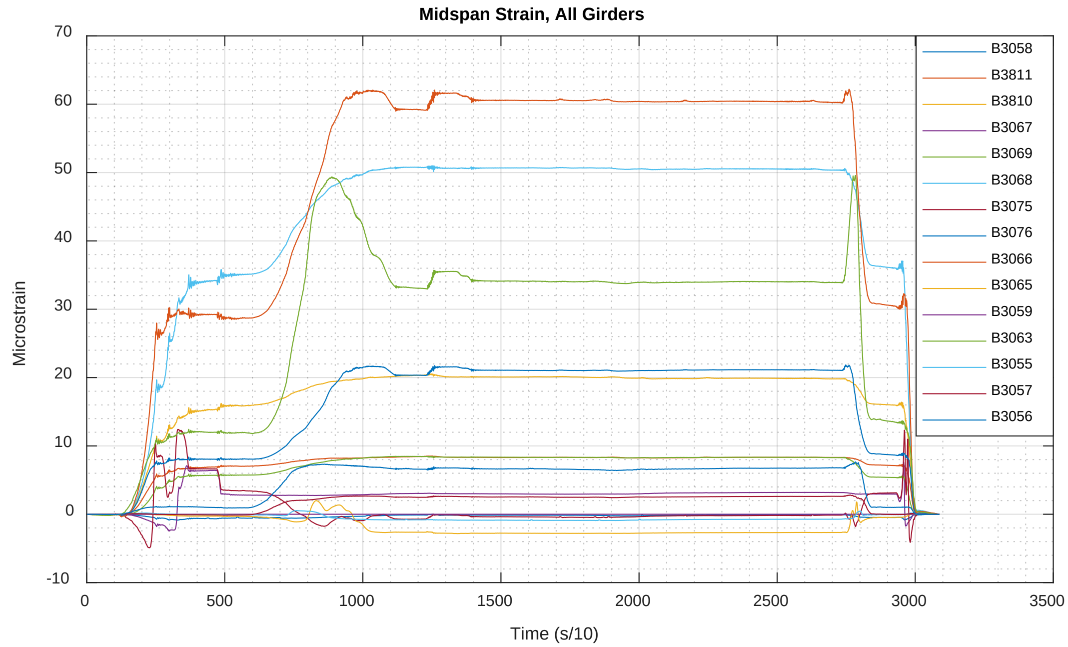

Figure 41: Bridge 5109 SBS_U_2_1 - Midspan

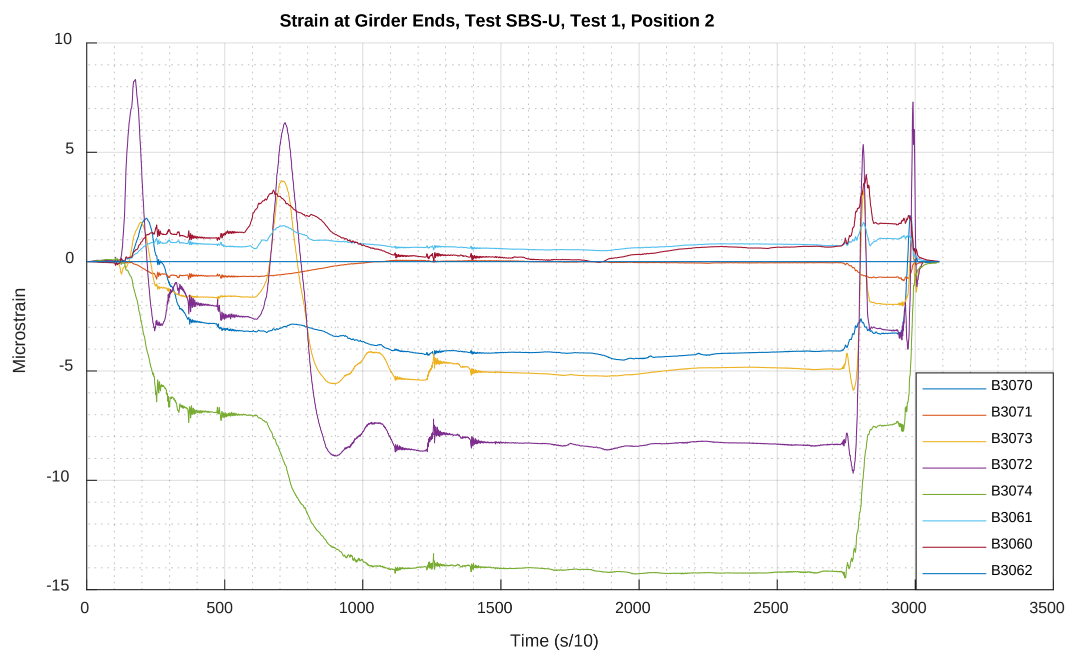

Figure 42: Bridge 5109 SBS_U_2_1 – Ends

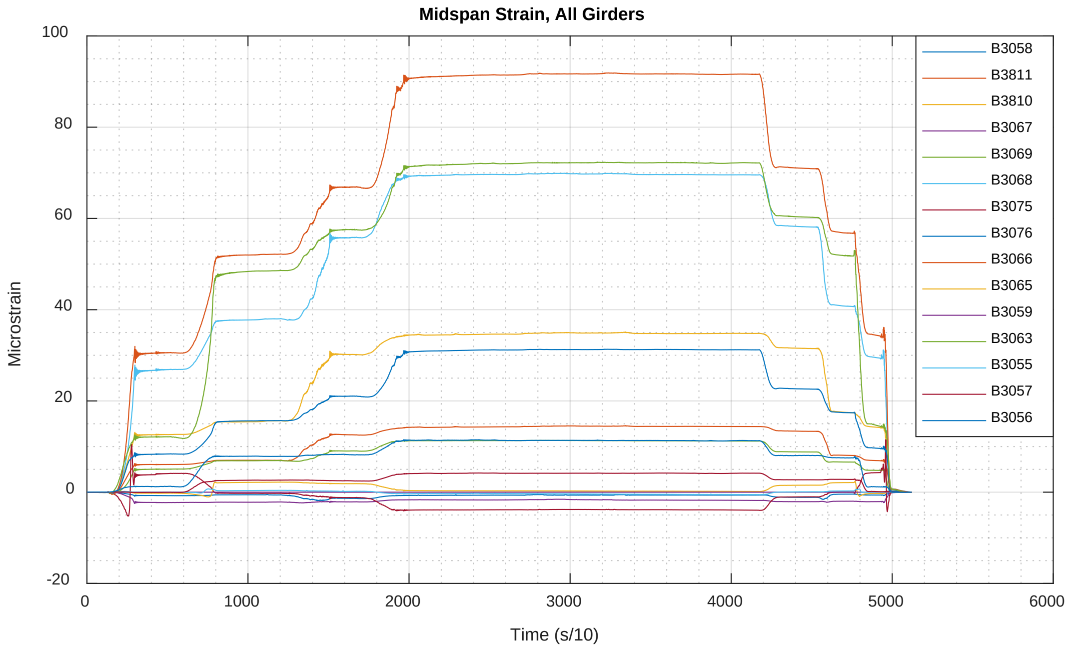

Figure 43: Bridge 5109 MAX_2_1 - Midspan

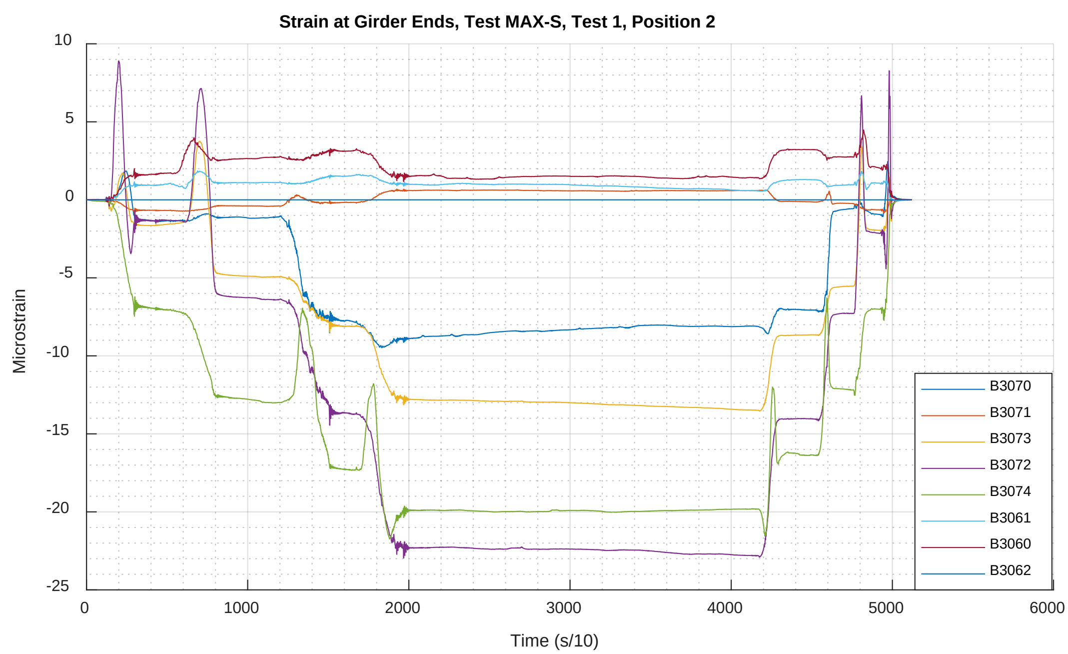

Figure 44: Bridge 5109 MAX_2_1 - Ends

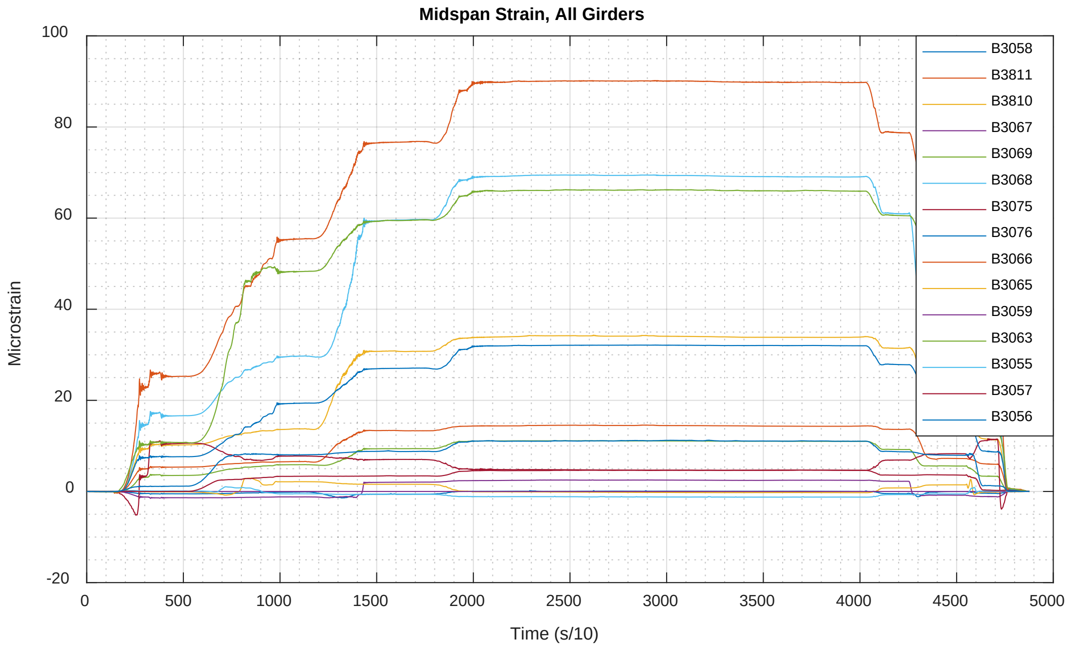

Figure 45: Bridge 5109 MAX_U_2_1 - Midspan

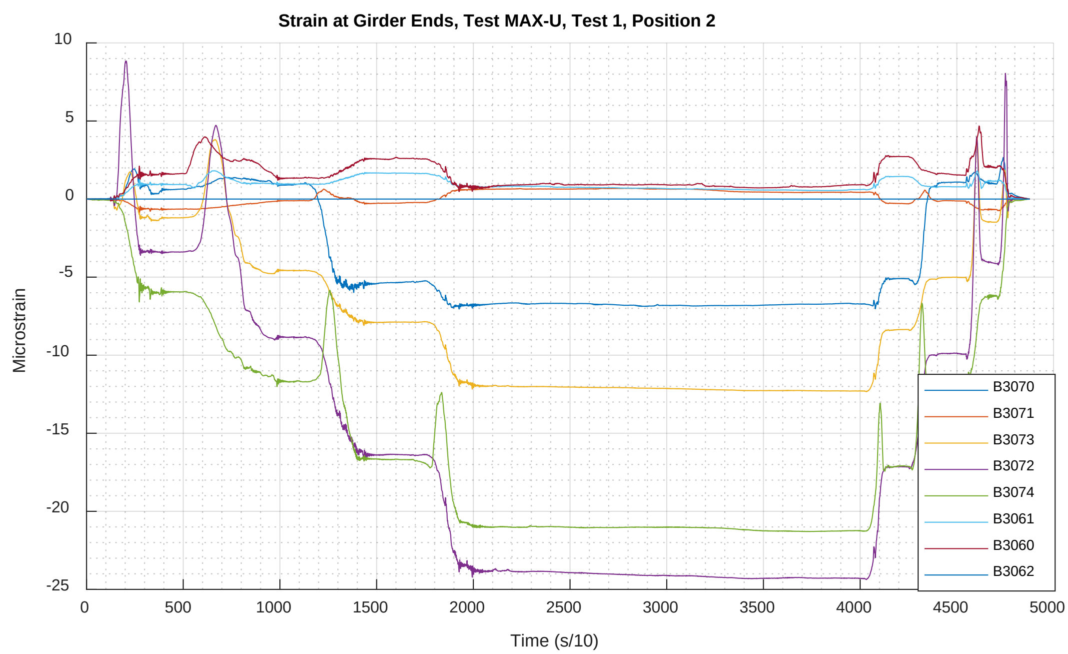

Figure 46: Bridge 5109 MAX_U_2_1 – Ends

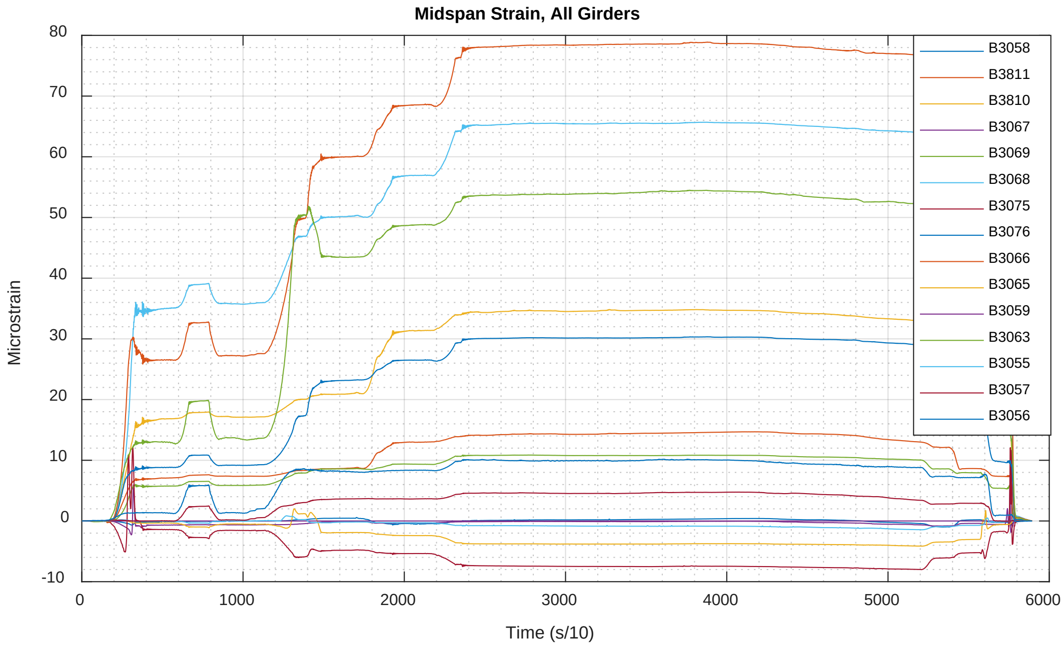

Figure 47: Bridge 5109 ALT_S_2_1 - Midspan

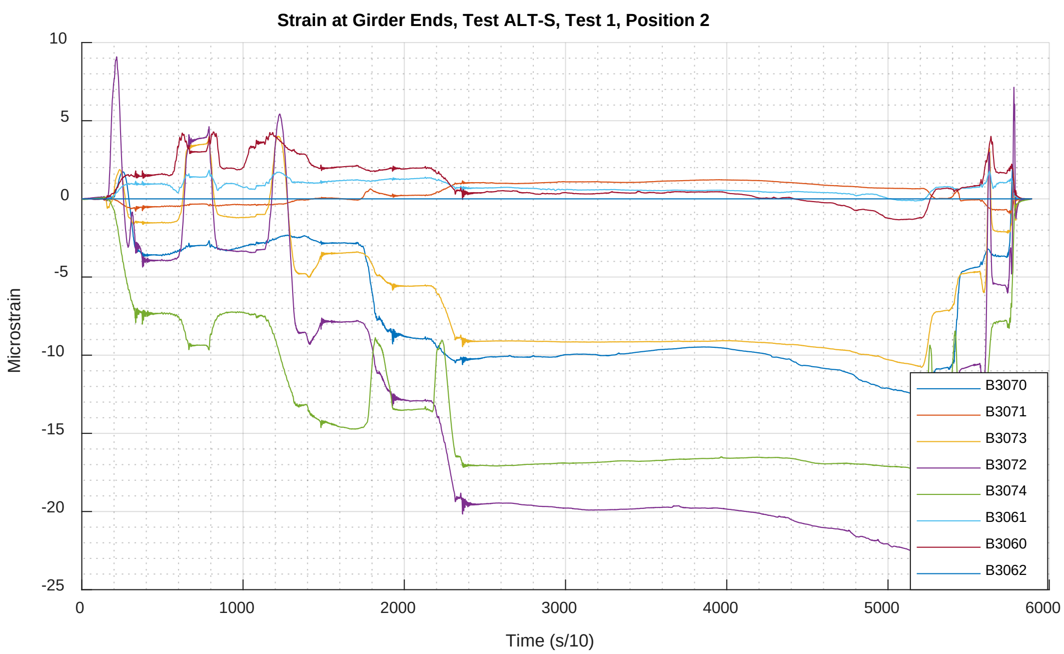

Figure 48: Bridge 5109 ALT_S_2_1 - Ends

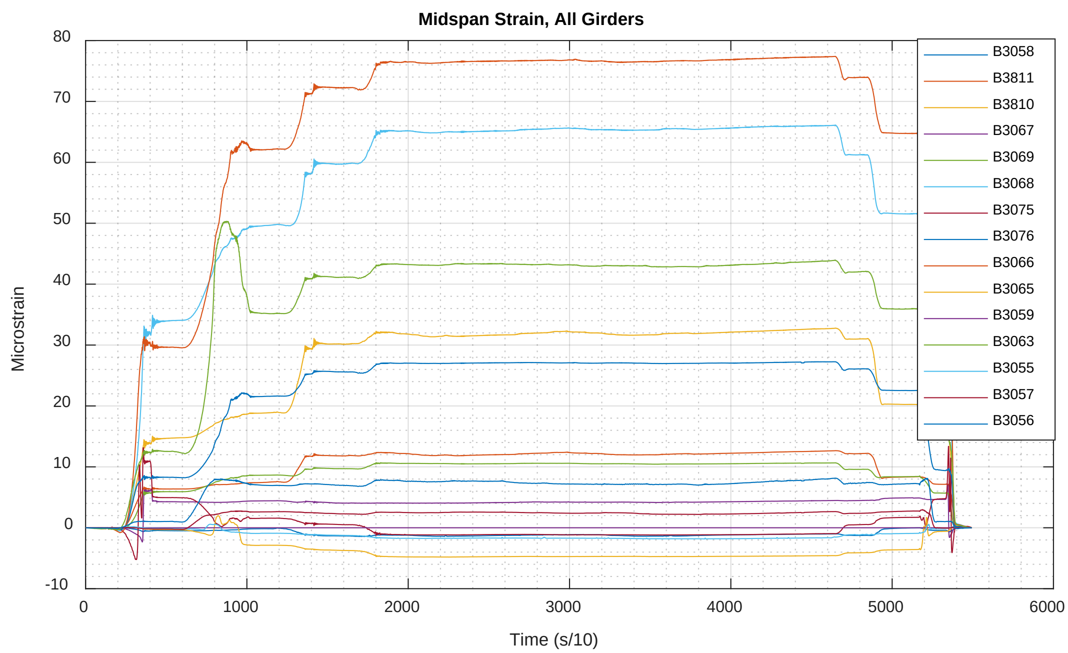

Figure 49: Bridge 5109 ALT_U_2_1 - Midspan

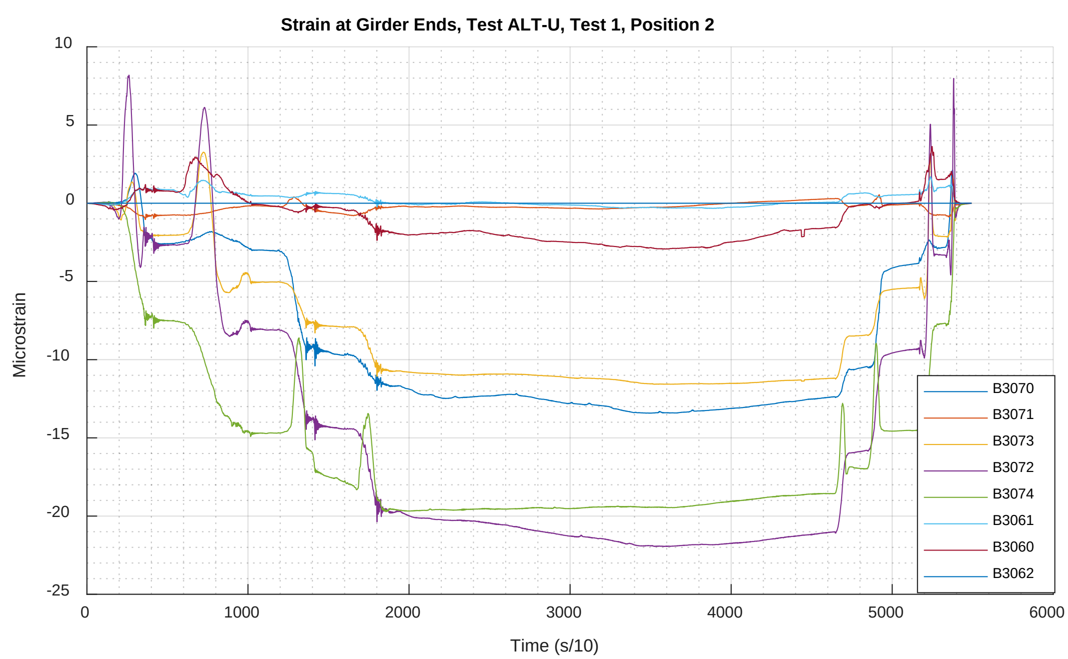

Figure 50: Bridge 5109 ALT_U_2_1 – Ends

A.3.5 Rating Factor Calculations

AASHTO Rating Calculations:

Bridge 5109 - Hampden, Maine

Material Parameters:

Concrete Compressive Strength	$f'_c := 2.5\ ksi$
Reinforcement Yield Strength	$F_y := 33\ ksi$
Unit Weight: Reinforced Concrete	$\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$
Unit Weight: Wearing Surface	$\gamma_{ws} := 0.150\ \frac{kip}{ft^3}$

Geometric Properties:

Span Length	$L := 47\ ft$
Girder Spacing - Interior	$S := 85.813\ in$
Girder Spacing - Exterior	$S_x := 57.31\ in$
Number of Girders	$NG := 5$
Skew Angle	$skew := 35\ °$
Lane Width	$lanewidth := 13\ ft$
Number of Lanes	$Nlane := 2$
Wearing Surface Thickness	$ws := 2.25\ in$
Thickness of Pavement Overlay	$ws_2 := 0\ in$
Girder Height - Interior	$h := 39.75\ in$
Girder Height - Exterior	$h_x := 39.75\ in$
Deck Thickness	$d_s := 6.25\ in$
Web Width - Interior	$b_w := 22.75\ in$
Web Width - Exterior	$b_{wx} := 22.75\ in$
Curb Depth	$h_{curb} := 11.25\ in$
Curb Width	$b_{curb} := 18\ in$
Height to Centroid of Reinforcement - Interior	$y_{bar} := \begin{bmatrix} 4.844 \\ 5.375 \\ 6.44 \\ 5.375 \\ 4.844 \end{bmatrix} in$
Height to Centroid of Reinforcement - Exterior	$y_{barx} := \begin{bmatrix} 4.844 \\ 5.375 \\ 6.44 \\ 5.375 \\ 4.844 \end{bmatrix} in$
Area of Reinforcement - Interior	$A_s := \begin{bmatrix} 10.125 \\ 12.656 \\ 15.188 \\ 12.656 \\ 10.125 \end{bmatrix} in^2$

