



Transportation Research Division



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Live Load Testing and Load Rating of Five Reinforced Concrete Bridges

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<p>Five cast-in-place concrete T-beam bridges – Eustis #5341, Whitefield #3831, Cambridge #3291, Eddington #5107, and Albion #2832 – were live load tested. Revised load ratings were computed either using test data or detailed analysis when possible. In four of the five bridges, increases in rating factors were made through analysis alone. The Cambridge bridge was shown to have HL-93 operating rating factors greater than 1.0 for both shear and moment without testing. Additionally, gains are presented for two of the five bridges (Eustis and Whitefield) where test data was used to compute an adjustment factor K for midspan bending, where the revised rating factor equals $K \times RF$. Adjustment factors were found to be 1.61 and 1.24 for Eustis and Whitefield, increasing their flexural rating factors to 1.14 and 1.06 if accepted by the Maine DOT. Similarly, our analyses indicate that Eustis has a rating capacity greater than 1.0 for shear. While Whitefield has a rating factor greater than 1.0 for shear at the supports, the rating factor for shear near the termination of the bent-up reinforcing is 0.48 based on our analyses.</p> <p>Low loads used in the live load tests of both Albion and Eddington, where only one truck was provided on site, preclude the development of revised rating factors based on the test data. However, both of these structures did exhibit better-than-expected live load distribution. Further, analysis of the Albion bridge indicates that the consultant-provided load ratings for this structure can likely be increased significantly.</p> <p>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.</p> <p>The objective of this study, to implement an effective live load testing bridge program, was fulfilled.</p>			
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LIVE LOAD TESTING AND LOAD RATING OF FIVE REINFORCED CONCRETE BRIDGES

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EXECUTIVE SUMMARY

Five cast-in-place concrete T-beam bridges – Eustis #5341, Whitefield #3831, Cambridge #3291, Eddington #5107, and Albion #2832 – were live load tested. Revised load ratings were computed either using test data or detailed analysis when possible. In four of the five bridges, increases in rating factors were made through analysis alone. The Cambridge bridge was shown to have HL-93 operating rating factors greater than 1.0 for both shear and moment without testing. Additionally, gains are presented for two of the five bridges (Eustis and Whitefield) where test data was used to compute an adjustment factor K for midspan bending, where the revised rating factor equals $K \times RF$. Adjustment factors were found to be 1.61 and 1.24 for Eustis and Whitefield, increasing their flexural rating factors to 1.14 and 1.06 if accepted by the Maine DOT. Similarly, our analyses indicate that Eustis has a rating capacity greater than 1.0 for shear. While Whitefield has a rating factor greater than 1.0 for shear at the supports, the rating factor for shear near the termination of the bent-up reinforcing is 0.48 based on our analyses.

Low loads used in the live load tests of both Albion and Eddington, where only one truck was provided on site, preclude the development of revised rating factors based on the test data. However, both of these structures did exhibit better-than-expected live load distribution. Further, analysis of the Albion bridge indicates that the consultant-provided load ratings for this structure can likely be increased significantly.

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.

REVIEW OF EXISTING LOAD RATING OF WHITEFIELD, EUSTIS & CAMBRIDGE BRIDGES

Independent load rating analyses of an interior girder were conducted for Whitefield #3831, Cambridge #3291 and Eustis #5341. These analyses were outside the scope of UMaine's planned work, but were necessary in light of the measured response of the structure observed during live load testing as described later in this report and conservatism in the load rating reports provided to UMaine. Major features of the rating analysis are summarized as follows:

- 1) The plans for Whitefield show a bridge length of 92' from abutment CL bearing to abutment CL bearing, and an overall length of 46'-7" for each span. Our field visit indicated that the beams are in full contact with the cap at the intermediate pier. Assuming a 6" bearing length at the intermediate pier, using the $\frac{1}{2}$ pier width of 27", and taking into account the 1" chamfer at the pier gives a span length of $46' - 27" + 1" + 6"/2 = 44.08'$.
- 2) The plans for Cambridge show a clear span of 25' from abutment face to abutment face, and our field visit indicates full bearing between the girders at each abutment. This justifies a span length of 25'-6".
- 3) The earlier ratings of the Whitefield, Cambridge and Eustis bridges did not account for existing bent-up longitudinal reinforcing when calculating the shear capacity near the

supports. The bent-up longitudinal reinforcing was hooked for both Whitefield and Cambridge, although it was not hooked at Eustis.