Area of Reinforcement - Exterior $A_{sx} := \begin{bmatrix} 10.125 \\ 12.656 \\ 15.188 \\ 12.656 \\ 10.125 \end{bmatrix} in^2$

Distance from Centerline of Girder to Edge of Curb $d_e := -3.625\ in$

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

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder $P_{dint} := 2.07\ kip$

Location of Intermediate Diaphragm (Half, Third, Quarter) $loc_d := \text{“Half”}$

Distributed Load Due to Rail $w_{rail} := 0.287\ \frac{kip}{ft}$

Structural Dead Load Factor $\gamma_{DC} := 1.25$

Wearing Surface Dead Load Factor $\gamma_{DW} := 1.25$

Live Load Factor $\gamma_{LL} := 1.35$

Live Load Impact Factor $IM := 0.33$

Flexural Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 1.0$

Initial Calculations

Web Height - Interior $d_g := h - d_s$

Web Height - Exterior $d_{gx} := h_x - d_s$

Include Wearing Surface in Section Height $h := h + \text{if}\left(\gamma_{ws} = 0.15\ \frac{kip}{ft^3}, ws, 0\right) = 42\ in$

Depth to Centroid of Reinforcement - Interior $d := h - y_{bar}$

Depth to Centroid of Reinforcement - Exterior $d_x := h_x - y_{barx} + h_{curb}$

Moment Applied to Interor Girders from Diaphragm

$$M_d := \begin{array}{l} \text{if } loc_d = \text{“Half”} \\ \quad \| \ P_{dint} \cdot \frac{L}{4} \\ \text{else if } loc_d = \text{“Third”} \\ \quad \| \ P_{dint} \cdot \frac{L}{3} \\ \text{else if } loc_d = \text{“Quarter”} \\ \quad \| \ P_{dint} \cdot \frac{L}{4} + P_{dint} \cdot \frac{L}{4} \end{array} = 24.323\ ft \cdot kip$$

Moment Applied to Exterior Girders from Diaphragm $M_{dx} := \frac{M_d}{2} = 12.161\ ft \cdot kip$

Distribution Factors

Dictance Between Centroids of Deck and Web	$e_g := \frac{d_g + d_s}{2} = 19.875\ \mathbf{in}$
Area of Web	$A := d_g \cdot b_w = 762.125\ \mathbf{in}^2$
Moment of Inertia of Web	$I := \frac{b_w \cdot d_g^{\,3}}{12} = \left(7.127 \cdot 10^4\right)\ \mathbf{in}^4$
Modular Ratio - Deck and Web	$n := 1$
Longitudinal Stiffness Parameter	$K_g := n \cdot \left(I + A \cdot e_g^{\,2}\right) = \left(3.723 \cdot 10^5\right)\ \mathbf{in}^4$
Interior Moment Distribution Factor - 1 Lane	$g_{m1} := 0.06 + \left(\frac{S}{14\ \mathbf{ft}}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\,3}}\right)^{0.1} = 0.54$
Interior Moment Distribution Factor - 2 Lane	$g_{m2} := 0.075 + \left(\frac{S}{9.5\ \mathbf{ft}}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\,3}}\right)^{0.1} = 0.714$
Controling Interior Moment Distribution Factor	$g_m := \max\left(g_{m1}, g_{m2}\right)$
Roadway Width	$W_r := lanewidth \cdot Nlane$
Eccentricity of Design Lane From C.G. of Girders	$e_1 := \frac{W_r}{2} - 5\ \mathbf{ft} + exc = 8\ \mathbf{ft}$
Eccentricity of Exterior Girder From C.G. of Girders	$X_{ext} := (NG - 1) \cdot \frac{S}{2} = 14.302\ \mathbf{ft}$
Eccentricity of Each Girder	$x_1 := X_{ext}$ $x_2 := X_{ext} - S$ $x_3 := X_{ext} - 2 \cdot S$ $x_4 := \mathbf{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ \mathbf{ft}\right)$ $x_5 := \mathbf{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ \mathbf{ft}\right)$
Lever Rule Distribution Factor - One Lane	$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\,2} + x_2^{\,2} + x_3^{\,2} + x_4^{\,2} + x_5^{\,2}} = 0.424$ $g_{mR1} := \mathbf{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0.508$
Lever Rule Distribution Factor - Two Lanes	$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot \left(e_1 - 5\ \mathbf{ft}\right)}{x_1^{\,2} + x_2^{\,2} + x_3^{\,2} + x_4^{\,2} + x_5^{\,2}} = 0.484$ $g_{mR2} := \mathbf{if}\left(P_{dint} > 0, R_2, 0\right) = 0.484$
Exterior Moment Distribution Factor	$g_{mx1} := \frac{1.2\ \left(S + d_e - 2\ \mathbf{ft}\right)}{2 \cdot S} = 0.407$ $ee := 0.77 + \frac{d_e}{9.1\ \mathbf{ft}} = 0.737$ $g_{mx2} := g_{m2} \cdot ee = 0.526$

$g_{mx} := \max(g_{mx1}, g_{mx2}) = 0.526$

Skew Correction Factor

$$c_1 := 0.25 \cdot \left(\frac{K_g}{12 \cdot L \cdot d_s^{\,3}}\right)^{0.25} \cdot \left(\frac{S}{L}\right)^{.5} = 0.067$$

$\theta := \mathbf{if}(skew \geq 30°, skew, 0°)$

$C_\theta := 1 - c_1 \cdot (\tan(\theta))^{1.5} = 0.961$

$g_m := g_m \cdot C_\theta = 0.686$

$g_{mx} := g_{mx} \cdot C_\theta = 0.506$

Interior DF	Exterior DF
$g_m = 0.686$	$g_{mx} = 0.506$

Loading

Interior Girder Dead Load $\quad w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.794 \dfrac{kip}{ft}$

Deck Dead Load $\quad w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.559 \dfrac{kip}{ft}$

Curb Dead Load $\quad w_{curb} := 2 \cdot \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.422 \dfrac{kip}{ft}$

Dead Load from Nonstructural Components $\quad w_{ns} := \dfrac{w_{curb}}{NG} + w_{rail} = 0.371 \dfrac{kip}{ft}$

Total Structural Dead Load on Interior Girders $\quad DC := w_{girder} + w_{deck} + w_{ns} = 1.724 \dfrac{kip}{ft}$

Exterior Girder Dead Load $\quad w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.794 \dfrac{kip}{ft}$

Exterior Deck Dead Load $\quad w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.373 \dfrac{kip}{ft}$

Total Structural Dead Load on Exterior Girders $\quad DC_x := w_{girderx} + w_{deckx} + w_{ns} = 1.538 \dfrac{kip}{ft}$

Wearing Surface Dead Load on Interior Girders $\quad DW := \gamma_{ws} \cdot (ws + ws_2) \cdot S = 0.201 \dfrac{kip}{ft}$

Wearing Surface Dead Load on Exterior Girders $\quad DW_x := \gamma_{ws} \cdot (ws + ws_2) \cdot S_x = 0.134 \dfrac{kip}{ft}$

Dead Load Moments

$$M_{DC} := \frac{DC \cdot L^2}{8} + M_d \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} + M_{dx}$$

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

$M_{DC}=500.344\ ft \cdot kip$ $M_{DCx}=436.943\ ft \cdot kip$

$M_{DW}=55.535\ ft \cdot kip$ $M_{DWx}=37.089\ ft \cdot kip$

Live Load Moment - Truck Load $M_{Truck}:=32\ kip \cdot \left(\frac{L}{4}\right)+\frac{40\ kip}{2} \cdot \left(\frac{L}{2}-14\ ft\right)=566\ ft \cdot kip$

Live Load Moment - Tandem $M_{Tandem}:=25\ kip \cdot \frac{L}{4}+\frac{25\ kip}{2} \cdot \left(\frac{L}{2}-4\ ft\right)=537.5\ ft \cdot kip$

Live Load Moment - Lane $M_{Lane}:=0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8}=176.72\ ft \cdot kip$

Total HL-93 Live Load $M_{LL}:=M_{Lane}+(1+IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right)$

$M_{LL}=929.5\ ft \cdot kip$

Nominal Resistance

Depth Whitney Stress Block - Interior $a:=A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S}=\begin{bmatrix}1.832\\2.29\\2.749\\2.29\\1.832\end{bmatrix} in$

Nominal Moment Resistance - Interior $M_n:=\overrightarrow{F_y \cdot A_s \cdot \left(d-\frac{a}{2}\right)}=\begin{bmatrix}1.009 \cdot 10^3\\1.235 \cdot 10^3\\1.428 \cdot 10^3\\1.235 \cdot 10^3\\1.009 \cdot 10^3\end{bmatrix} ft \cdot kip$

Interior Nominal Moment Capacity

$M_{capacity}:=\max\left(M_n\right)=\left(1.428 \cdot 10^3\right) ft \cdot kip$

Depth Whitney Stress Block - Exterior $a_x:=A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x}=\begin{bmatrix}2.744\\3.429\\4.116\\3.429\\2.744\end{bmatrix} in$

Nominal Moment Resistance - Exterior $M_{nx}:=\overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x-\frac{a_x}{2}\right)}=\begin{bmatrix}1.247 \cdot 10^3\\1.528 \cdot 10^3\\1.775 \cdot 10^3\\1.528 \cdot 10^3\\1.247 \cdot 10^3\end{bmatrix} (ft \cdot kip)$

Exterior Nominal Moment Capacity

$M_{capacityx}:=\max\left(M_{nx}\right)=\left(1.775 \cdot 10^3\right) ft \cdot kip$

Rating Factors

Interior Moment Rating Factor

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

Exterior Moment Rating Factor

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

Interior

$RF_{Interior} = 0.686$

Exterior

$RF_{Exterior} = 1.585$

Rating Factor Improvements

Concrete Compressive Strength - Larger is More Conservative	$f'_c := 5\ ksi$
Concrte Elastic Modulus	$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = (4.07 \cdot 10^3)\ ksi$

Interior Girders

Maximum Recorded Strain	$\varepsilon_T := 90.50 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 830.05\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 10619\ in^3$
Cracked Section Modulus	$S_{cr} := 3672\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \mathbf{if}\ (Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.581 \cdot 10^{-4}$
Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.747$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.893$
Test Understanding Factor	$k_b := \mathbf{if}\ (r_M > 0.7, 0.5, 0) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.374$
Improved Rating Factor	$RF_{Improved} := RF_{Interior} \cdot k = 0.942$

Exterior Girders

Maximum Recorded Strain	$\varepsilon_T := 34.5 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 853.3\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 9767\ in^3$
Cracked Section Modulus	$S_{cr} := 3552\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \mathbf{if}\ (Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 1.302 \cdot 10^{-4}$

Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 2.775$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.918$
Test Understanding Factor	$k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 2.387$
Improved Rating Factor	$RF_{ImprovedExt} := RF_{Exterior} \cdot k = 3.783$

Improved Rating Factors

Interior	Exterior
$RF_{Improved} = 0.942$	$RF_{ImprovedExt} = 3.783$

Figure 51: Bridge 5109 calculations

A.4 Unity No. 2390

A.4.1 Experimental Configuration and Experimental Data Collected

Table 19: Bridge 2390 experimental configuration and experimental data collected

File Contents	File Name	File Type
Sensors	Br2390 _Sensors.csv	CSV Format
Sensor Layout	Br2390 _SensorLayout.mat	MATLAB Data File
Sensor Data	Br2390_ALT_S_2_1_Strain.mat	MATLAB Data File
	Br2390_ALT_U_2_1_Strain.mat	MATLAB Data File
	Br2390_MAX_S_1_2_Strain.mat	MATLAB Data File
	Br2390_MAX_S_2_1_Strain.mat	MATLAB Data File
	Br2390_MAX_S_3_1_Strain.mat	MATLAB Data File
	Br2390_MAX_U_2_1_Strain.mat	MATLAB Data File
	Br2390_SBS_S_2_1_Strain.mat	MATLAB Data File
	Br2390_SBS_U_2_1_Strain.mat	MATLAB Data File