- 4) Following from point (3), the critical location for shear may be located just after the pair of bent-up longitudinal reinforcing bars located furthest from the support, since at that point the stirrup spacing is larger than near the support while live and dead load shear remain significant.
- 5) In the case of the Cambridge structure, existing rating calculations assumed 1 in diameter round bars for the flexural reinforcing, whereas the plans show 1 in square bars. The 1 in square bars have significantly more cross-sectional area.
- 6) The live load test results from the summer of 2013 indicate that the concrete wearing surface of the Whitefield structure contributes to the stiffness of the bridge under live loading and effectively increasing the deck thickness. This additional deck thickness cannot be used in strength calculations since it is not effective for self-weight. However, it is justifiable to include the wearing surface as additional deck thickness when computing the live load distribution factors, which are stiffness- and not strength-driven.

Copies of the UMaine rating calculations for the three bridges are provided in Appendix 1. The results of the analyses are summarized and compared with the existing ratings in Table 1.

Table 1: HL-93 Operating Rating Factors for Interior Girders Based on Analysis

Structure	Mid-span Moment		Shear at support		Shear at bent-up bar termination	
	UMaine RF	Existing RF	UMaine RF	Existing RF	UMaine RF	Existing RF
Whitefield	0.85	0.58	1.00	0.36	0.48	NA
Cambridge	1.19	0.62	1.38	0.77	1.59	NA
Eustis	NA	0.71	1.05	0.42	0.96	NA

The results in Table 1 indicate that the interior beams of both structures have significantly more capacity than the original ratings predict. Indeed, the HL-93 operating RFs for the Cambridge bridge interior girder are all greater than one. However, the Whitefield bridge still has a very low (0.48) rating factor for shear at the termination of the last pair of bent-up longitudinal bars. This rating factor is lower than for Eustis and Cambridge because of the use of small (#3) stirrups at a large spacing at Whitefield relative to the other two bridges.

Based on our independent load rating for an interior beam of these structures, it is recommended that a MaineDOT bridge engineer review these ratings and supporting calculations to assess their accuracy. In particular, the shear rating factors calculated by UMaine may be unconservative for Eustis since the bent-up bars are not hooked.

ANALYSIS OF LIVE LOAD TEST DATA

A quantitative analysis of the live load data has been performed for Eustis, Whitefield, Eddington and Albion as described here. The live load test data for Cambridge is not critical since results presented in the last section show that it is structurally sufficient.

EUSTIS #5341

The Eustis bridge was tested in May, 2014 with four loaded dump trucks to ensure at least one T-beam structure was tested under a heavy load. Calculations show that the moment produced by the four-truck loading was approximately 87.5% of that produced by an HL-93 loading with impact (truck + lane).

The measured strains have several key features. First, the strain response varied nearly linearly with increasing load up to the full four-truck load. Second, the measured strains are typical of an uncracked section, even under a four-truck loading. Third, the distribution of mid span strains among the five girders indicate more equal load sharing among the five girders than predicted by the AASHTO distribution factor.

The maximum measured flexural tensile strain (ϵ_t) at an interior girder was 87.8 $\mu\epsilon$, which is significantly less than the maximum calculated strain (ϵ_c) of 195 $\mu\epsilon$ for an uncracked section. The calculated strain conservatively assumes a concrete strength of 5000 psi which is twice the nominal value of 2500 psi. The use of a larger-than-specified value for concrete strength is conservative because it leads to a larger calculated elastic modulus and therefore a lower calculated strain.

The AASHTO Manual for Bridge Evaluation (MBE) permits a calculated rating factor to be increased if certain conditions are met. Given that the applied load effect exceeded 0.7 times the maximum rating load effect, but not assuming that the results can be safely extrapolated to 1.33 times the HL-93 rating vehicle because of the likelihood of concrete flexural cracking at higher loads, the moment rating factor can be scaled by K as computed below.

$$K = 1 + K_b \times \left(\frac{\epsilon_c}{\epsilon_t} - 1 \right) = 1 + 0.5 \times \left(\frac{195}{87.8} - 1 \right) = 1.61$$

This implies that the rating factor for flexure at midspan can be increased from 0.71 to 1.14 for Eustis.

WHITEFIELD #3831

The Whitefield bridge was tested during the summer of 2013 with two loaded dump trucks. As with the Eustis bridge, the measured strains are typical of an uncracked section. Calculations show some conservatism in the AASHTO load factors, but less than was inferred for Eustis. This is likely because the Eustis exterior girders were affected by a large area of integral concrete sidewalk, which gave them a large stiffness relative to the interior girders. The maximum measured strain at Whitefield was 51.9 $\mu\epsilon$, which is only slightly less than the peak strain of 53.1 $\mu\epsilon$ measured at Eustis under a two-truck loading. The computed strain for this condition was 77.1 $\mu\epsilon$, conservatively assuming 5000 psi concrete as done with Eustis.