A.4.2 Instrumentation

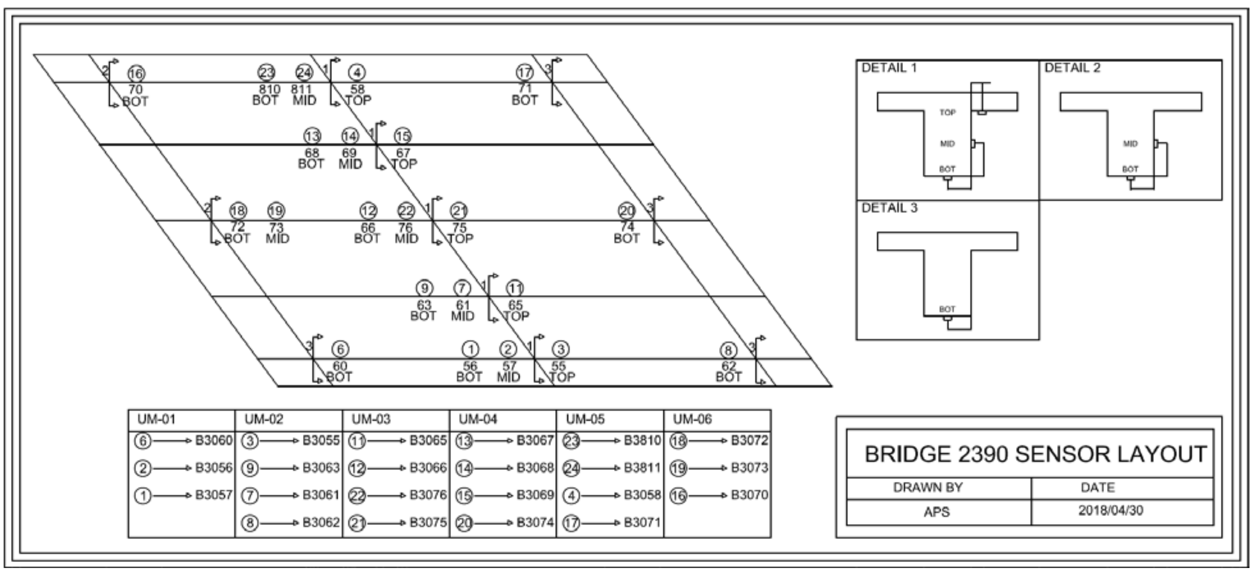

Figure 52: Bridge 2390 sensor layout

A.4.3 Loading

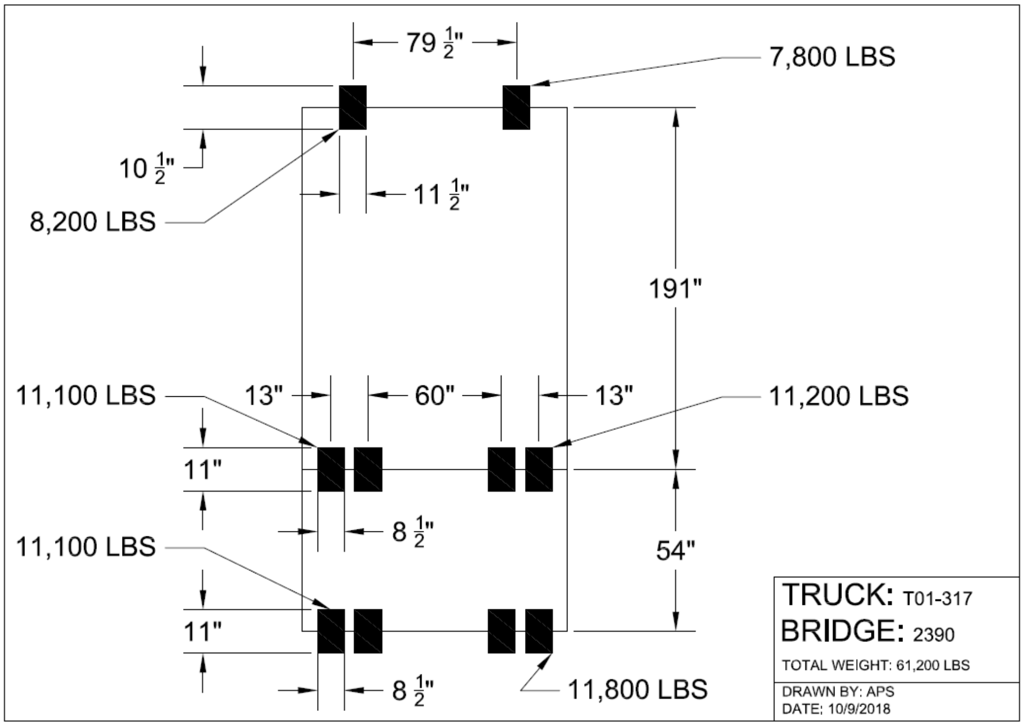

Figure 53: Bridge 2390 Truck T01-317 loading

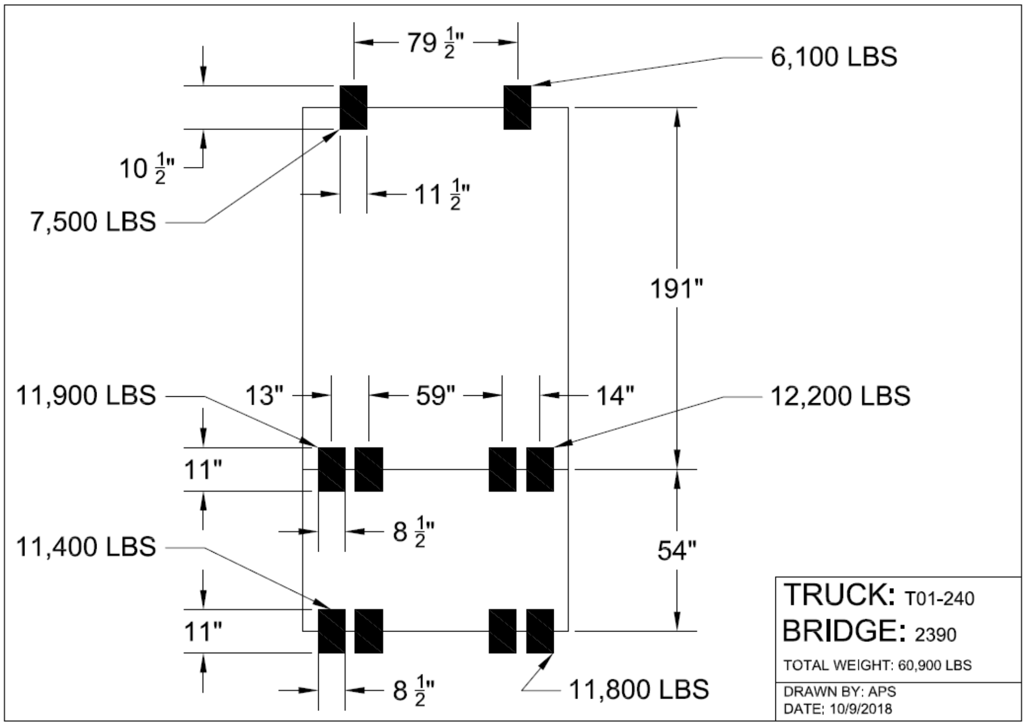

Figure 54: Bridge 2390 Truck T01-240 loading

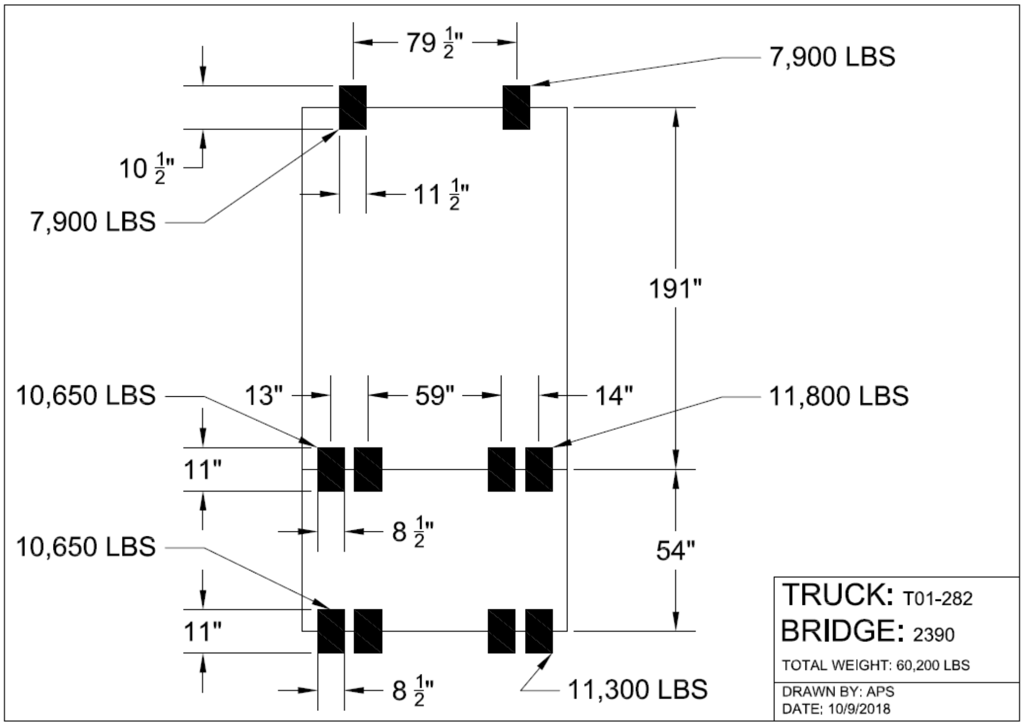

Figure 55: Bridge 2390 Truck T01-282 loading

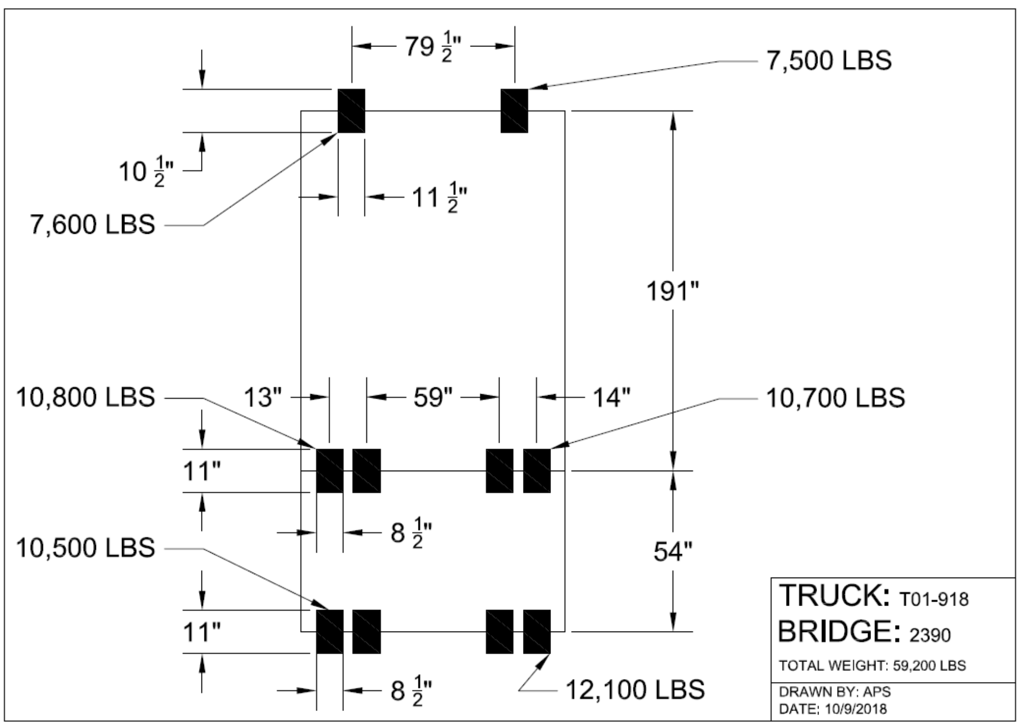

Figure 56: Bridge 2390 Truck T01-918 loading

A.4.4 Representative Data Plots

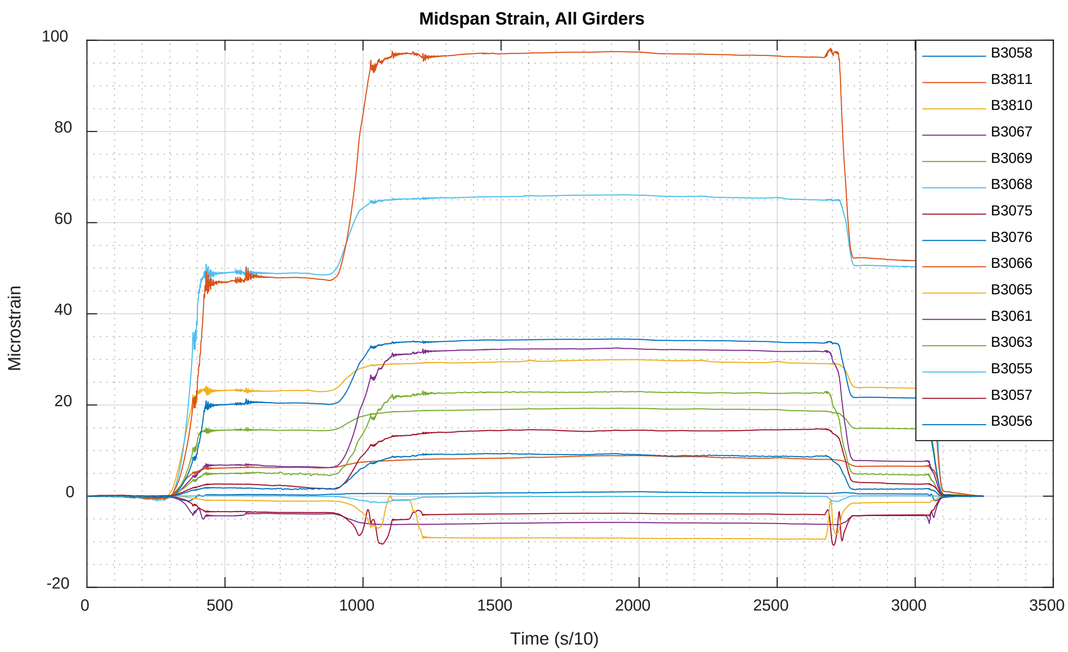

Figure 57: Bridge 2390 SBS_S_2_2 strains - Midspan

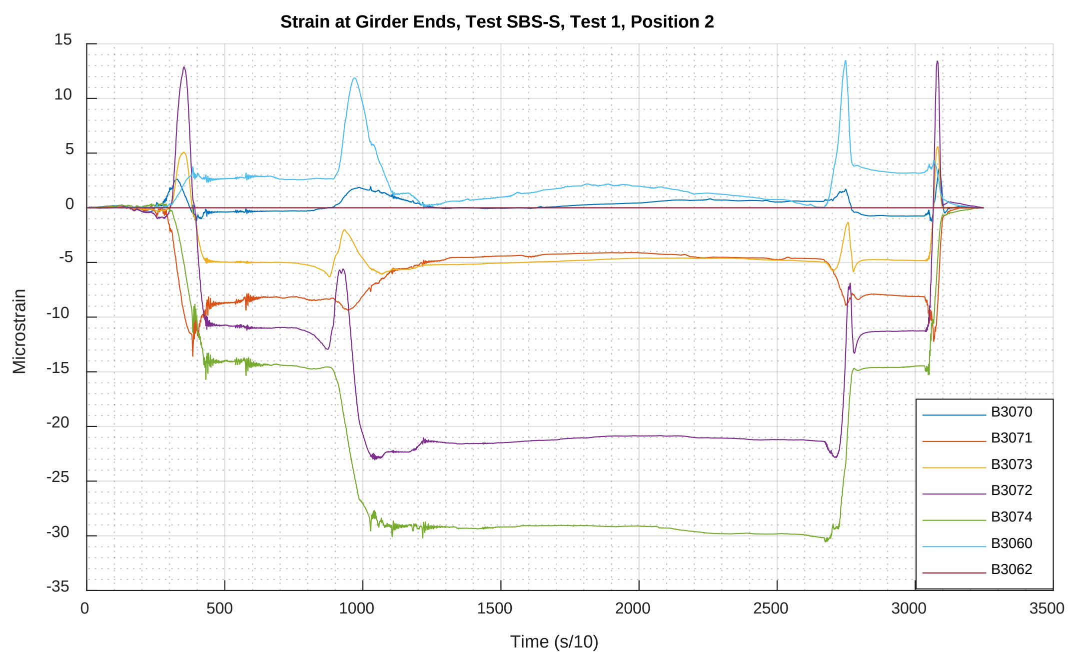

Figure 58: Bridge 2390 SBS_S_2_1 strains - Ends

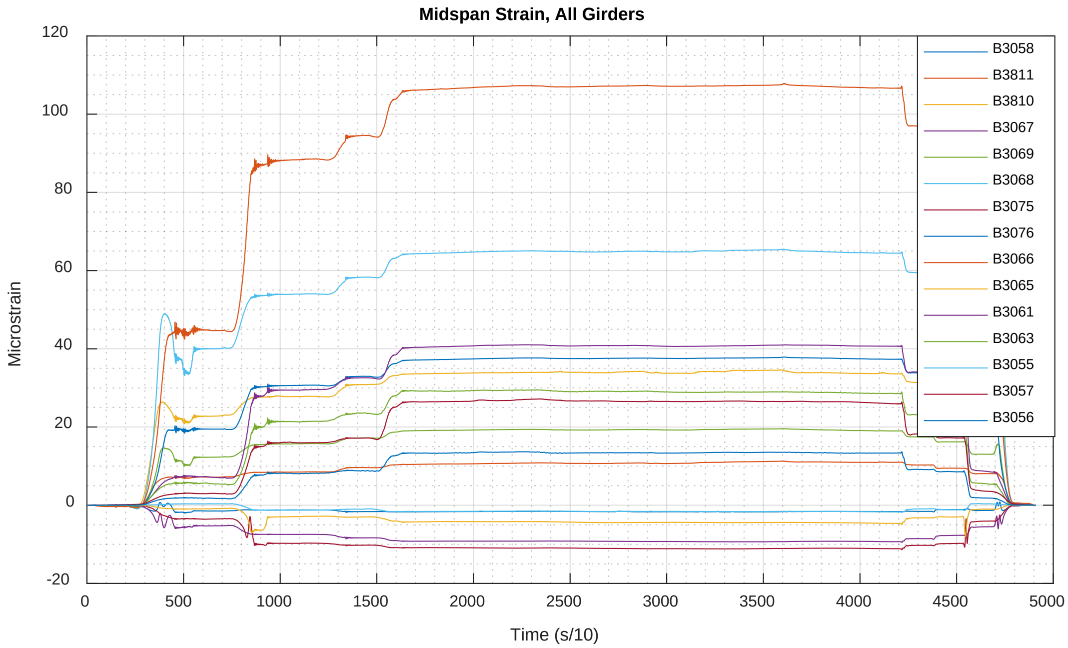

Figure 59: Bridge 2390 SBS_U_2_1 strains - Midspan

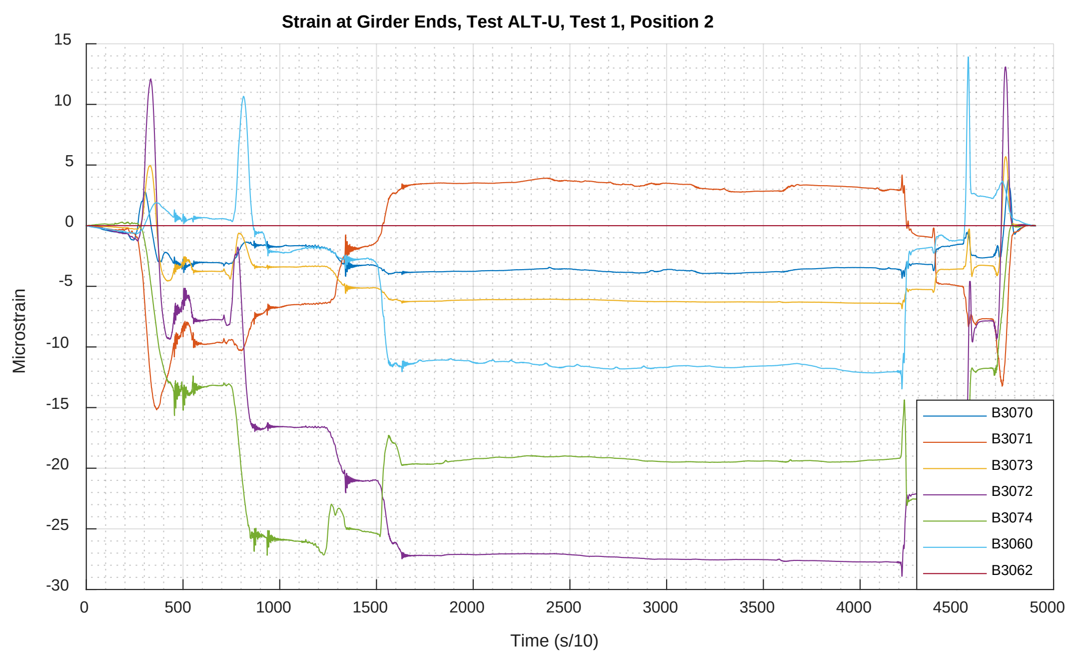

Figure 60: Bridge 2390 SBS_U_2_1 strains – Ends

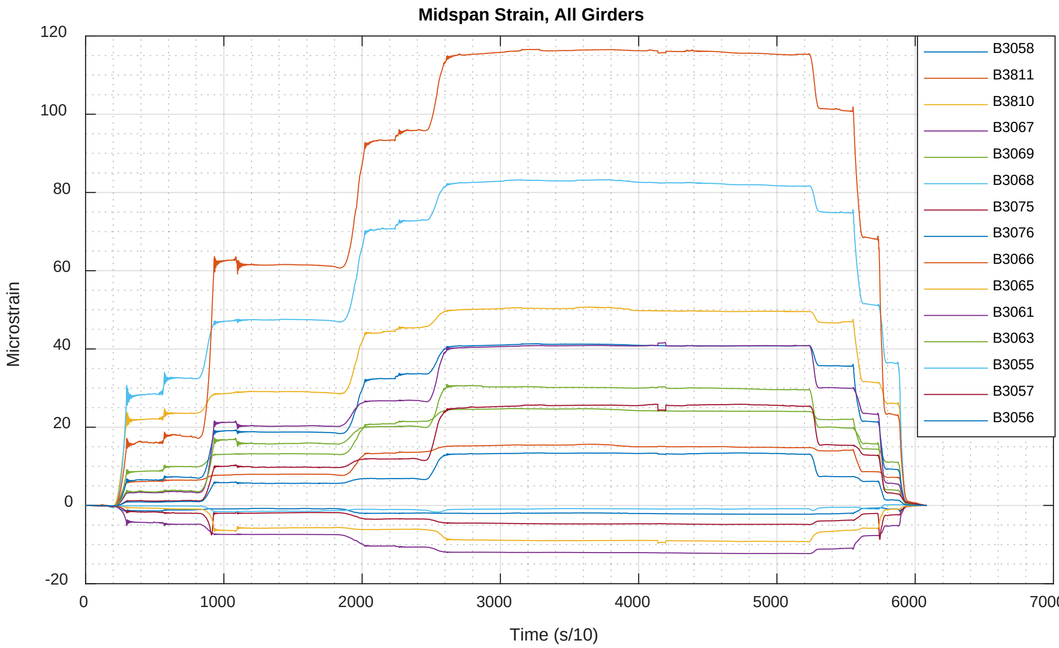

Figure 61: Bridge 2390 MAX_S_2_1 strains - Midspan

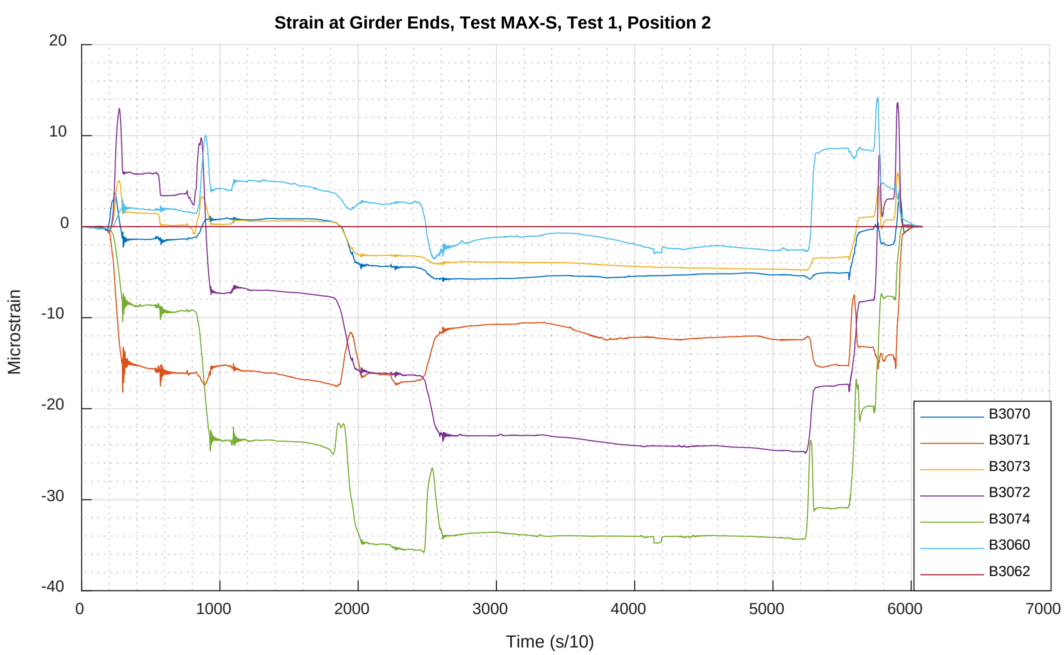

Figure 62: Bridge 2390 MAX_S_2_1 strains - Ends

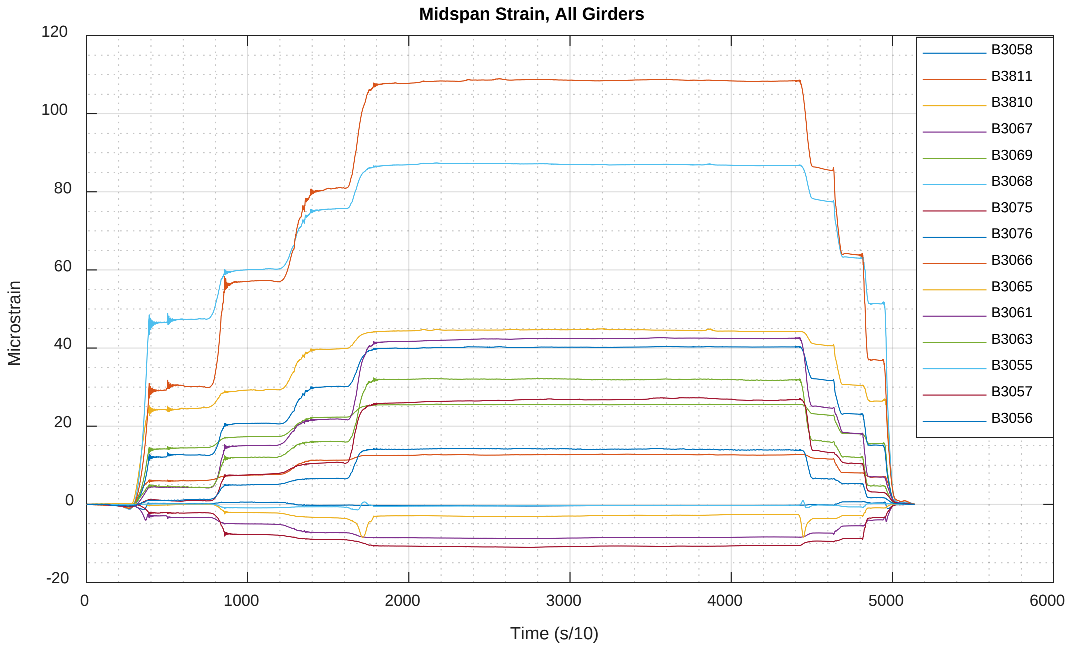

Figure 63: Bridge 2390 MAX_U_2_1 strains - Midspan

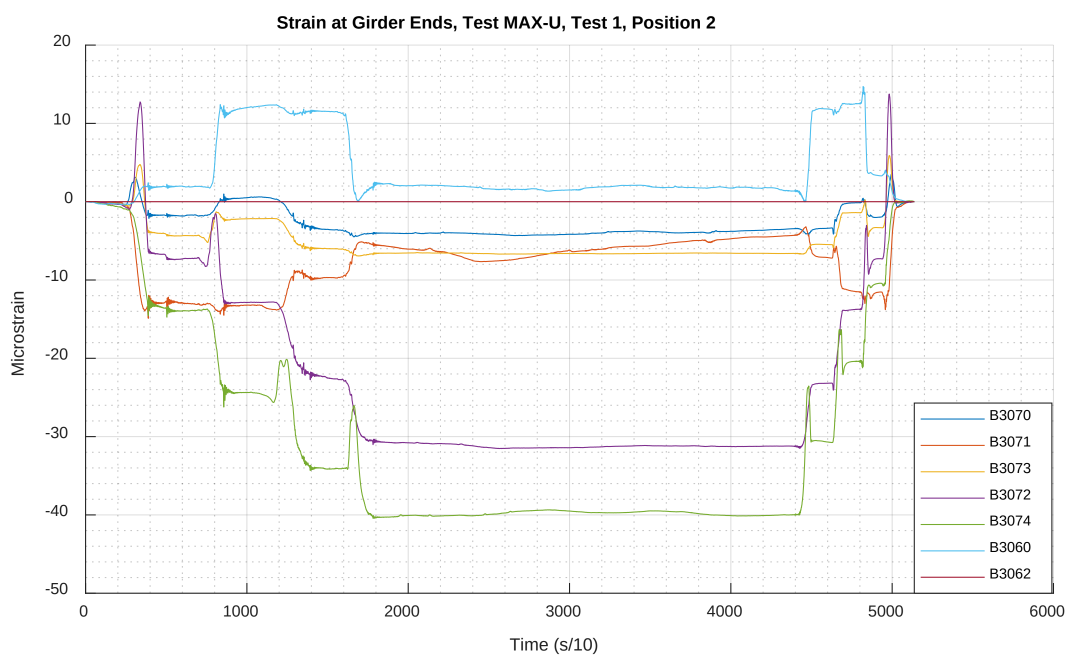

Figure 64 Bridge 2390 MAX_U_2_1 strains – Ends

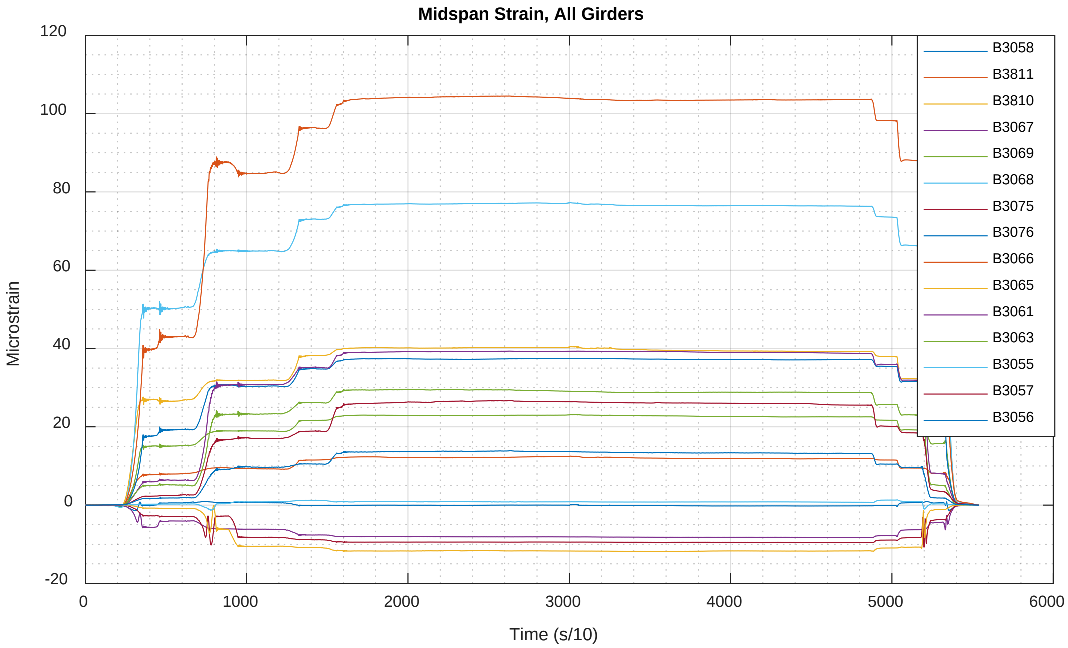

Figure 65: Bridge 2390 ALT_S_2_1 strains - Midspan

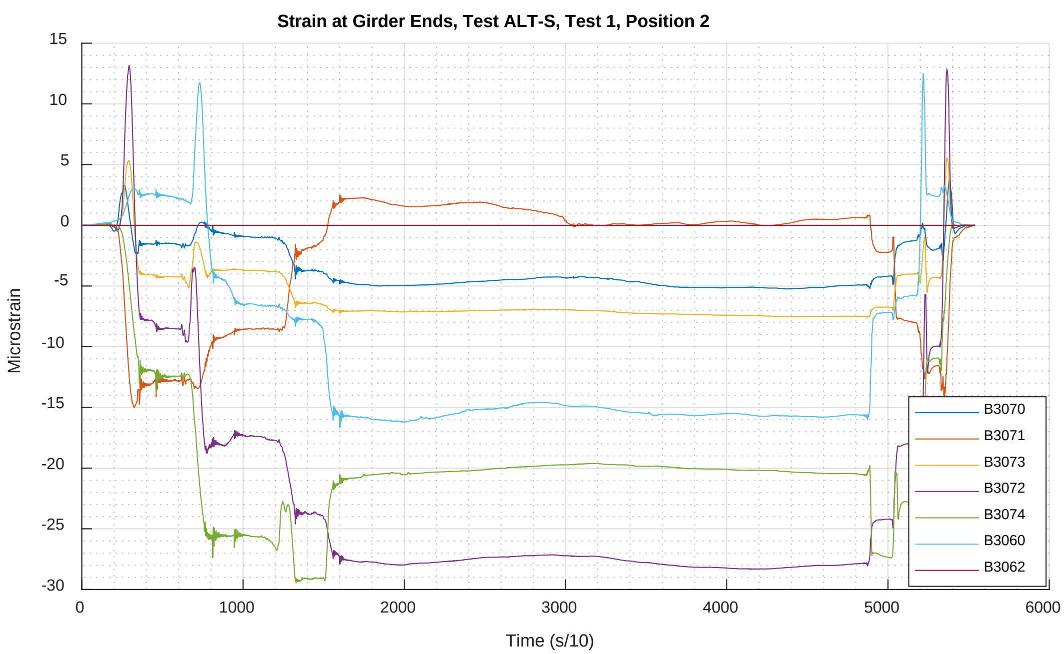

Figure 66 Bridge 2390 ALT_S_2_1 strains – Ends

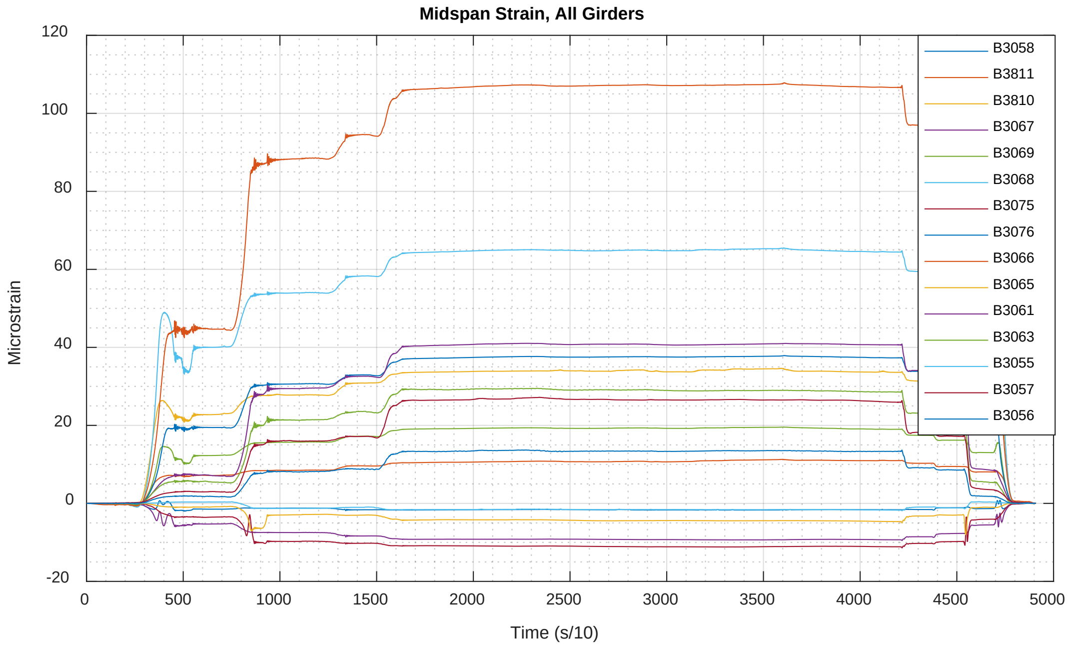

Figure 67: Bridge 2390 ALT_U_2_1 strains - Midspan

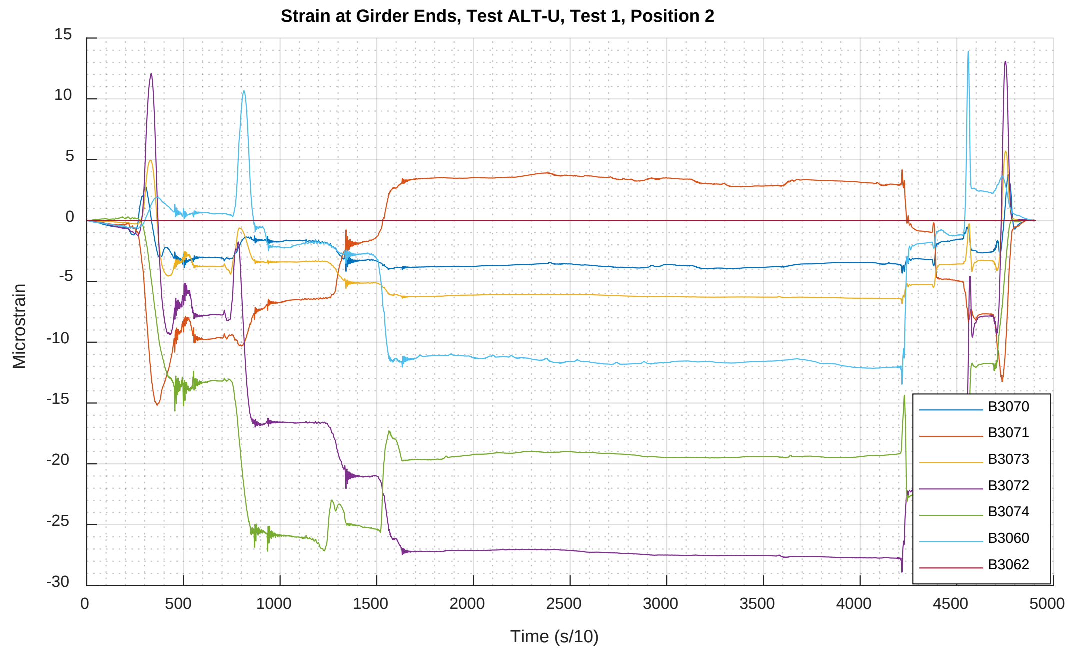

Figure 68 Bridge 2390 ALT_U_2_1 strains - Ends

A.4.5 Rating Factor Calculations

AASHTO Rating Calculations:

Bridge 2390 - Unity, Maine

Material Parameters:

Concrete Compressive Strength	$f'_c := 2.5\ ksi$
Reinforcement Yield Strength	$F_y := 33\ ksi$
Unit Weight: Reinforced Concrete	$\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$
Unit Weight: Wearing Surface	$\gamma_{ws} := 0.150\ \frac{kip}{ft^3}$

Geometric Properties:

Span Length	$L := 37\ ft$
Girder Spacing - Interior	$S := 73.5\ in$
Girder Spacing - Exterior	$S_x := 42.75\ in$
Number of Girders	$NG := 5$
Skew Angle	$skew := 30\ °$
Lane Width	$lanewidth := 11\ ft$
Number of Lanes	$Nlane := 2$
Wearing Surface Thickness	$ws := 4\ in$
Thickness of Pavement Overlay	$ws_2 := 5\ in$
Girder Height - Interior	$h := 31.25\ in$
Girder Height - Exterior	$h_x := 31.25\ in$
Deck Thickness	$d_s := 5.75\ in$
Web Width - Interior	$b_w := 24\ in$
Web Width - Exterior	$b_{wx} := 15\ in$
Curb Depth	$h_{curb} := 12\ in$
Curb Width	$b_{curb} := 21\ in$
Height to Centroid of Reinforcement - Interior	$y_{bar} := \begin{bmatrix} 4.375 \\ 5.069 \\ 5.3125 \\ 5.069 \\ 4.375 \end{bmatrix} in$
Height to Centroid of Reinforcement - Exterior	$y_{barx} := \begin{bmatrix} 4.875 \\ 4.875 \\ 4.875 \\ 4.875 \\ 4.875 \end{bmatrix} in$
Area of Reinforcement - Interior	$A_s := \begin{bmatrix} 8.859 \\ 11.391 \\ 12.656 \\ 11.391 \\ 8.859 \end{bmatrix} in^2$

Area of Reinforcement - Exterior $A_{sx} := \begin{bmatrix} 7.8125 \\ 7.8125 \\ 7.8125 \\ 7.8125 \\ 7.8125 \end{bmatrix} in^2$

Distance from Centerline of Girder to Edge of Curb $d_e := -10.5\ in$

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

Load and Analysis Parameters

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

Location of Intermediate Diaphragm (Half, Third, Quarter) $loc_d :=$ "Half"

Distributed Load Due to Rail $w_{rail} := 0.319\ \frac{kip}{ft}$

Structural Dead Load Factor $\gamma_{DC} := 1.25$

Wearing Surface Dead Load Factor $\gamma_{DW} := 1.25$

Live Load Factor $\gamma_{LL} := 1.35$

Live Load Impact Factor $IM := 0.33$

Flexural Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 1.0$

Initial Calculations

Web Height - Interior $d_g := h - d_s$

Web Height - Exterior $d_{gx} := h_x - d_s$

Include Wearing Surface in Section Height $h := h + \textbf{if}\left(\gamma_{ws} = 0.15\ \frac{kip}{ft^3}, ws, 0\right) = 35.25\ in$

Depth to Centroid of Reinforcement - Interior $d := h - y_{bar}$

Depth to Centroid of Reinforcement - Exterior $d_x := h_x - y_{barx} + h_{curb}$

Moment Applied to Interor Girders from Diaphragm

$$M_d := \begin{array}{l} \text{if } loc_d = \text{“Half”} \\ \quad P_{dint} \cdot \frac{L}{4} \\ \text{else if } loc_d = \text{“Third”} \\ \quad P_{dint} \cdot \frac{L}{3} \\ \text{else if } loc_d = \text{“Quarter”} \\ \quad P_{dint} \cdot \frac{L}{4} + P_{dint} \cdot \frac{L}{4} \end{array} = 0\ ft \cdot kip$$

Moment Applied to Exterior Girders from Diaphragm $M_{dx} := \frac{M_d}{2} = 0\ ft \cdot kip$

Distribution Factors

Dictance Between Centroids of Deck and Web	$e_g := \frac{d_g + d_s}{2} = 15.625\ in$
Area of Web	$A := d_g \cdot b_w = 612\ in^2$
Moment of Inertia of Web	$I := \frac{b_w \cdot d_g^{3}}{12} = \left(3.316 \cdot 10^4\right)\ in^4$
Modular Ratio - Deck and Web	$n := 1$
Longitudinal Stiffness Parameter	$K_g := n \cdot \left(I + A \cdot e_g^{2}\right) = \left(1.826 \cdot 10^5\right)\ in^4$
Interior Moment Distribution Factor - 1 Lane	$g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{3}}\right)^{0.1} = 0.512$
Interior Moment Distribution Factor - 2 Lane	$g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{3}}\right)^{0.1} = 0.654$
Controling Interior Moment Distribution Factor	$g_m := \max\left(g_{m1}, g_{m2}\right)$
Roadway Width	$W_r := lanewidth \cdot Nlane$
Eccentricity of Design Lane From C.G. of Girders	$e_1 := \frac{W_r}{2} - 5\ ft + exc = 6\ ft$
Eccentricity of Exterior Girder From C.G. of Girders	$X_{ext} := (NG - 1) \cdot \frac{S}{2} = 12.25\ ft$
Eccentricity of Each Girder	$x_1 := X_{ext}$
	$x_2 := X_{ext} - S$
	$x_3 := X_{ext} - 2 \cdot S$
	$x_4 := \mathbf{if}\left(NG > 3, X_{ext} - 3 \cdot S, 0\ ft\right)$
	$x_5 := \mathbf{if}\left(NG > 4, X_{ext} - 4 \cdot S, 0\ ft\right)$
Lever Rule Distribution Factor - One Lane	$R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{2} + x_2^{2} + x_3^{2} + x_4^{2} + x_5^{2}} = 0.396$
	$g_{mR1} := \mathbf{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0$
Lever Rule Distribution Factor - Two Lanes	$R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot \left(e_1 - 5\ ft\right)}{x_1^{2} + x_2^{2} + x_3^{2} + x_4^{2} + x_5^{2}} = 0.433$
	$g_{mR2} := \mathbf{if}\left(P_{dint} > 0, R_2, 0\right) = 0$
Exterior Moment Distribution Factor	$g_{mx1} := \frac{1.2\ \left(S + d_e - 2\ ft\right)}{2 \cdot S} = 0.318$
	$ee := 0.77 + \frac{d_e}{9.1\ ft} = 0.674$
	$g_{mx2} := g_{m2} \cdot ee = 0.441$

$g_{mx} := \max\left(g_{mx1}, g_{mx2}\right) = 0.441$

Skew Correction Factor

$c_1 := 0.25 \cdot \left(\dfrac{K_g}{12 \cdot L \cdot d_s^{\,3}}\right)^{0.25} \cdot \left(\dfrac{S}{L}\right)^{.5} = 0.066$

$\theta := \mathbf{if}\left(skew \ge 30\,°, skew, 0\,°\right)$

$C_\theta := 1 - c_1 \cdot \left(\tan(\theta)\right)^{1.5} = 0.971$

$g_m := g_m \cdot C_\theta = 0.635$

$g_{mx} := g_{mx} \cdot C_\theta = 0.428$

Interior DF — Exterior DF

$g_m = 0.635$ — $g_{mx} = 0.428$

Loading

Interior Girder Dead Load: $w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.638\ \dfrac{kip}{ft}$

Deck Dead Load: $w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.44\ \dfrac{kip}{ft}$

Curb Dead Load: $w_{curb} := 2 \cdot \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.525\ \dfrac{kip}{ft}$

Dead Load from Nonstructural Components: $w_{ns} := \dfrac{w_{curb}}{NG} + w_{rail} = 0.424\ \dfrac{kip}{ft}$

Total Structural Dead Load on Interior Girders: $DC := w_{girder} + w_{deck} + w_{ns} = 1.502\ \dfrac{kip}{ft}$

Exterior Girder Dead Load: $w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.398\ \dfrac{kip}{ft}$

Exterior Deck Dead Load: $w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.256\ \dfrac{kip}{ft}$

Total Structural Dead Load on Exterior Girders: $DC_x := w_{girderx} + w_{deckx} + w_{ns} = 1.078\ \dfrac{kip}{ft}$

Wearing Surface Dead Load on Interior Girders: $DW := \gamma_{ws} \cdot \left(ws + ws_2\right) \cdot S = 0.689\ \dfrac{kip}{ft}$

Wearing Surface Dead Load on Exterior Girders: $DW_x := \gamma_{ws} \cdot \left(ws + ws_2\right) \cdot S_x = 0.401\ \dfrac{kip}{ft}$

Dead Load Moments: $M_{DC} := \dfrac{DC \cdot L^2}{8} + M_d \qquad M_{DCx} := \dfrac{DC_x \cdot L^2}{8} + M_{dx}$

$M_{DW} := \dfrac{DW \cdot L^2}{8} \qquad M_{DWx} := \dfrac{DW_x \cdot L^2}{8}$

$M_{DC} = 256.984\ ft \cdot kip$ $\quad M_{DCx} = 184.557\ ft \cdot kip$

$M_{DW} = 117.916\ ft \cdot kip$ $\quad M_{DWx} = 68.584\ ft \cdot kip$

Live Load Moment - Truck Load $\quad M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 386\ ft \cdot kip$

Live Load Moment - Tandem $\quad M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 412.5\ ft \cdot kip$

Live Load Moment - Lane $\quad M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 109.52\ ft \cdot kip$

Total HL-93 Live Load $\quad M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right)$

$M_{LL} = 658.145\ ft \cdot kip$

Nominal Resistance

Depth Whitney Stress Block - Interior

$$a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 1.872 \\ 2.407 \\ 2.674 \\ 2.407 \\ 1.872 \end{bmatrix} in$$

Nominal Moment Resistance - Interior

$$M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 729.384 \\ 907.731 \\ 995.412 \\ 907.731 \\ 729.384 \end{bmatrix} ft \cdot kip$$

Interior Nominal Moment Capacity

$M_{capacity} := \max\left(M_n\right) = 995.412\ ft \cdot kip$

Depth Whitney Stress Block - Exterior

$$a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 2.838 \\ 2.838 \\ 2.838 \\ 2.838 \\ 2.838 \end{bmatrix} in$$

Nominal Moment Resistance - Exterior

$$M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 793.977 \\ 793.977 \\ 793.977 \\ 793.977 \\ 793.977 \end{bmatrix} (ft \cdot kip)$$

Exterior Nominal Moment Capacity

$M_{capacityx} := \max\left(M_{nx}\right) = 793.977\ ft \cdot kip$

Rating Factors

Interior Moment Rating Factor

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

Exterior Moment Rating Factor

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

Interior	Exterior
$RF_{Interior} = 0.757$	$RF_{Exterior} = 1.047$

Rating Factor Improvements

Concrete Compressive Strength - Larger is More Conservative	$f'_c := 5\ ksi$
Concrte Elastic Modulus	$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = (4.07 \cdot 10^3)\ ksi$

Interior Girders

Maximum Recorded Strain	$\varepsilon_T := 112.56 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 613.45\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 8415\ in^3$
Cracked Section Modulus	$S_{cr} := 2764\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \text{if}(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.366 \cdot 10^{-4}$
Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.213$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.932$
Test Understanding Factor	$k_b := \text{if}(r_M > 0.7, 0.5, 0) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.107$
Improved Rating Factor	$RF_{Improved} := RF_{Interior} \cdot k = 0.838$

Exterior Girders

Maximum Recorded Strain	$\varepsilon_T := 79.28 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 613.45\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 8132\ in^3$
Cracked Section Modulus	$S_{cr} := 2071\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \text{if}(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 9.522 \cdot 10^{-5}$

Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.201$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.932$
Test Understanding Factor	$k_b := \mathbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.101$
Improved Rating Factor	$RF_{ImprovedExt} := RF_{Exterior} \cdot k = 1.152$

Improved Rating Factors

Interior	Exterior
$RF_{Improved} = 0.838$	$RF_{ImprovedExt} = 1.152$

Figure 69: Bridge 2390 calculations

A.5 Atkinson No. 2879

A.5.1 Experimental Configuration and Experimental Data Collected

Table 20: Bridge 2879 experimental configuration and experimental data collected

File Contents	***File Name***	***File Type***
Sensors	Br2130_Sensors.csv	CSV Format
Sensor Layout	Br2130_SensorLayout.csv	MATLAB Data File
Sensor Data	Br2879_ALT_S_2_1_Strain.mat	MATLAB Data File
	Br2879_ALT_U_2_1_Strain.mat	MATLAB Data File
	Br2879_MAX_S_1_1_Strain.mat	MATLAB Data File
	Br2879_MAX_S_2_1_Strain.mat	MATLAB Data File
	Br2879_MAX_S_3_1_Strain.mat	MATLAB Data File
	Br2879_MAX_U_2_1_Strain.mat	MATLAB Data File
	Br2879_SBS_S_2_1_Strain.mat	MATLAB Data File
	Br2879_SBS_U_2_1_Strain.mat	MATLAB Data File

A.5.2 Instrumentation

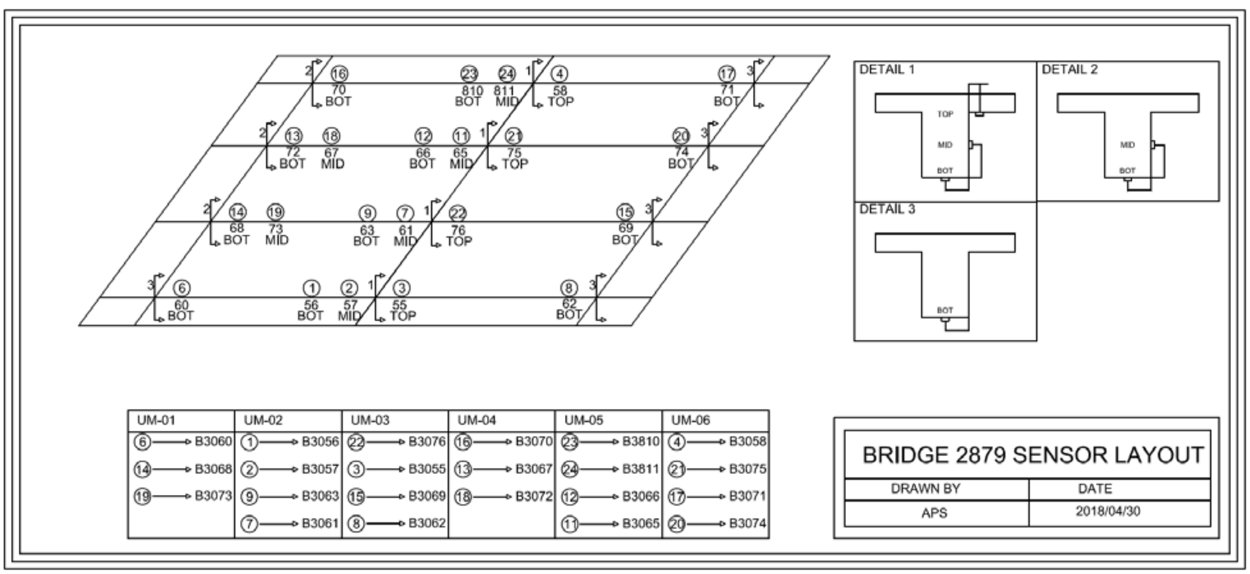

Figure 70: Bridge 2879 sensor layout

A.5.3 Loading

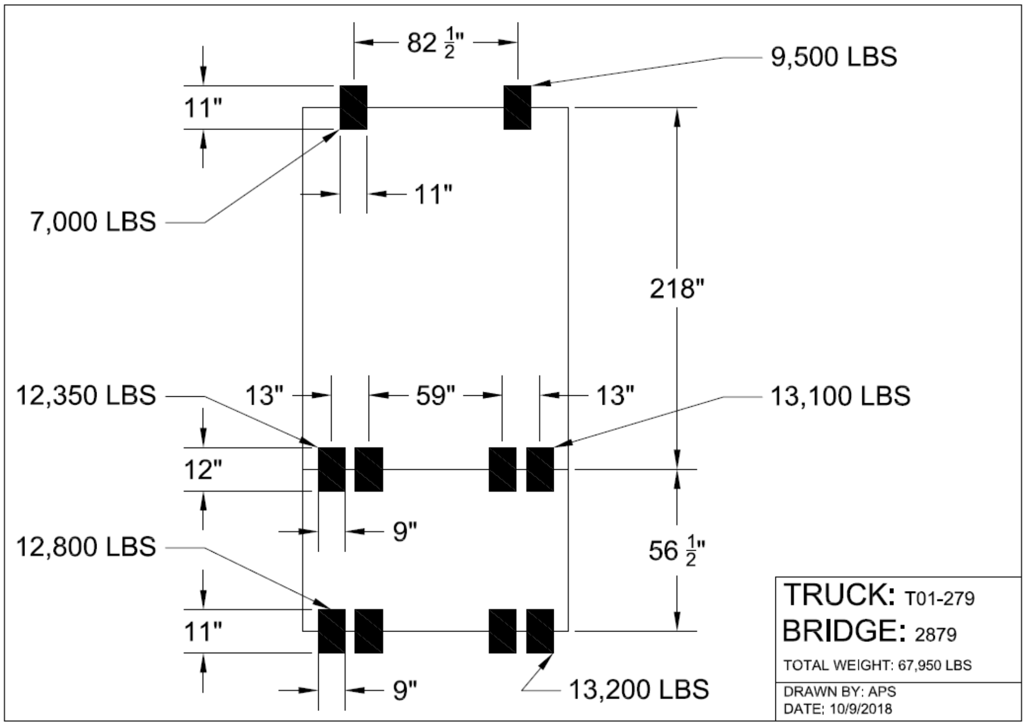

Figure 71: Bridge 2879 Truck T01-279 loading

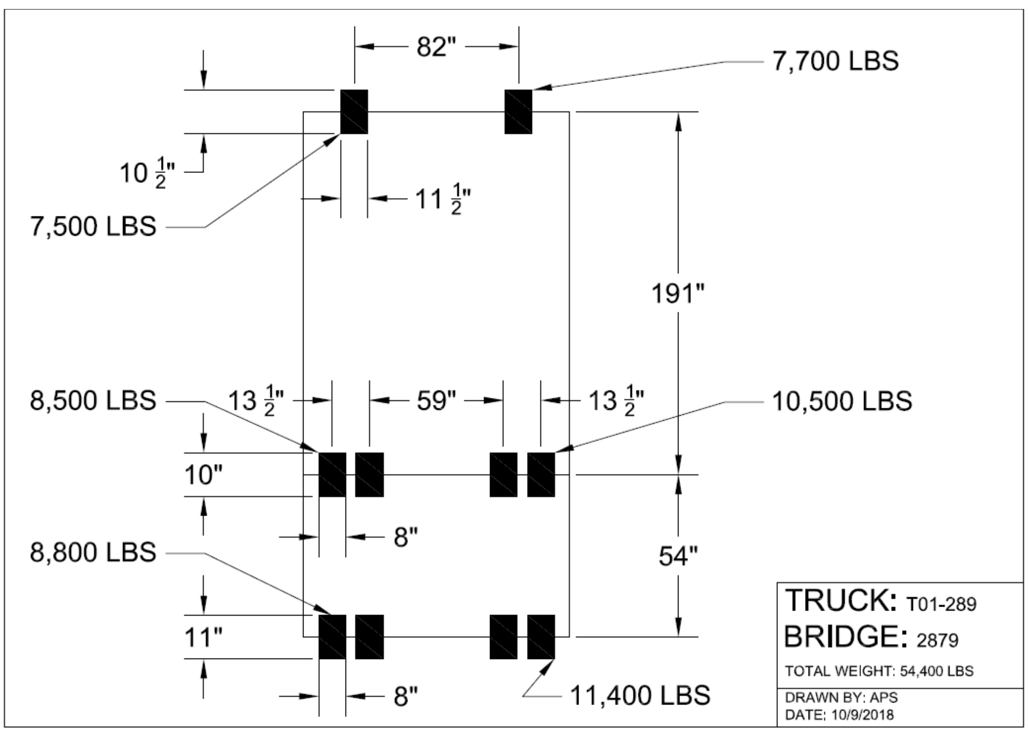

Figure 72: Bridge 2879 Truck T01-289 loading

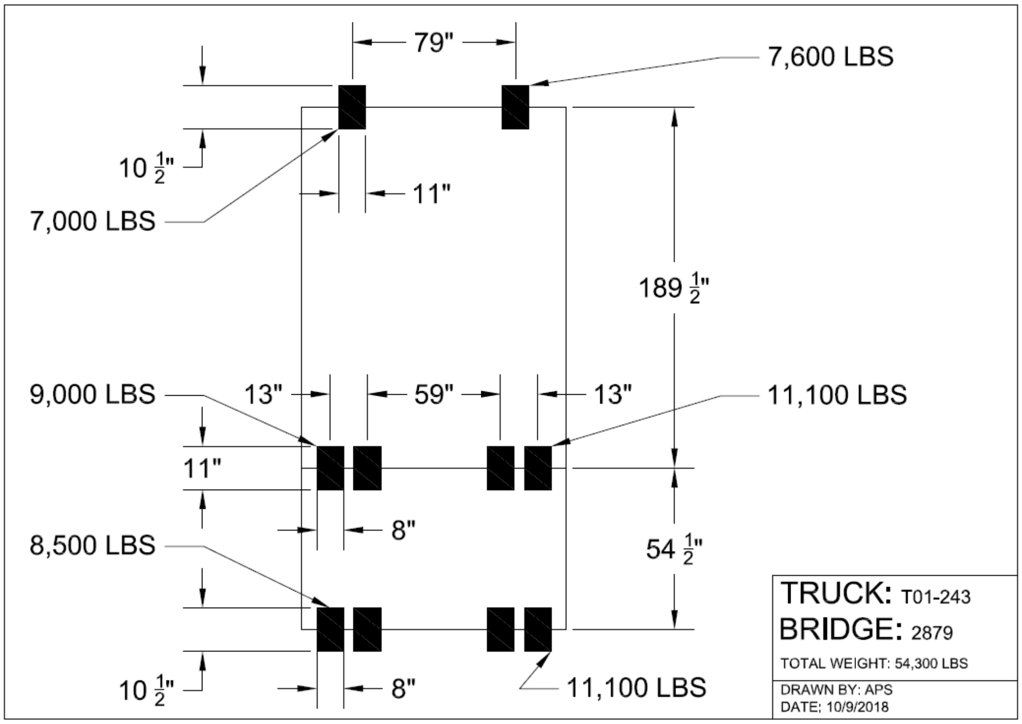

Figure 73: Bridge 2879 Truck T01-243 loading

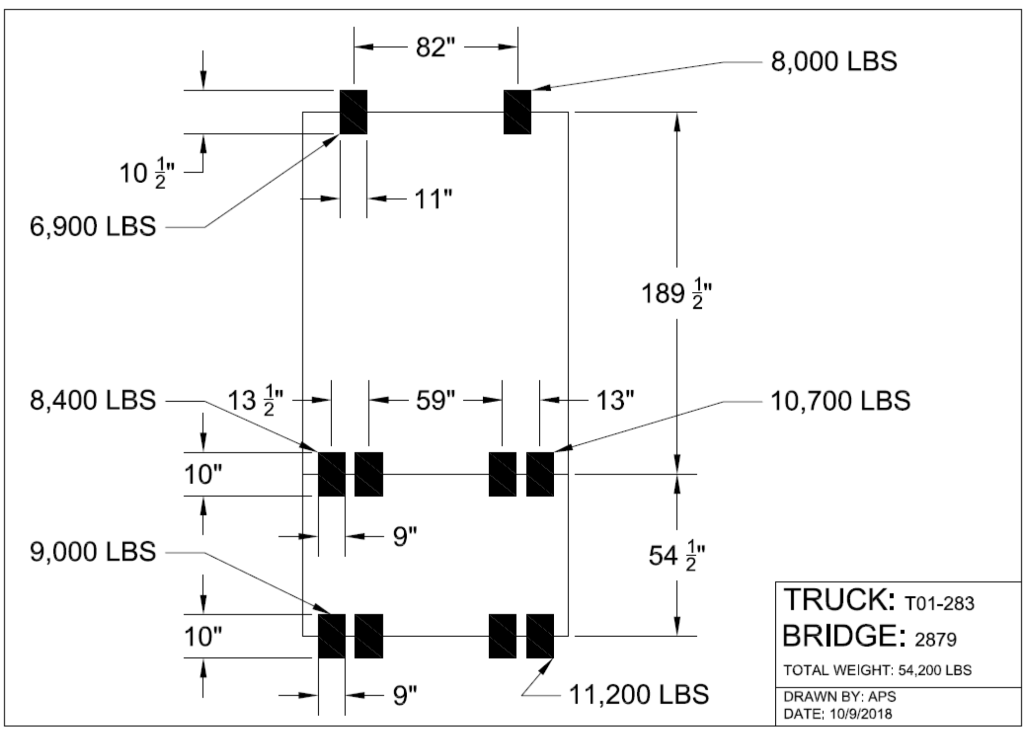

Figure 74: Bridge 2879 Truck T01-283 loading

A.5.4 Representative Data Plots

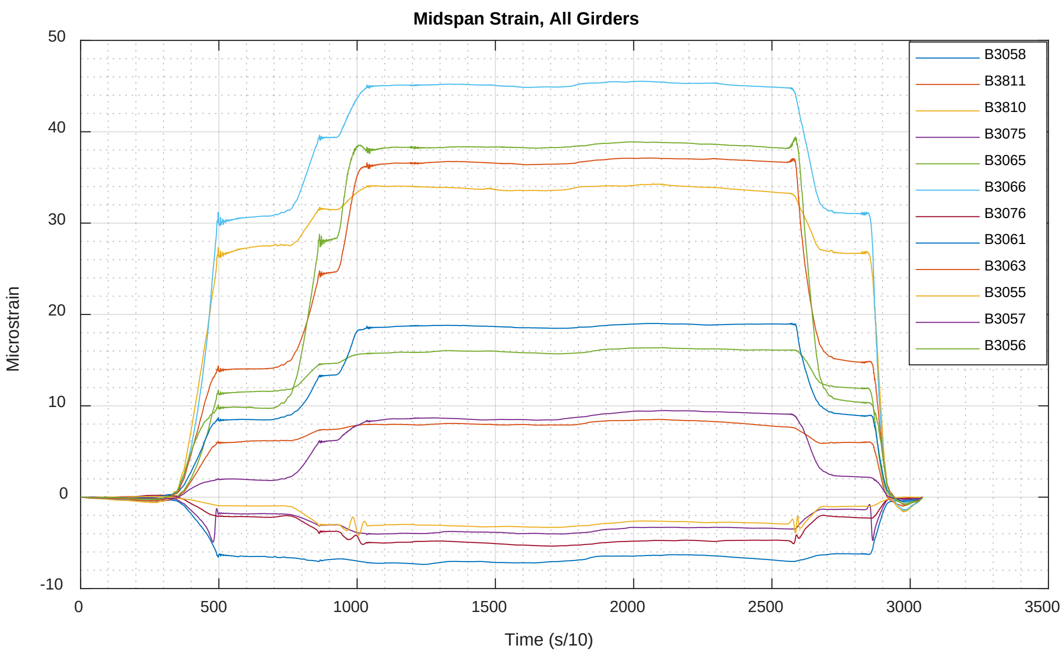

Figure 75: Bridge 2879 SBS_S_2_1 strains - Midspan

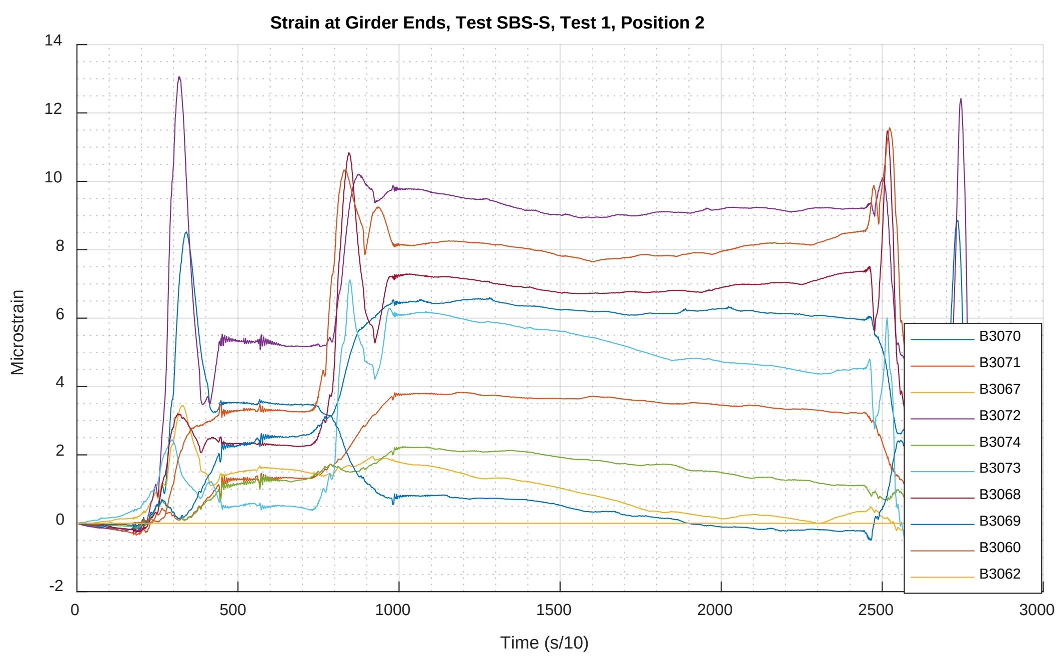

Figure 76: Bridge 2879 SBS_S_2_1 strains - Ends

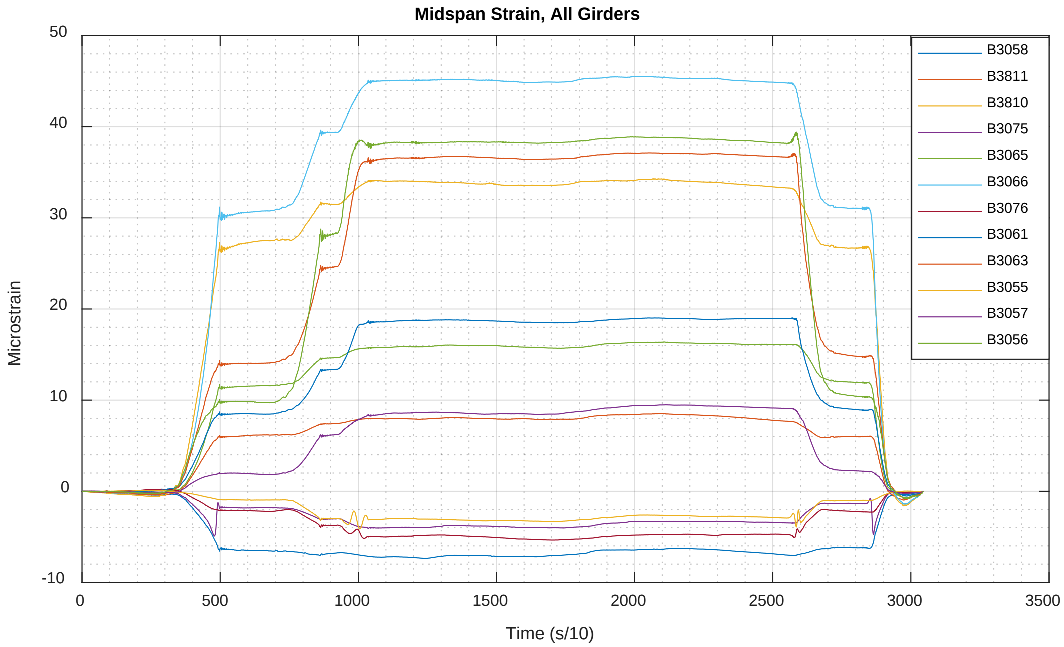

Figure 77: Bridge 2879 SBS_U_2_1 strains - Midspan

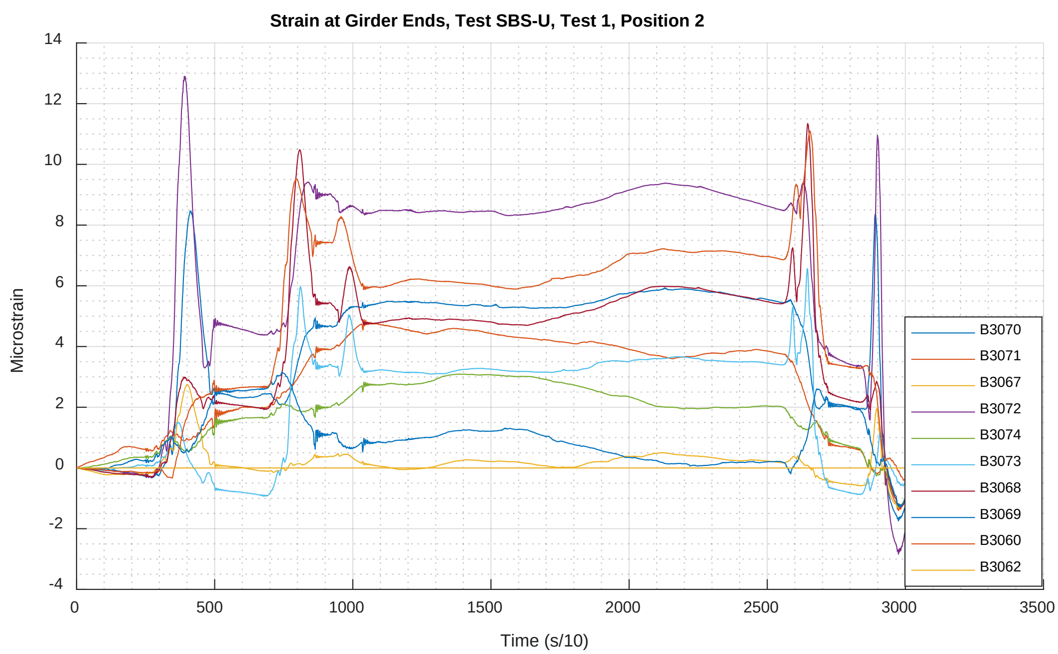

Figure 78: Bridge 2879 SBS_U_2_1 strains – Ends

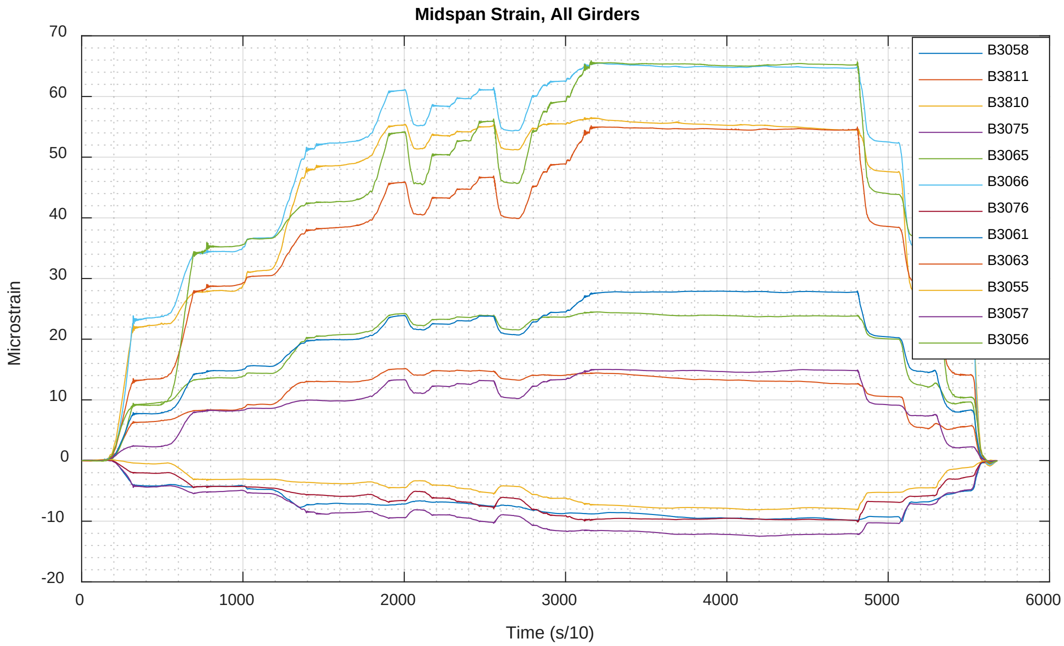

Figure 79: Bridge 2879 MAX_S_2_1 strains - Midspan

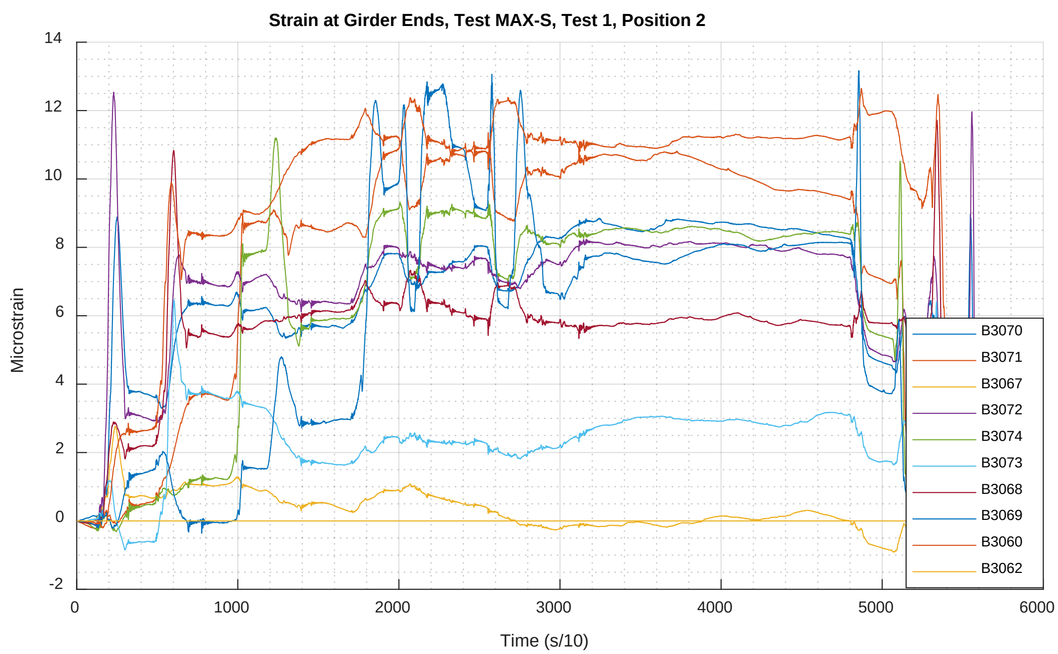

Figure 80: Bridge 2879 MAX_S_2_1 strains - Ends

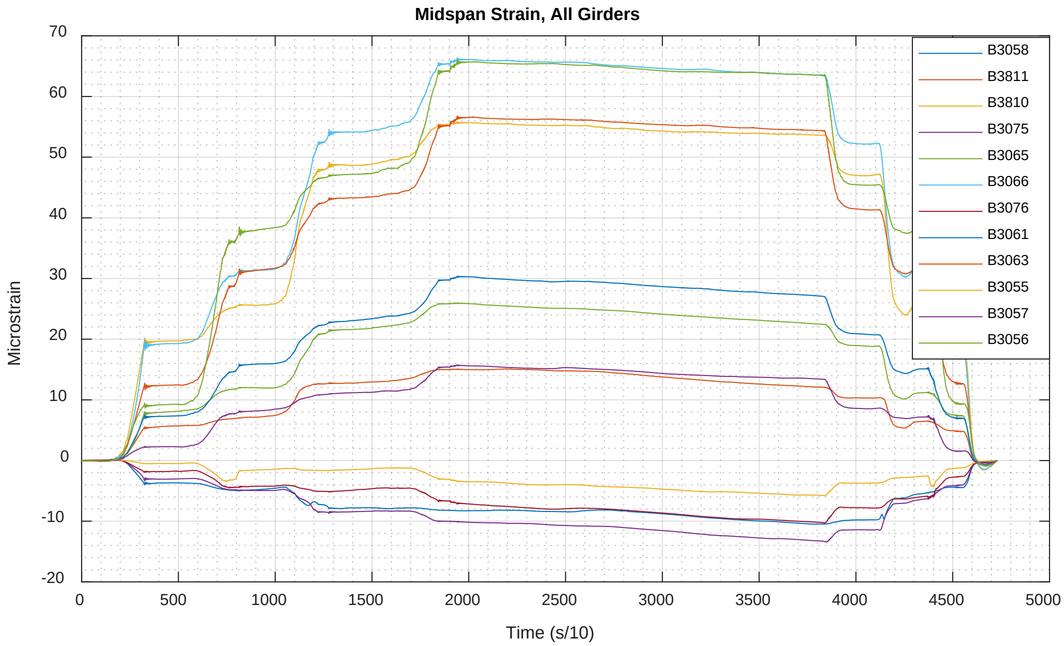

Figure 81: Bridge 2879 MAX_U_2_1 strains - Midspan

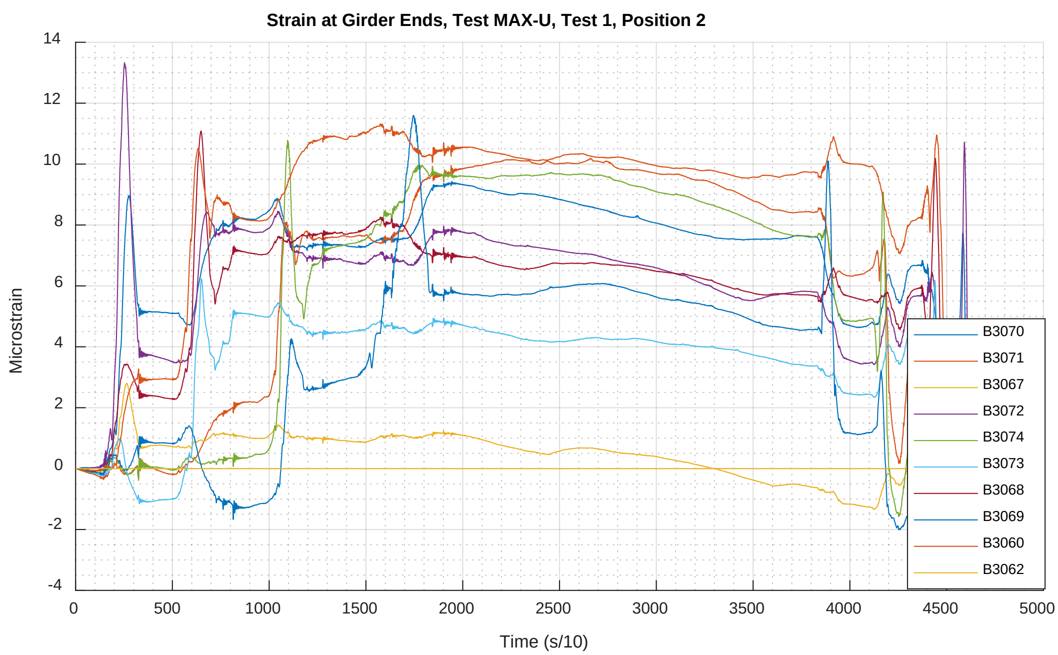

Figure 82: Bridge 2879 MAX_U_2_1 strains – Ends

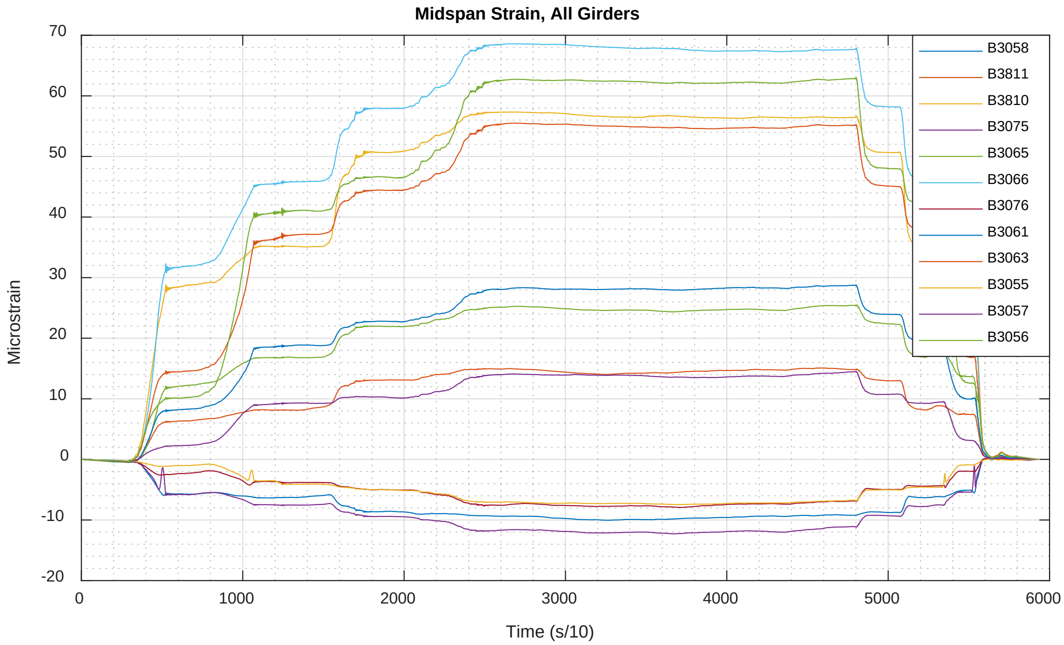

Figure 83: Bridge 2879 ALT_S_2_1 strains - Midspan

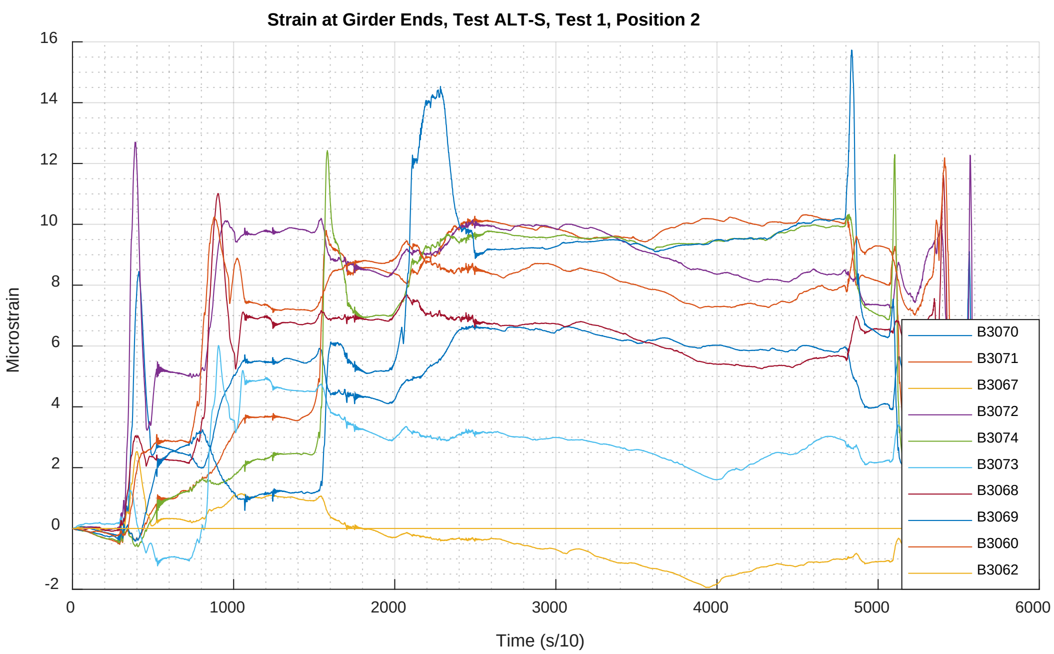

Figure 84: Bridge 2879 ALT_S_2_1 strains – Ends

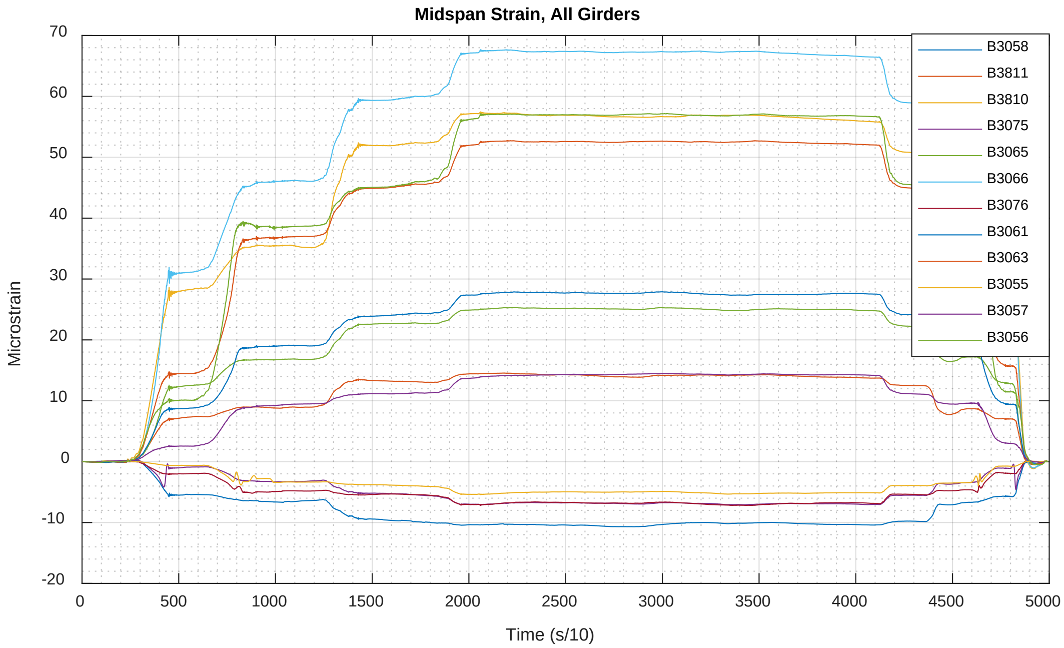

Figure 85: Bridge 2879 ALT_U_2_1 strains - Midspan

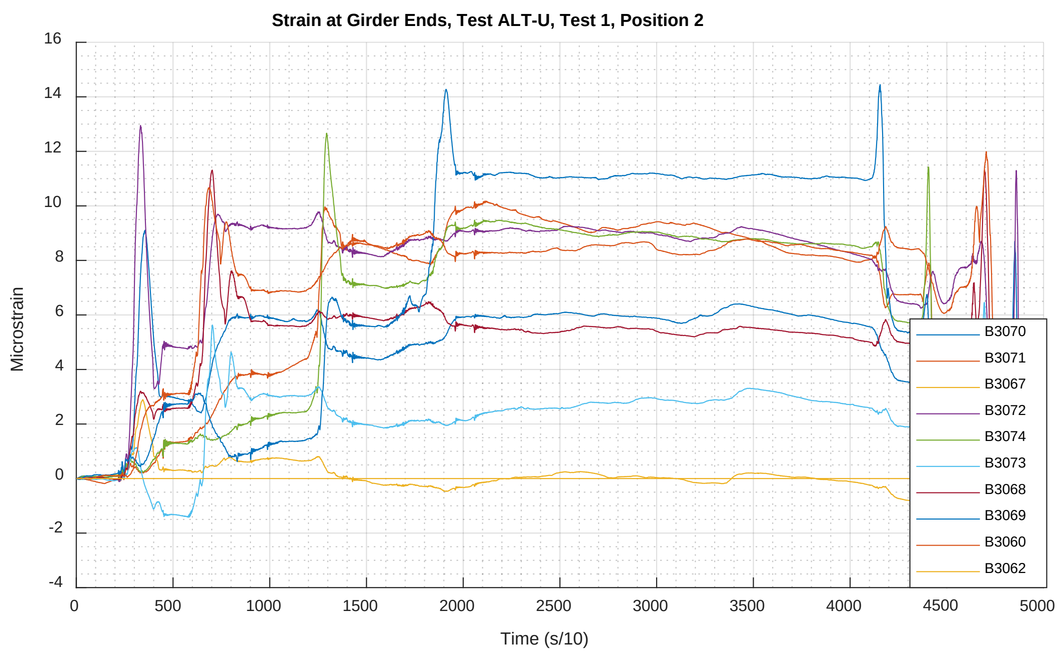

Figure 86: Bridge 2879 ALT_U_2_1 strains - Ends

A.5.5 Rating Factor Calculations

AASHTO Rating Calculations:

Bridge 2879 - Atkinson, Maine

Material Parameters:

Concrete Compressive Strength	$f'_c := 2.5\ ksi$
Reinforcement Yield Strength	$F_y := 33\ ksi$
Unit Weight: Reinforced Concrete	$\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$
Unit Weight: Wearing Surface	$\gamma_{ws} := 0.150\ \frac{kip}{ft^3}$

Geometric Properties:

Span Length	$L := 50\ ft$
Girder Spacing - Interior	$S := 90\ in$
Girder Spacing - Exterior	$S_x := 54\ in$
Number of Girders	$NG := 4$
Skew Angle	$skew := 30\ °$
Lane Width	$lanewidth := 10.5\ ft$
Number of Lanes	$Nlane := 2$
Wearing Surface Thickness	$ws := 4\ in$
Thickness of Pavement Overlay	$ws_2 := 0\ in$
Girder Height - Interior	$h := 50\ in$
Girder Height - Exterior	$h_x := 50\ in$
Deck Thickness	$d_s := 8\ in$
Web Width - Interior	$b_w := 22\ in$
Web Width - Exterior	$b_{wx} := 17\ in$
Curb Depth	$h_{curb} := 12\ in$
Curb Width	$b_{curb} := 18\ in$
Height to Centroid of Reinforcement - Interior	$y_{bar} := \begin{bmatrix} 3.333 \\ 3.875 \\ 5.518 \\ 3.875 \\ 3.333 \end{bmatrix} in$
Height to Centroid of Reinforcement - Exterior	$y_{barx} := \begin{bmatrix} 3.583 \\ 4.125 \\ 5.75 \\ 4.125 \\ 3.583 \end{bmatrix} in$
Area of Reinforcement - Interior	$A_s := \begin{bmatrix} 7.5938 \\ 12.656 \\ 17.719 \\ 12.656 \\ 7.5938 \end{bmatrix} in^2$

Area of Reinforcement - Exterior $A_{sx} := \begin{bmatrix} 9.375 \\ 12.5 \\ 18.75 \\ 12.5 \\ 9.375 \end{bmatrix} in^2$

Distance from Centerline of Girder to Edge of Curb $d_e := -6.5\ in$

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

Load and Analysis Parameters

Concentrated Load Due to Diaphragms on One Girder $P_{dint} := 2.833\ kip$

Location of Intermediate Diaphragm (Half, Third, Quarter) $loc_d :=$ "Third"

Distributed Load Due to Rail $w_{rail} := .219\ \frac{kip}{ft}$

Structural Dead Load Factor $\gamma_{DC} := 1.25$

Wearing Surface Dead Load Factor $\gamma_{DW} := 1.25$

Live Load Factor $\gamma_{LL} := 1.35$

Live Load Impact Factor $IM := 0.33$

Flexural Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 1.0$

Initial Calculations

Web Height - Interior $d_g := h - d_s$

Web Height - Exterior $d_{gx} := h_x - d_s$

Include Wearing Surface in Section Height $h := h + \text{if}\left(\gamma_{ws} = 0.15\ \frac{kip}{ft^3}, ws, 0\right) = 54\ in$

Depth to Centroid of Reinforcement - Interior $d := h - y_{bar}$

Depth to Centroid of Reinforcement - Exterior $d_x := h_x - y_{barx} + h_{curb}$

Moment Applied to Interor Girders from Diaphragm

$$M_d := \begin{array}{l} \text{if } loc_d = \text{"Half"} \\ \quad \left\| P_{dint} \cdot \frac{L}{4} \right. \\ \text{else if } loc_d = \text{"Third"} \\ \quad \left\| P_{dint} \cdot \frac{L}{3} \right. \\ \text{else if } loc_d = \text{"Quarter"} \\ \quad \left\| P_{dint} \cdot \frac{L}{4} + P_{dint} \cdot \frac{L}{4} \right. \end{array} = 47.217\ ft \cdot kip$$

Moment Applied to Exterior Girders from Diaphragm $M_{dx} := \frac{M_d}{2} = 23.608\ ft \cdot kip$

Distribution Factors

Dictance Between Centroids of Deck and Web $e_g := \frac{d_g + d_s}{2} = 25\ in$

Area of Web $A := d_g \cdot b_w = 924\ in^2$

Moment of Inertia of Web $I := \frac{b_w \cdot d_g^{\ 3}}{12} = \left(1.358 \cdot 10^5\right)\ in^4$

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

Longitudinal Stiffness Parameter $K_g := n \cdot \left(I + A \cdot e_g^{\ 2}\right) = \left(7.133 \cdot 10^5\right)\ in^4$

Interior Moment Distribution Factor - 1 Lane $g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.54$

Interior Moment Distribution Factor - 2 Lane $g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.721$

Controling Interior Moment Distribution Factor $g_m := \max\left(g_{m1}, g_{m2}\right)$

Roadway Width $W_r := lanewidth \cdot Nlane$

Eccentricity of Design Lane From C.G. of Girders $e_1 := \frac{W_r}{2} - 5\ ft + exc = 5.5\ ft$

Eccentricity of Exterior Girder From C.G. of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 11.25\ ft$

Eccentricity of Each Girder

$x_1 := X_{ext}$

$x_2 := X_{ext} - S$

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

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

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

Lever Rule Distribution Factor - One Lane $R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.47$

$g_{mR1} := \mathbf{if}\left(P_{dint} > 0, 1.2 \cdot R_1, 0\right) = 0.564$

Lever Rule Distribution Factor - Two Lanes $R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot \left(e_1 - 5\ ft\right)}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.52$

$g_{mR2} := \mathbf{if}\left(P_{dint} > 0, R_2, 0\right) = 0.52$

Exterior Moment Distribution Factor $g_{mx1} := \frac{1.2\ \left(S + d_e - 2\ ft\right)}{2 \cdot S} = 0.397$