Calculations indicate that the moment produced by the two-truck loading at Whitefield was approximately 55% of that produced by an HL-93 loading. If results cannot be reliably extrapolated to 1.33 times the HL-93 loading, AASHTO does not permit the rating factor to be

increased based on strains measured at this load level. However, given the similarity in response and construction details between Whitefield to Eustis, it is reasonable to assume that if four trucks had been used to load Whitefield, strains would have increased linearly when compared to the two-truck loading. Given this assumption, the rating factor modifier K can be computed as shown below.

$$K = 1 + K_b \times \left(\frac{\varepsilon_c}{\varepsilon_t} - 1 \right) = 1 + 0.5 \times \left(\frac{77.1}{51.9} - 1 \right) = 1.24$$

This implies that the rating factor of 0.85 for flexure at midspan can be increased to 1.06 for Whitefield.

EDDINGTON #5107

The Eddington bridge was the first test of the T-beams during the summer of 2013. This bridge had two widenings and was under construction to repair the railing on each side of the bridge at the time of testing. While two loaded dump trucks were requested for load testing, only one was provided on the day of testing, and the structure was tested with only one loaded dump truck. The test vehicle moment ratio to the HL-93 loading with impact is 0.56. This loading is not high enough to allow the calculation of a rating adjustment factor as done for Eustis and Whitefield, and K_b for this case would be zero assuming behavior could not be extrapolated to 1.33 times the design loading for the bridge. However, the data do allow general behavior of the structure to be inferred as detailed below.



Figure 1: Loading of Eddington bridge showing extent of construction

Peak strains of $14 \mu\epsilon$ were seen in the centerline T-beam at midspan when the truck was centered in the road. This is roughly half the strain observed during single truck loading of the Eustis bridge ($30 \mu\epsilon$). Eddington has fill over the concrete structure of roughly 3'-6" in depth and T-beam spacings of 5'-2" versus 7-10" at the Eustis bridge.

Load distribution was investigated with peak strain data for the load case where the truck was on the outside of the lane. In this case, peak strains on the first two interior girders were nearly identical and the exterior girder was lower. The next two interior girders were not instrumented

and a clear picture of load distribution cannot be inferred. However, if it is assumed all the load from the single truck was carried by these three girders (exterior and first two interior) and distributed by their relative stiffnesses as assumed in the analytical solution, we can develop a general comparison for the conservatism in the analytical load rating based on load distribution.

Using uncracked, transformed section moduli of the interior and exterior girders of the widened section assuming a 5000psi concrete strength and the measured strains, moments can be inferred from the test data. It should be noted that calculated section moduli for the interior girders as part of this analysis were approximately 22% higher than those reported by Erdman Anthony (10/26/2010) where the transformed tension reinforcement was not taken into account ($n = 1$). Using the larger value and the following equation, the moment in the girder can be computed.

$$M_{test} = E_c \varepsilon S_x$$

All three girders on the Southerly side shown in Figure 2 showed between 31% and 48% of the moment calculated using the total truck load applied to the three girders as computed analytically (see Appendix D). Table 2 gives a summary of strain.

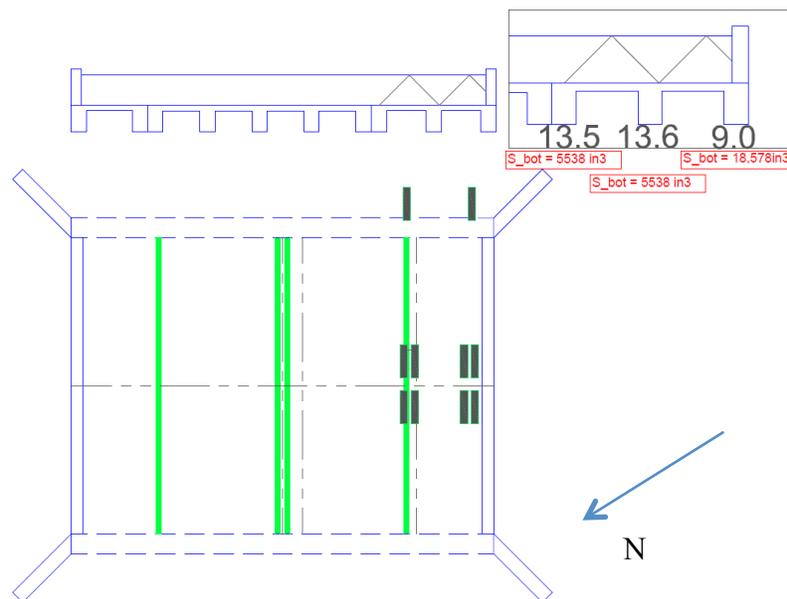


Figure 2 – Plan of bridge with maximum strain values

Table 2: Summary of strain data collected

Truck Position	Cross Section	Max Strain ($\mu\epsilon$)	Min Strain ($\mu\epsilon$)
Y1	5	13.2	-4.9
Y2	4	13.8	-4.5
Y3	3	7.7	-2.7
Y4	3	12.0	-3.9
Y5	3	11.0	-3.6
Y6=Y1	5	13.6	-5.5