$ee := 0.77 + \frac{d_e}{9.1\ ft} = 0.71$

$g_{mx2} := g_{m2} \cdot ee = 0.512$

$g_{mx} := \max(g_{mx1}, g_{mx2}) = 0.512$

Skew Correction Factor

$$c_1 := 0.25 \cdot \left(\frac{K_g}{12 \cdot L \cdot d_s^{\,3}}\right)^{0.25} \cdot \left(\frac{S}{L}\right)^{.5} = 0.064$$

$\theta := \mathbf{if}(skew \geq 30\,°, skew, 0\,°)$

$C_\theta := 1 - c_1 \cdot (\tan(\theta))^{1.5} = 0.972$

$g_m := g_m \cdot C_\theta = 0.701$

$g_{mx} := g_{mx} \cdot C_\theta = 0.498$

Interior DF	Exterior DF
$g_m = 0.701$	$g_{mx} = 0.498$

Loading

Interior Girder Dead Load $\quad w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.963\ \frac{kip}{ft}$

Deck Dead Load $\quad w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.75\ \frac{kip}{ft}$

Curb Dead Load $\quad w_{curb} := 2 \cdot \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.45\ \frac{kip}{ft}$

Dead Load from Nonstructural Components $\quad w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.332\ \frac{kip}{ft}$

Total Structural Dead Load on Interior Girders $\quad DC := w_{girder} + w_{deck} + w_{ns} = 2.044\ \frac{kip}{ft}$

Exterior Girder Dead Load $\quad w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.744\ \frac{kip}{ft}$

Exterior Deck Dead Load $\quad w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.45\ \frac{kip}{ft}$

Total Structural Dead Load on Exterior Girders $\quad DC_x := w_{girderx} + w_{deckx} + w_{ns} = 1.525\ \frac{kip}{ft}$

Wearing Surface Dead Load on Interior Girders $\quad DW := \gamma_{ws} \cdot (ws + ws_2) \cdot S = 0.375\ \frac{kip}{ft}$

Wearing Surface Dead Load on Exterior Girders $\quad DW_x := \gamma_{ws} \cdot (ws + ws_2) \cdot S_x = 0.225\ \frac{kip}{ft}$

Dead Load Moments $\quad M_{DC} := \frac{DC \cdot L^2}{8} + M_d \qquad M_{DCx} := \frac{DC_x \cdot L^2}{8} + M_{dx}$

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

$M_{DC} = 685.967\ ft \cdot kip$ $M_{DCx} = 500.249\ ft \cdot kip$

$M_{DW} = 117.188\ ft \cdot kip$ $M_{DWx} = 70.313\ ft \cdot kip$

Live Load Moment - Truck Load $M_{Truck} := 32\ kip \cdot \left(\frac{L}{4}\right) + \frac{40\ kip}{2} \cdot \left(\frac{L}{2} - 14\ ft\right) = 620\ ft \cdot kip$

Live Load Moment - Tandem $M_{Tandem} := 25\ kip \cdot \frac{L}{4} + \frac{25\ kip}{2} \cdot \left(\frac{L}{2} - 4\ ft\right) = 575\ ft \cdot kip$

Live Load Moment - Lane $M_{Lane} := 0.64\ \frac{kip}{ft} \cdot \frac{L^2}{8} = 200\ ft \cdot kip$

Total HL-93 Live Load $M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right)$

$M_{LL} = \left(1.025 \cdot 10^3\right)\ ft \cdot kip$

Nominal Resistance

Depth Whitney Stress Block - Interior $a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 1.31 \\ 2.184 \\ 3.057 \\ 2.184 \\ 1.31 \end{bmatrix} in$

Nominal Moment Resistance - Interior $M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 1.044 \cdot 10^3 \\ 1.707 \cdot 10^3 \\ 2.288 \cdot 10^3 \\ 1.707 \cdot 10^3 \\ 1.044 \cdot 10^3 \end{bmatrix} ft \cdot kip$

Interior Nominal Moment Capacity

$M_{capacity} := \max\left(M_n\right) = \left(2.288 \cdot 10^3\right)\ ft \cdot kip$

Depth Whitney Stress Block - Exterior $a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 2.696 \\ 3.595 \\ 5.392 \\ 3.595 \\ 2.696 \end{bmatrix} in$

Nominal Moment Resistance - Exterior $M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 1.471 \cdot 10^3 \\ 1.928 \cdot 10^3 \\ 2.761 \cdot 10^3 \\ 1.928 \cdot 10^3 \\ 1.471 \cdot 10^3 \end{bmatrix} (ft \cdot kip)$

Exterior Nominal Moment Capacity

$M_{capacityx} := \max\left(M_{nx}\right) = \left(2.761 \cdot 10^3\right)\ ft \cdot kip$

Rating Factors

Interior Moment Rating Factor

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

Exterior Moment Rating Factor

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

Interior

$RF_{Interior} = 1.089$

Exterior

$RF_{Exterior} = 2.574$

Rating Factor Improvements

Concrete Compressive Strength - Larger is More Conservative	$f'_c := 5\ ksi$
Concrte Elastic Modulus	$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = (4.07 \cdot 10^3)\ ksi$

Interior Girders

Maximum Recorded Strain	$\varepsilon_T := 62.49 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 946.9\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 21210\ in^3$
Cracked Section Modulus	$S_{cr} := 8413\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \text{if}\ (Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 9.223 \cdot 10^{-5}$
Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.476$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.924$
Test Understanding Factor	$k_b := \text{if}\ (r_M > 0.7, 0.5, 0) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.238$
Improved Rating Factor	$RF_{Improved} := RF_{Interior} \cdot k = 1.348$

Exterior Girders

Maximum Recorded Strain	$\varepsilon_T := 77.32 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 946.9\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 15738\ in^3$
Cracked Section Modulus	$S_{cr} := 4196\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \text{if}\ (Behavior = \text{"Uncracked"}, S_{unc}, S_{cr})$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 8.831 \cdot 10^{-5}$

Test Benefit Factor $k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.142$

Ratio of Applied to HL-93 Moment $r_M := \frac{M_{Max}}{M_{LL}} = 0.924$

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

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.071$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 2.757$

Improved Rating Factors

Interior
$RF_{Improved} = 1.348$

Exterior
$RF_{ImprovedExt} = 2.757$

Figure 87: Bridge 2879 calculations

A.6 Columbia No. 3848

A.6.1 Experimental Configuration and Experimental Data Collected

Table 21:Bridge 3848 experimental configuration and experimental data collected

File Contents	***File Name***	***File Type***
Sensors	Br3307 _Sensors.csv	CSV Format
Sensor Layout	Br3307_SensorLayout.mat	MATLAB Data File
Sensor Data	Br3848_ALT_S_2_1_Strain.mat	MATLAB Data File
	Br3848_ALT_U_2_1_Strain.mat	MATLAB Data File
	Br3848_MAX_S_1_1_Strain.mat	MATLAB Data File
	Br3848_MAX_S_2_1_Strain.mat	MATLAB Data File
	Br3848_MAX_S_3_1_Strain.mat	MATLAB Data File
	Br3848_MAX_U_2_1_Strain.mat	MATLAB Data File
	Br3848_SBS_S_2_1_Strain.mat	MATLAB Data File
	Br3848_SBS_U_2_1_Strain.mat	MATLAB Data File

A.6.2 Instrumentation

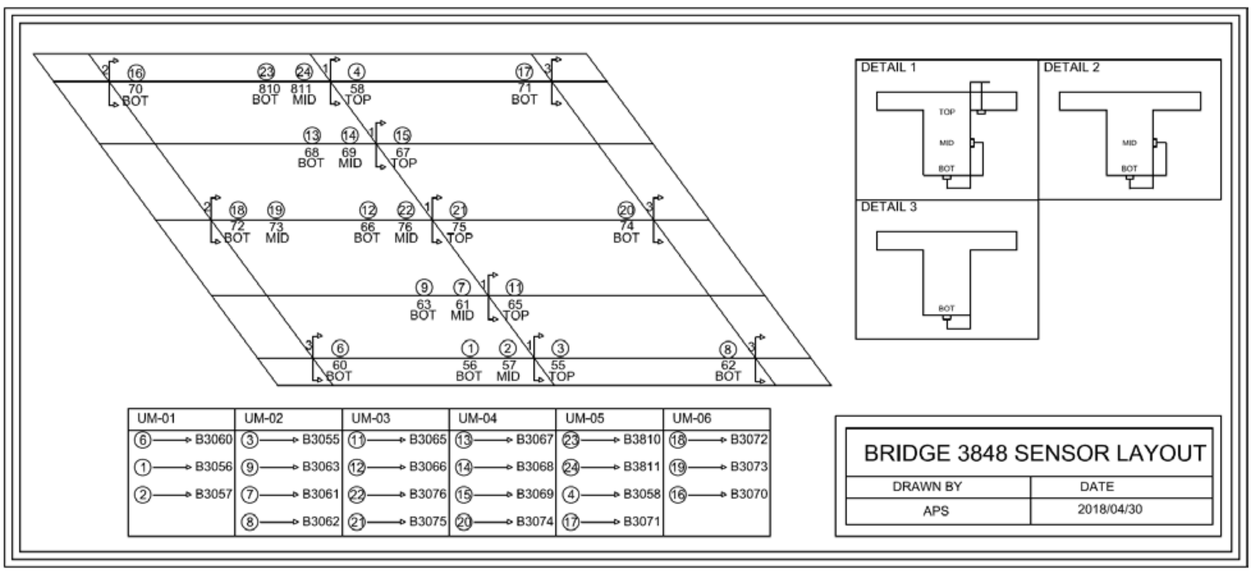

Figure 88: Bridge 3848 sensor layout

A.6.3 Loading

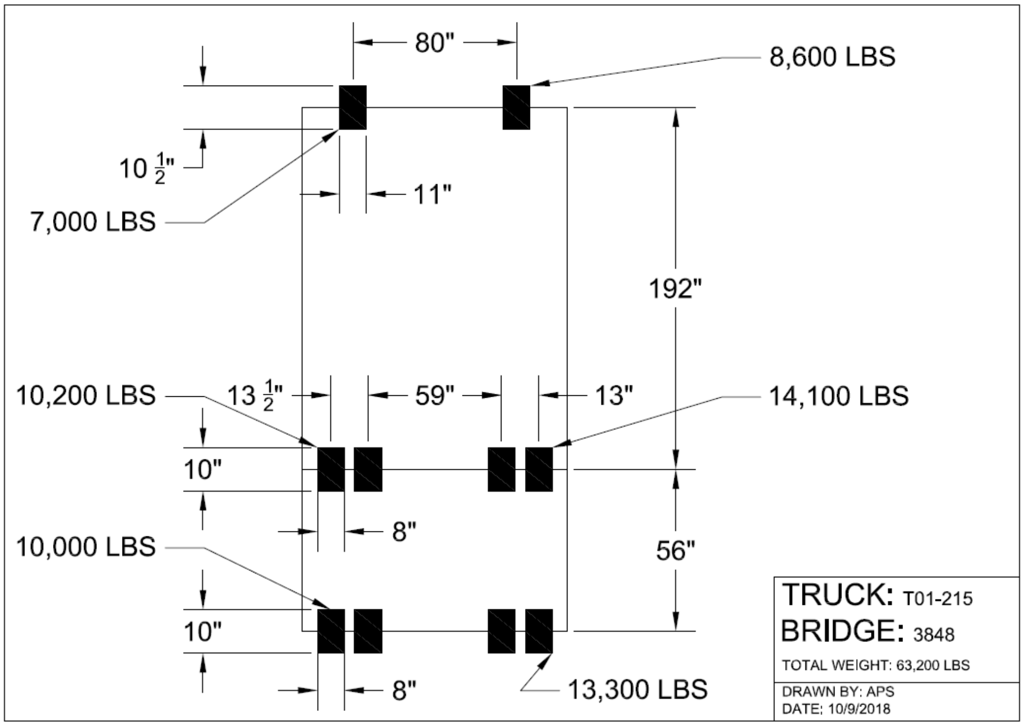

Figure 89: Bridge 3848 Truck T01-215 loading

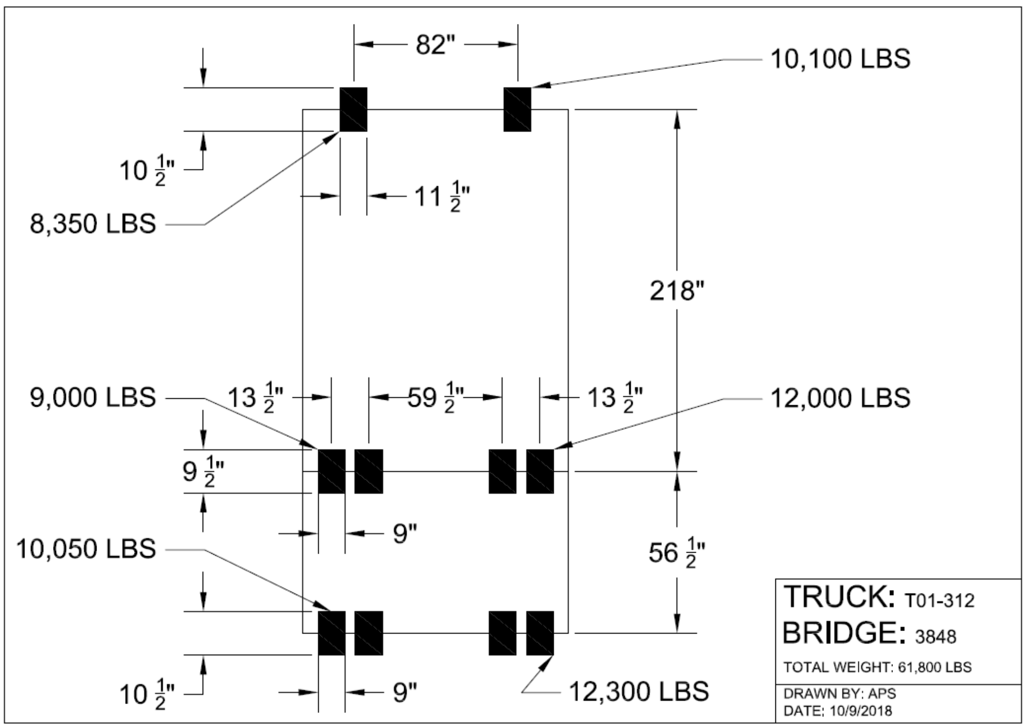

Figure 90: Bridge 3848 Truck T01-312 loading

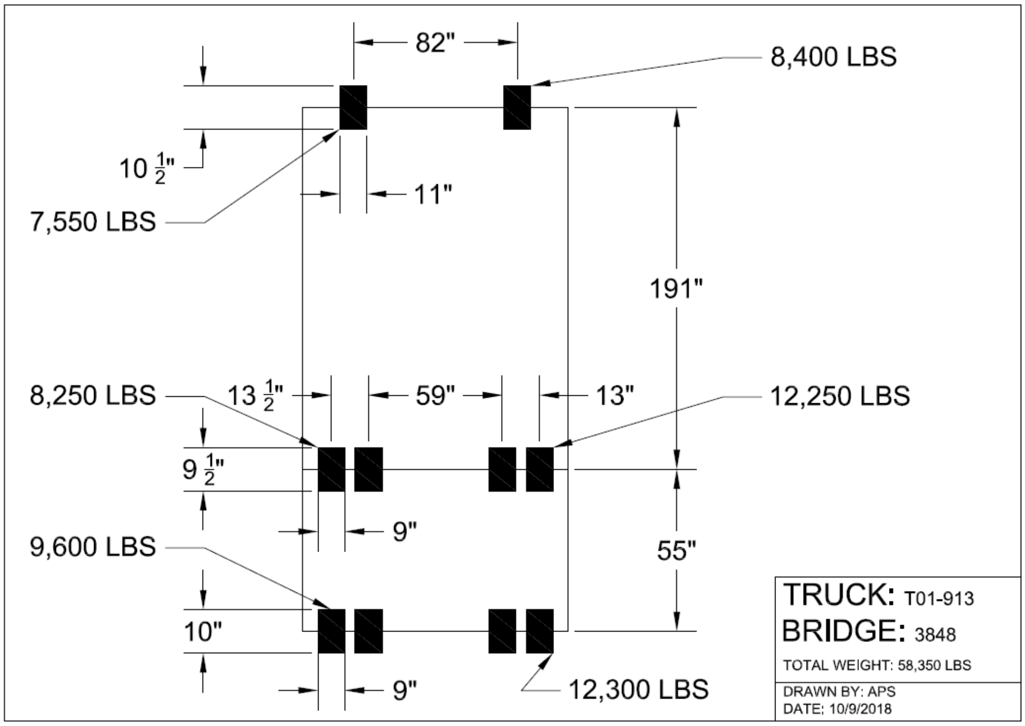

Figure 91: Bridge 3848 Truck T01-913 loading

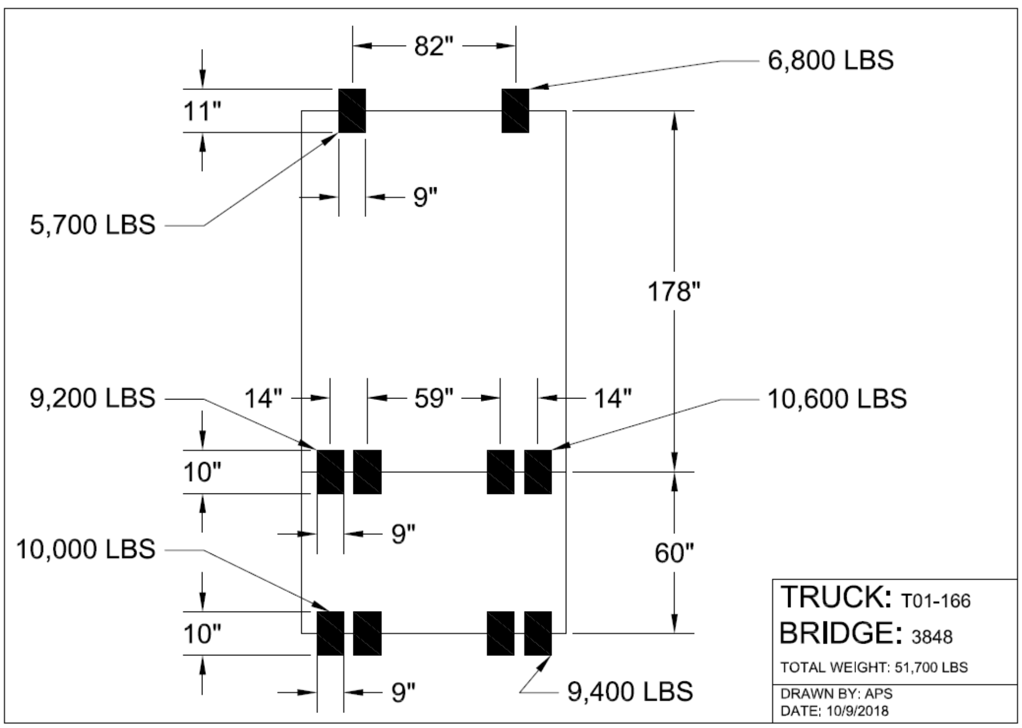

Figure 92: Bridge 3848 Truck T01-166 loading

A.6.4 Representative Data Plots

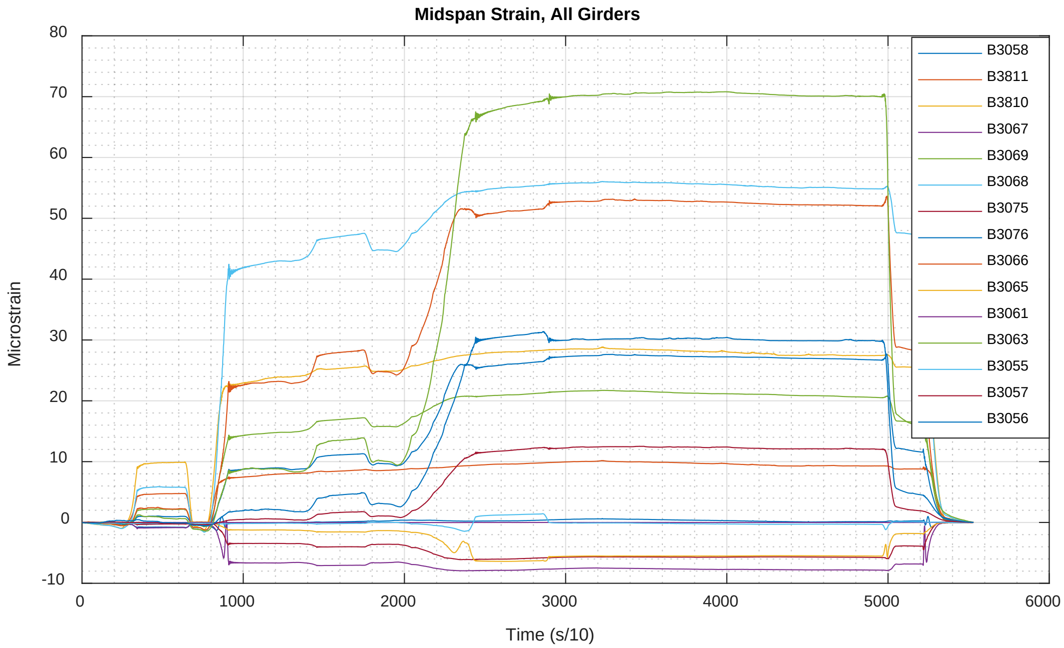

Figure 93: Bridge 3848 SBS_S_2_1 strains - Midspan

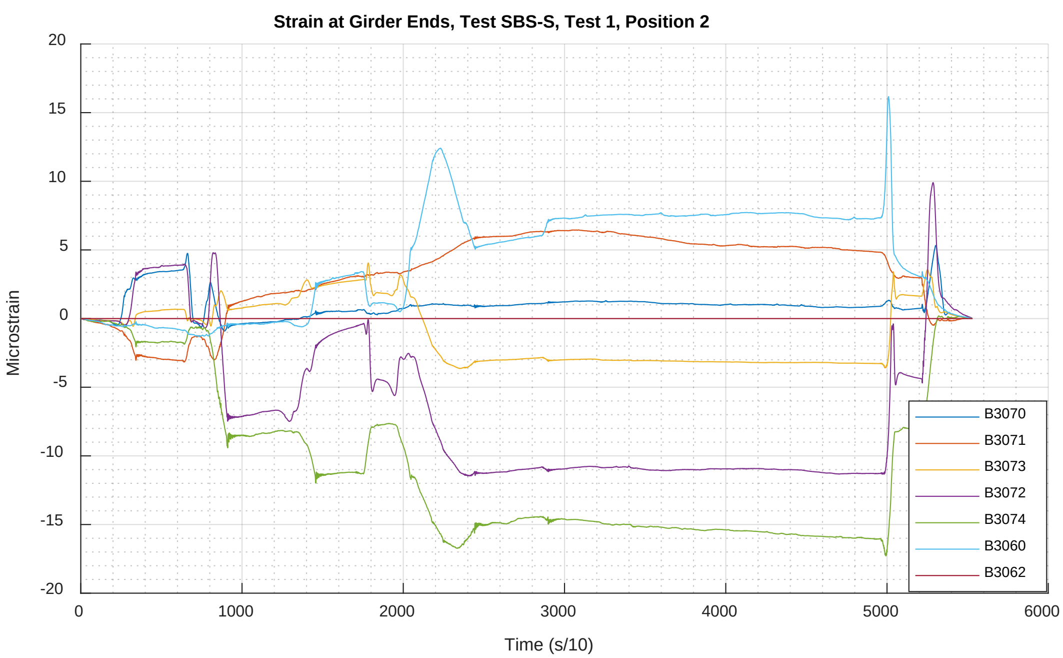

Figure 94: Bridge 3848 SBS_S_2_1 strains – Ends

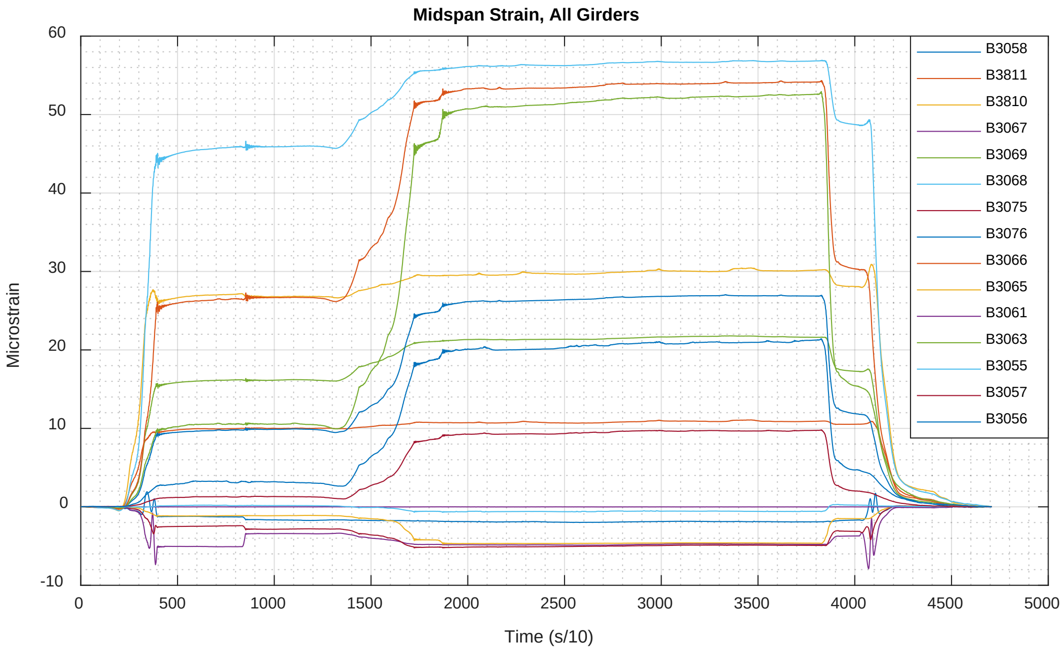

Figure 95: Bridge 3848 SBS_U_2_1 strains - Midspan

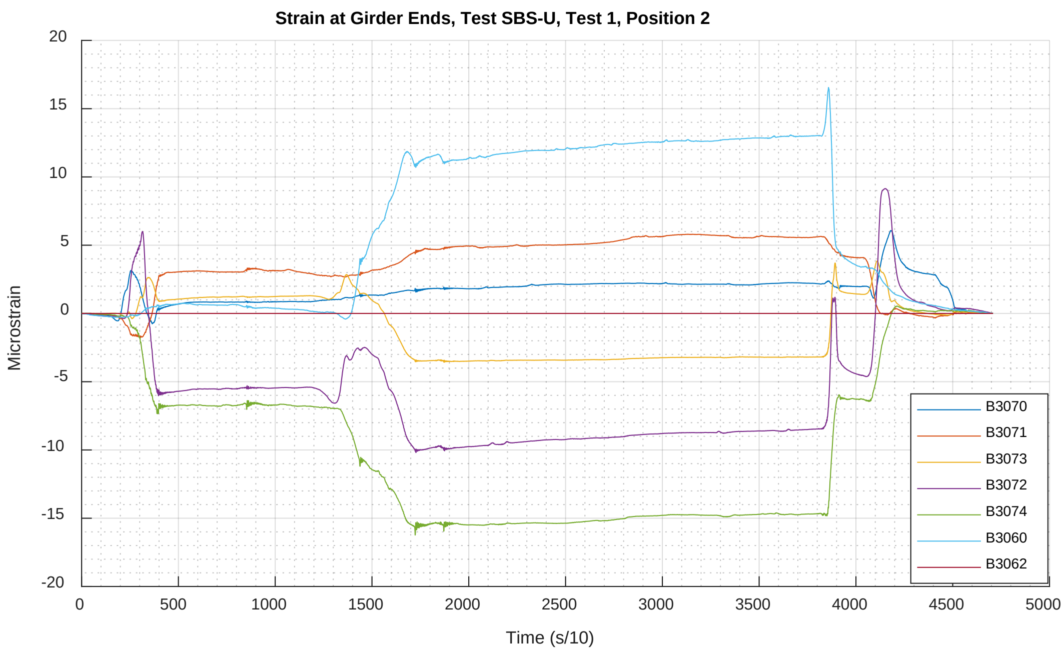

Figure 96: Bridge 3848 SBS_U_2_1 strains - Ends

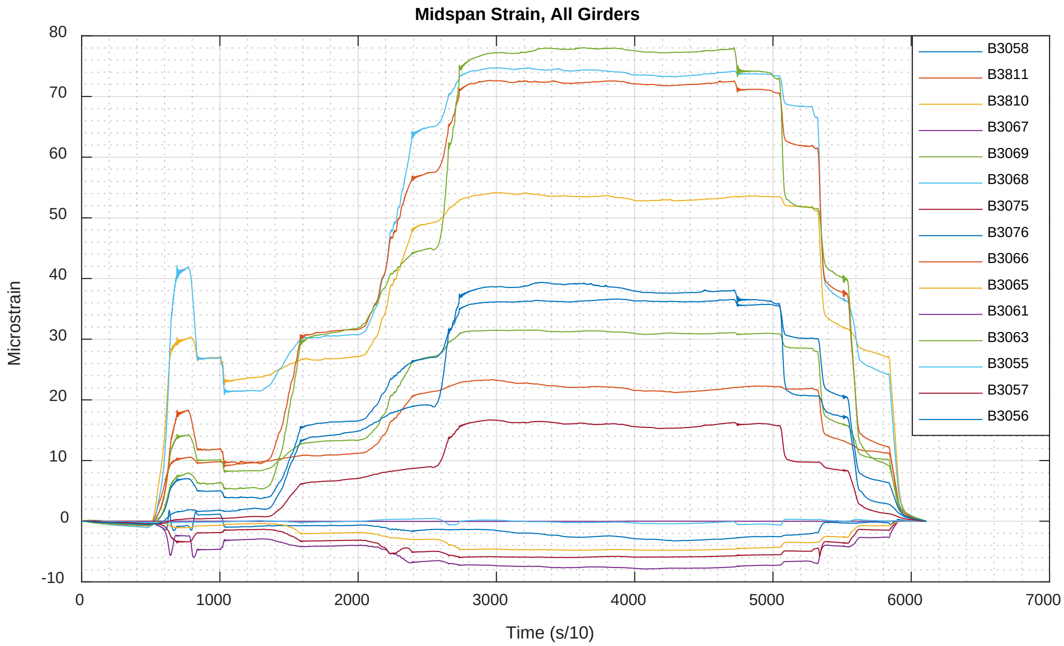

Figure 97: Bridge 3848 MAX_S_2_1 strains - Midspan

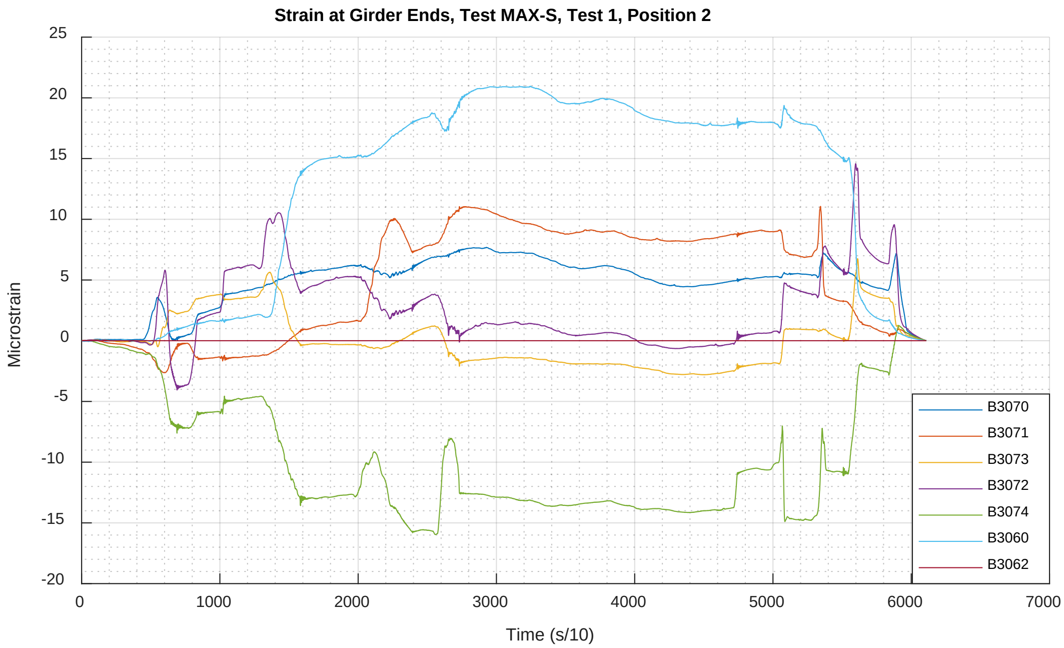

Figure 98: Bridge 3848 MAX_S_2_1 strains - Ends

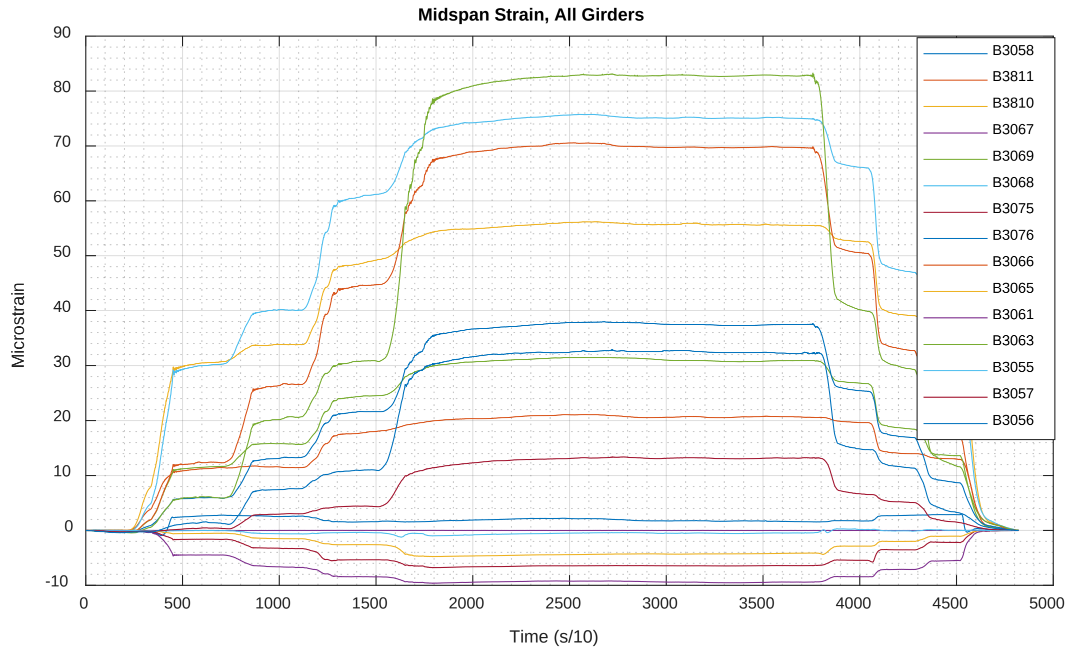

Figure 99: Bridge 3848 MAX_U_2_1 strains - Midspan

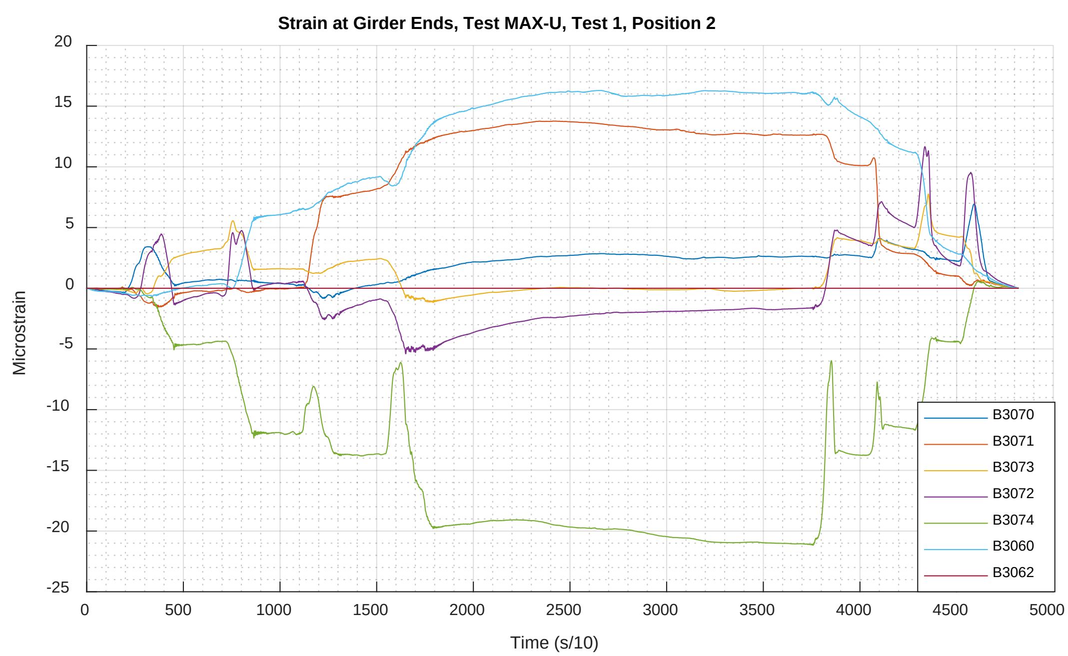

Figure 100: Bridge 3848 MAX_U_2_1 strains – Ends

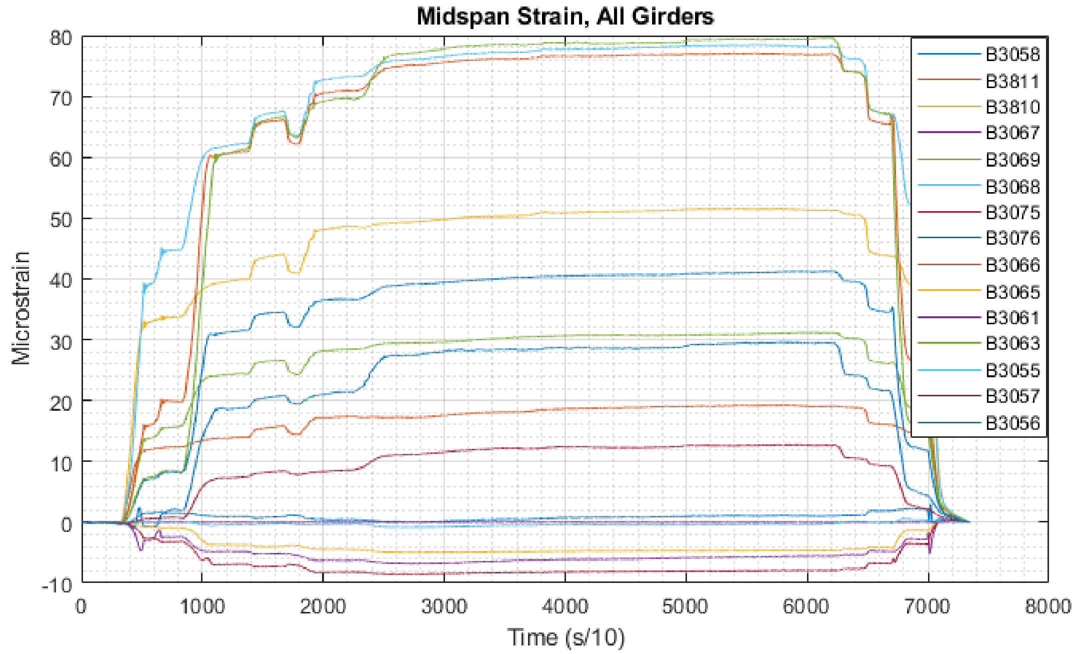

Figure 101: Bridge 3848 ALT_S_2_1 strains - Midspan

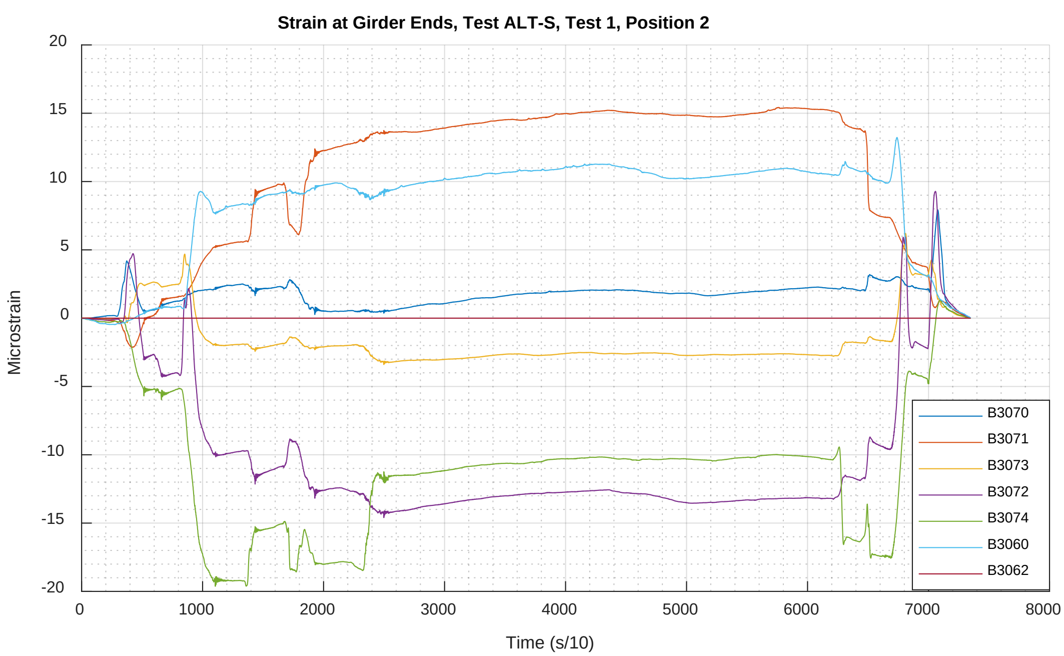

Figure 102: Bridge 3848 ALT_S_2_1 strains - Ends

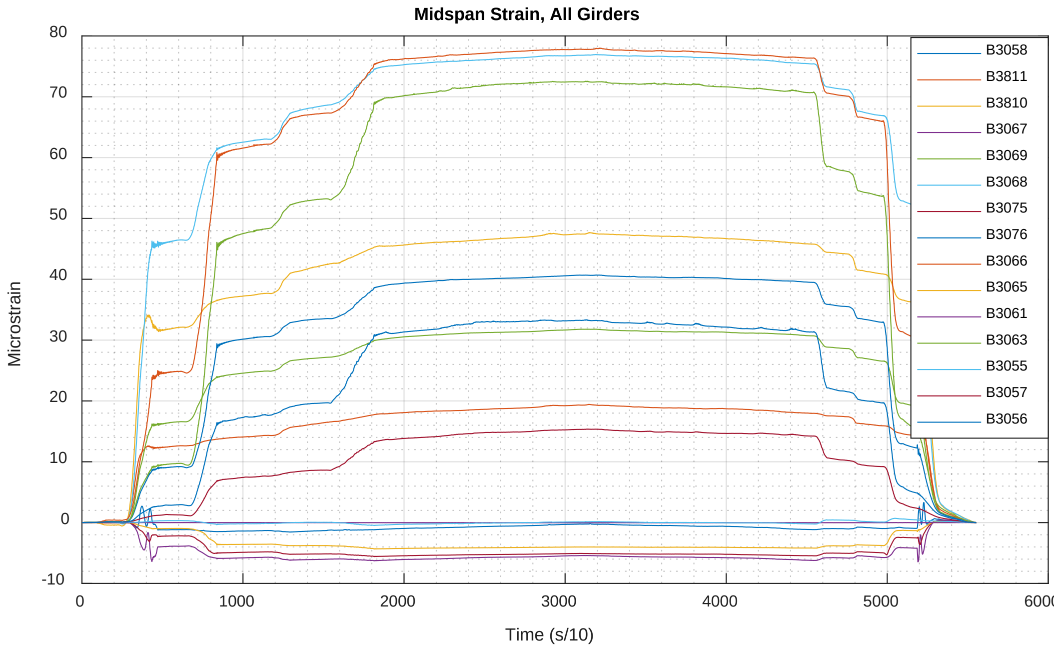

Figure 103: Bridge 3848 ALT_U_2_1 strains - Midspan

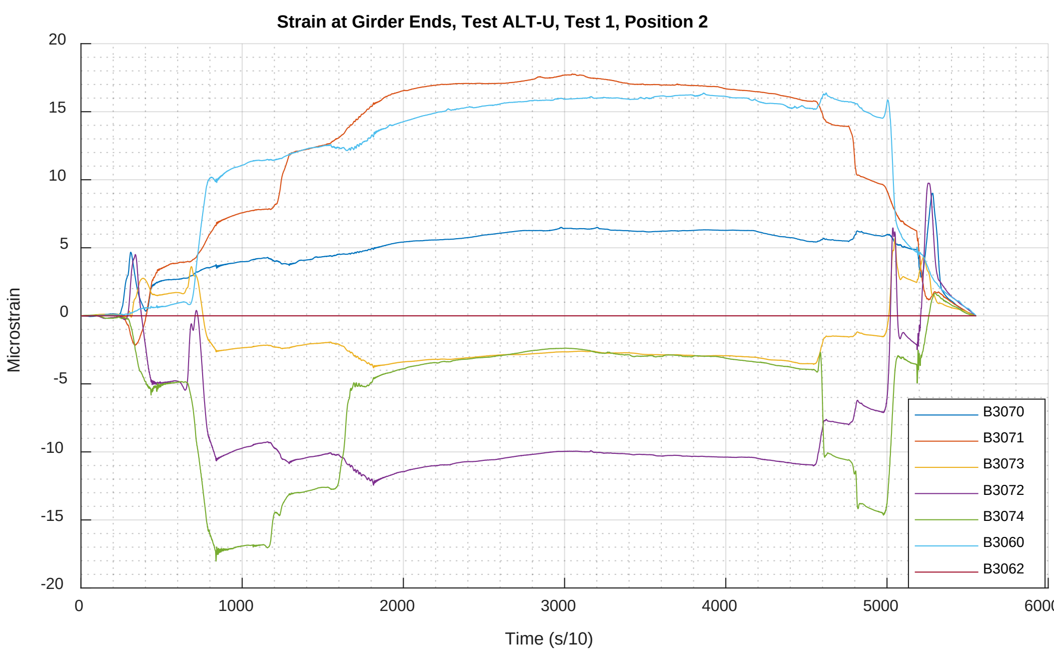

Figure 104: Bridge 3848 ALT_U_2_1 strains - Ends

A.6.5 Rating Factor Calculations

AASHTO Rating Calculations:

Bridge 3848 - Columbia, Maine

Material Parameters:

Concrete Compressive Strength	$f'_c := 2.5\ ksi$
Reinforcement Yield Strength	$F_y := 33\ ksi$
Unit Weight: Reinforced Concrete	$\gamma_{RC} := 0.150\ \frac{kip}{ft^3}$
Unit Weight: Wearing Surface	$\gamma_{ws} := 0.150\ \frac{kip}{ft^3}$

Geometric Properties:

Span Length	$L := 34\ ft$
Girder Spacing - Interior	$S := 70.38\ in$
Girder Spacing - Exterior	$S_x := 45.19\ in$
Number of Girders	$NG := 5$
Skew Angle	$skew := 30\ °$
Lane Width	$lanewidth := 11\ ft$
Number of Lanes	$Nlane := 2$
Wearing Surface Thickness	$ws := 3\ in$
Thickness of Pavement Overlay	$ws_2 := 1\ in$
Girder Height - Interior	$h := 29.75\ in$
Girder Height - Exterior	$h_x := 29.75\ in$
Deck Thickness	$d_s := 5.75\ in$
Web Width - Interior	$b_w := 19.5\ in$
Web Width - Exterior	$b_{wx} := 16\ in$
Curb Depth	$h_{curb} := 12\ in$
Curb Width	$b_{curb} := 18\ in$
Height to Centroid of Reinforcement - Interior	$y_{bar} := \begin{bmatrix} 3.188 \\ 4.115 \\ 4.977 \\ 4.115 \\ 3.188 \end{bmatrix} in$
Height to Centroid of Reinforcement - Exterior	$y_{barx} := \begin{bmatrix} 3.188 \\ 4.888 \\ 5.313 \\ 4.888 \\ 3.188 \end{bmatrix} in$
Area of Reinforcement - Interior	$A_s := \begin{bmatrix} 5.0265 \\ 7.594 \\ 10.125 \\ 7.594 \\ 5.0625 \end{bmatrix} in^2$

Area of Reinforcement - Exterior $A_{sx} := \begin{bmatrix} 3.80 \\ 6.33 \\ 7.59 \\ 6.33 \\ 3.80 \end{bmatrix} in^2$

Distance from Centerline of Girder to Edge of Curb $d_e := -7\ in$

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

Load and Analysis Parameters

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

Location of Intermediate Diaphragm (Half, Third, Quarter) $loc_d :=$ "Half"

Distributed Load Due to Rail $w_{rail} := 0.0121\ \frac{kip}{ft}$

Structural Dead Load Factor $\gamma_{DC} := 1.25$

Wearing Surface Dead Load Factor $\gamma_{DW} := 1.25$

Live Load Factor $\gamma_{LL} := 1.35$

Live Load Impact Factor $IM := 0.33$

Flexural Resistance Factor $\phi := .9$

System Factor $\phi_s := 1.0$

Condition Factor $\phi_c := 1.0$

Initial Calculations

Web Height - Interior $d_g := h - d_s$

Web Height - Exterior $d_{gx} := h_x - d_s$

Include Wearing Surface in Section Height $h := h + \mathbf{if}\left(\gamma_{ws} = 0.15\ \frac{kip}{ft^3}, ws, 0\right) = 32.75\ in$

Depth to Centroid of Reinforcement - Interior $d := h - y_{bar}$

Depth to Centroid of Reinforcement - Exterior $d_x := h_x - y_{barx} + h_{curb}$

Moment Applied to Interor Girders from Diaphragm

$$M_d := \begin{array}{l} \text{if } loc_d = \text{"Half"} \\ \quad \left\| P_{dint} \cdot \frac{L}{4} \right. \\ \text{else if } loc_d = \text{"Third"} \\ \quad \left\| P_{dint} \cdot \frac{L}{3} \right. \\ \text{else if } loc_d = \text{"Quarter"} \\ \quad \left\| P_{dint} \cdot \frac{L}{4} + P_{dint} \cdot \frac{L}{4} \right. \end{array} = 0\ ft \cdot kip$$

Moment Applied to Exterior Girders from Diaphragm $M_{dx} := \frac{M_d}{2} = 0\ ft \cdot kip$

Distribution Factors

Dictance Between Centroids of Deck and Web $e_g := \frac{d_g + d_s}{2} = 14.875\ in$

Area of Web $A := d_g \cdot b_w = 468\ in^2$

Moment of Inertia of Web $I := \frac{b_w \cdot d_g^{\ 3}}{12} = (2.246 \cdot 10^4)\ in^4$

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

Longitudinal Stiffness Parameter $K_g := n \cdot (I + A \cdot e_g^{\ 2}) = (1.26 \cdot 10^5)\ in^4$

Interior Moment Distribution Factor - 1 Lane $g_{m1} := 0.06 + \left(\frac{S}{14\ ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.497$

Interior Moment Distribution Factor - 2 Lane $g_{m2} := 0.075 + \left(\frac{S}{9.5\ ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot d_s^{\ 3}}\right)^{0.1} = 0.628$

Controling Interior Moment Distribution Factor $g_m := \max(g_{m1}, g_{m2})$

Roadway Width $W_r := lanewidth \cdot Nlane$

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

Eccentricity of Exterior Girder From C.G. of Girders $X_{ext} := (NG - 1) \cdot \frac{S}{2} = 11.73\ ft$

Eccentricity of Each Girder

$x_1 := X_{ext}$

$x_2 := X_{ext} - S$

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

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

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

Lever Rule Distribution Factor - One Lane $R_1 := \frac{1}{NG} + \frac{X_{ext} \cdot e_1}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.405$

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

Lever Rule Distribution Factor - Two Lanes $R_2 := \frac{2}{NG} + \frac{X_{ext} \cdot (e_1 - 5\ ft)}{x_1^{\ 2} + x_2^{\ 2} + x_3^{\ 2} + x_4^{\ 2} + x_5^{\ 2}} = 0.434$