Eddington Summary

An explicit rating factor adjustment could not be calculated due to the low loading of the bridge during the test, but test data showed greater load distribution than accounted for in analytical solutions. Using test data and uncracked, transformed section moduli for the girders, and assuming the loading scenario shown in Figure 2, the girders each appear to receive between 30% and 48% of the moment produced by a single truck.

ALBION #2832

The Albion bridge was tested in August 2013 with a single truck. Two trucks were required here to give adequate results for revising load ratings with test data, but only one truck arrived on the site. This bridge was widened in 1949 with a pie shaped, three girder section on the upstream side. Drawings were provided for this portion of the bridge only. Road alignment was now skewed to all girders except the exterior upstream girder.

A review of rating factor calculations was also performed for this bridge and is presented below for flexure. The points that we feel are more accurately represented in our calculations are listed below.

1. The area of reinforcing used in the calculations provided uses round bars where square bars as referenced in 1949 drawings. This increases the area of flexural reinforcement in the calculations by approximately 26.6%.
2. Distribution factor for pie shaped section uses the maximum spacing of the girders. Using the girder spacing at midspan where the loading maximizes moment may better represent the actual cross section resisting load. This reduces the distribution factor from 0.624 to 0.526.
3. The centroid of reinforcing bars used in calculations is approximately 1.5 inches higher than shown in drawings reducing the depth of the bending section.

With these changes, a new operating flexural rating factor with loads calculated from EA in 2010 is calculated to be 0.77 where the EA value was 0.31 for the centerline interior girder where the fill depth is approximately 12 inches. Shear calculations were not reviewed, but it can be

expected that an increased bar area with the inclusion of bent bars would increase the shear rating factor.

A summary of peak strain data for each transverse truck position is given in Table 3.

Table 3 – Summary of Strain Data from Albion Bridge

Truck Position	Cross Section	Max Strain (microstrain)	Min Strain (microstrain)
Y1	3	19.1	-4.0
Y2	4	19.8	-4.2
Y3	5	17.8	-5.3
Y4	5	17.9	-4.8
Y5	1	16.4	-5.8

The peak strain at midspan of the first interior girder that was analyzed by EA (2010) and reviewed above is shown in position Y1 as 19.1 $\mu\epsilon$. Assuming an uncracked section and f'_c of 5000 psi this gives an inferred moment of 48.2 kip-ft ($S_x = 7,445 \text{ in}^3$). This moment is 19% of moment caused by the test truck with distribution factor for this cross section ($M_{\text{tandem}} \times DF$ with no impact).

Two factors were identified that would contribute to the reduced bending moment in the girder. First, the fill over the deck helps distribute the load. While AASHTO specifies that fill depths less than 2 feet be neglected, the fill still contributes to load distribution. Second, with the exception of the exterior girder, the test vehicle traveled at a skew of about 10 degrees relative to the girders.

An explicit adjustment to the rating factor is not included due to the low loading. Loading levels during this test are very similar to the single truck load cases at the Eddington and Eustis bridges. Recorded strain levels are very similar as well. Extrapolating results based on test data for other structures may be done at the discretion of the MaineDOT, but is not presented here. Additional details of the test and data is presented in Appendix E.