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

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

$ee := 0.77 + \frac{d_e}{9.1\ ft} = 0.706$

$g_{mx2} := g_{m2} \cdot ee = 0.443$

$g_{mx} := \max(g_{mx1}, g_{mx2}) = 0.443$

Skew Correction Factor

$c_1 := 0.25 \cdot \left(\frac{K_g}{12 \cdot L \cdot d_s^3}\right)^{0.25} \cdot \left(\frac{S}{L}\right)^{.5} = 0.063$

$\theta := \mathbf{if}(skew \geq 30°, skew, 0°)$

$C_\theta := 1 - c_1 \cdot (\tan(\theta))^{1.5} = 0.972$

$g_m := g_m \cdot C_\theta = 0.611$

$g_{mx} := g_{mx} \cdot C_\theta = 0.431$

Interior DF — Exterior DF

$g_m = 0.611$ — $g_{mx} = 0.431$

Loading

Interior Girder Dead Load: $w_{girder} := \gamma_{RC} \cdot b_w \cdot d_g = 0.488\ \frac{kip}{ft}$

Deck Dead Load: $w_{deck} := \gamma_{RC} \cdot S \cdot d_s = 0.422\ \frac{kip}{ft}$

Curb Dead Load: $w_{curb} := 2 \cdot \gamma_{RC} \cdot h_{curb} \cdot b_{curb} = 0.45\ \frac{kip}{ft}$

Dead Load from Nonstructural Components: $w_{ns} := \frac{w_{curb}}{NG} + w_{rail} = 0.102\ \frac{kip}{ft}$

Total Structural Dead Load on Interior Girders: $DC := w_{girder} + w_{deck} + w_{ns} = 1.011\ \frac{kip}{ft}$

Exterior Girder Dead Load: $w_{girderx} := \gamma_{RC} \cdot b_{wx} \cdot d_{gx} = 0.4\ \frac{kip}{ft}$

Exterior Deck Dead Load: $w_{deckx} := \gamma_{RC} \cdot S_x \cdot d_s = 0.271\ \frac{kip}{ft}$

Total Structural Dead Load on Exterior Girders: $DC_x := w_{girderx} + w_{deckx} + w_{ns} = 0.773\ \frac{kip}{ft}$

Wearing Surface Dead Load on Interior Girders: $DW := \gamma_{ws} \cdot (ws + ws_2) \cdot S = 0.293\ \frac{kip}{ft}$

Wearing Surface Dead Load on Exterior Girders: $DW_x := \gamma_{ws} \cdot (ws + ws_2) \cdot S_x = 0.188\ \frac{kip}{ft}$

Dead Load Moments: $M_{DC} := \frac{DC \cdot L^2}{8} + M_d \quad M_{DCx} := \frac{DC_x \cdot L^2}{8} + M_{dx}$

$M_{DW} := \frac{DW \cdot L^2}{8} \quad M_{DWx} := \frac{DW_x \cdot L^2}{8}$

$M_{DC} = 146.111\ \mathbf{ft \cdot kip}$ $\quad M_{DCx} = 111.665\ \mathbf{ft \cdot kip}$

$M_{DW} = 42.375\ \mathbf{ft \cdot kip}$ $\quad M_{DWx} = 27.208\ \mathbf{ft \cdot kip}$

Live Load Moment - Truck Load $\quad M_{Truck} := 32\ \mathbf{kip} \cdot \left(\frac{L}{4}\right) + \frac{40\ \mathbf{kip}}{2} \cdot \left(\frac{L}{2} - 14\ \mathbf{ft}\right) = 332\ \mathbf{ft \cdot kip}$

Live Load Moment - Tandem $\quad M_{Tandem} := 25\ \mathbf{kip} \cdot \frac{L}{4} + \frac{25\ \mathbf{kip}}{2} \cdot \left(\frac{L}{2} - 4\ \mathbf{ft}\right) = 375\ \mathbf{ft \cdot kip}$

Live Load Moment - Lane $\quad M_{Lane} := 0.64\ \frac{\mathbf{kip}}{\mathbf{ft}} \cdot \frac{L^2}{8} = 92.48\ \mathbf{ft \cdot kip}$

Total HL-93 Live Load $\quad M_{LL} := M_{Lane} + (1 + IM) \cdot \max\left(M_{Truck}, M_{Tandem}\right)$

$M_{LL} = 591.23\ \mathbf{ft \cdot kip}$

Nominal Resistance

Depth Whitney Stress Block - Interior $\quad a := A_s \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S} = \begin{bmatrix} 1.109 \\ 1.676 \\ 2.234 \\ 1.676 \\ 1.117 \end{bmatrix} \mathbf{in}$

Nominal Moment Resistance - Interior $\quad M_n := \overrightarrow{F_y \cdot A_s \cdot \left(d - \frac{a}{2}\right)} = \begin{bmatrix} 400.966 \\ 580.503 \\ 742.202 \\ 580.503 \\ 403.783 \end{bmatrix} \mathbf{ft \cdot kip}$

Interior Nominal Moment Capacity

$M_{capacity} := \max\left(M_n\right) = 742.202\ \mathbf{ft \cdot kip}$

Depth Whitney Stress Block - Exterior $\quad a_x := A_{sx} \cdot \frac{F_y}{0.85 \cdot f'_c \cdot S_x} = \begin{bmatrix} 1.306 \\ 2.175 \\ 2.608 \\ 2.175 \\ 1.306 \end{bmatrix} \mathbf{in}$

Nominal Moment Resistance - Exterior $\quad M_{nx} := \overrightarrow{F_y \cdot A_{sx} \cdot \left(d_x - \frac{a_x}{2}\right)} = \begin{bmatrix} 396.15 \\ 622.742 \\ 733.311 \\ 622.742 \\ 396.15 \end{bmatrix} (\mathbf{ft \cdot kip})$

Exterior Nominal Moment Capacity

$M_{capacityx} := \max\left(M_{nx}\right) = 733.311\ \mathbf{ft \cdot kip}$

Rating Factors

Interior Moment Rating Factor

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

Exterior Moment Rating Factor

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

Interior

$RF_{Interior} = 0.887$

Exterior

$RF_{Exterior} = 1.414$

Rating Factor Improvements

Concrete Compressive Strength - Larger is More Conservative	$f'_c := 5\ ksi$
Concrte Elastic Modulus	$E_c := 1820\ ksi \cdot \sqrt{\frac{f'_c}{ksi}} = (4.07 \cdot 10^3)\ ksi$

Interior Girders

Maximum Recorded Strain	$\varepsilon_T := 89.35 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 478.1\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 6065\ in^3$
Cracked Section Modulus	$S_{cr} := 1991\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \textbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right)$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_m}{S_e \cdot E_c} = 1.419 \cdot 10^{-4}$
Test Benefit Factor	$k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 0.589$
Ratio of Applied to HL-93 Moment	$r_M := \frac{M_{Max}}{M_{LL}} = 0.809$
Test Understanding Factor	$k_b := \textbf{if}\left(r_M > 0.7, 0.5, 0\right) = 0.5$
Rating Improvement Factor	$k := 1 + k_a \cdot k_b = 1.294$
Improved Rating Factor	$RF_{Improved} := RF_{Interior} \cdot k = 1.148$

Exterior Girders

Maximum Recorded Strain	$\varepsilon_T := 47.36 \cdot 10^{-6}$
Maximum Applied Moment per Lane	$M_{Max} := 478.1\ ft \cdot kip$
Uncracked Section Modulus	$S_{unc} := 6086\ in^3$
Cracked Section Modulus	$S_{cr} := 1457\ in^3$
Section Behavior	$Behavior :=$ "Uncracked"
Section Modulus Effective for Behavior	$S_e := \textbf{if}\left(Behavior = \text{"Uncracked"}, S_{unc}, S_{cr}\right)$
Calculated Strain	$\varepsilon_c := \frac{M_{Max} \cdot g_{mx}}{S_e \cdot E_c} = 9.986 \cdot 10^{-5}$

Test Benefit Factor $k_a := \frac{\varepsilon_c}{\varepsilon_T} - 1 = 1.108$

Ratio of Applied to HL-93 Moment $r_M := \frac{M_{Max}}{M_{LL}} = 0.809$

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

Rating Improvement Factor $k := 1 + k_a \cdot k_b = 1.554$

Improved Rating Factor $RF_{ImprovedExt} := RF_{Exterior} \cdot k = 2.197$

Improved Rating Factors

Interior	Exterior
$RF_{Improved} = 1.148$	$RF_{ImprovedExt} = 2.197$

Figure 105: Bridge 3848 calculations