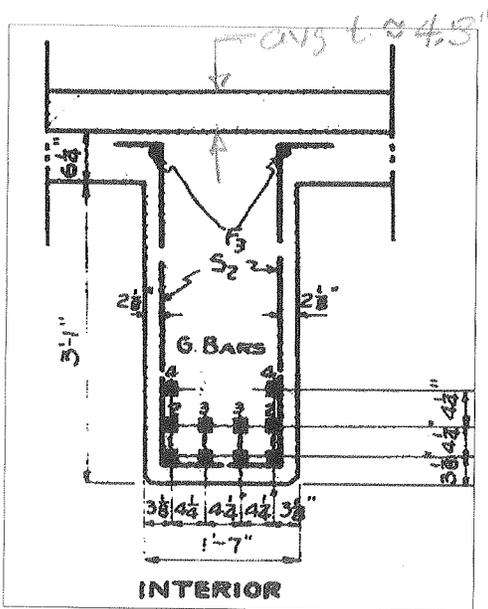
REVIEW OF EA RATING - WHITEFIELD

W. Davids
10/16/13

- EA Rating used span length of 46'-7"
Plans show 92' & BEING to & BEING, beams resting on intermediate pier that is 34" wide w/ 1" chamfer.
- Spans are therefore $92\frac{1}{2} - 34\frac{1}{2} + 1" + \text{say } 3" = 44.08'$
assume 6" beam
- EA Plans use simplified method to compute shear strength - Capacity can be increased by use of General procedure specified in AASHTO 5.8.3.4.2
- EA does not account for bent long. reinforcing when computing V_s , (although simplified method for computing V_c must be used in this case)

REVISED HL-93 OPERATING MOMENT, INTERIOR

- Use span length of 44.08'
- field test data indicate V_s contribution, but DL shears & moments will be unaffected. Further, stirrups are anchored in structural deck, limiting the effective shear depth d_v . However, for LL, dest, it is acceptable to include V_s in t_s , since deck stiffness is driver.



Deck $\approx 6\frac{1}{4}$ " Thick
 $A_s = 15.63 \text{ in}^2$, $f_y = 33 \text{ ksi}$
 bars @ 6.53 " up from base
 $d_e = \text{spacing} = 99.5$ "

$$0.85 f_c' a b = A_s f_y \Rightarrow$$

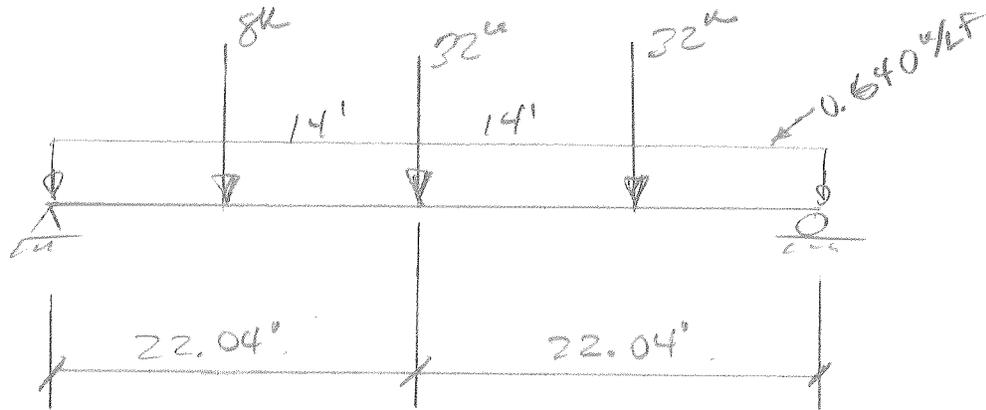
$$a = \frac{15.63 \times 33}{0.85 \times 2.5 \times 99.5} = 2.44 \text{ in} < 10.5$$

$$d = 37" + 6\frac{1}{4}" - 6.53" = 36.72"$$

$$M_n = 15.63 \times 33 \left(36.72 - \frac{2.44}{2} \right) \times \frac{1}{12}$$

REVISED HL-93 OPERATING MOMENT, INTERIOR W. J. Davids
10/16/13

LIVE LOADS



$$M_{truck} = \frac{32 \times 44.08}{4} + \left(\frac{8 \times 8.04 \times 36.04}{44.08} \right) \frac{22.04}{36.04}$$

$$+ \left(\frac{32 \times 8.04 \times 36.04}{44.08} \right) \frac{22.04}{36.04}$$

$$= 353 + 160 = 514 \quad \left(\text{vs. } \frac{25 \times 44.08}{4} + \frac{25 \times 18.04 \times 26.04}{44.08} \right) \left(\frac{22.04}{26.04} \right)$$

$$= 276 + 226 = 502 \quad \text{(for tandem)}$$

$$M_{lane} = \frac{0.640 \times 44.08^2}{8} = 155 \text{ ft-k}$$

$$M_{HL93} = 669 \text{ ft-k} \quad \leftarrow \text{per lane, no IM}$$

$$M_{93+IM} = 1.33 \times 514 + 155 = 839 \text{ ft-k (w/Impact)}$$

DEAD LOADS

Per EA report:

- Deck $\text{e} = 0.651 \text{ k/ft}$
- Beam $\text{e} = 0.732 \text{ k/ft}$
- Railing $\text{e} = 0.075 \text{ k/ft}$

$$\text{DIST. DC} = 1.458 \text{ k/ft}$$

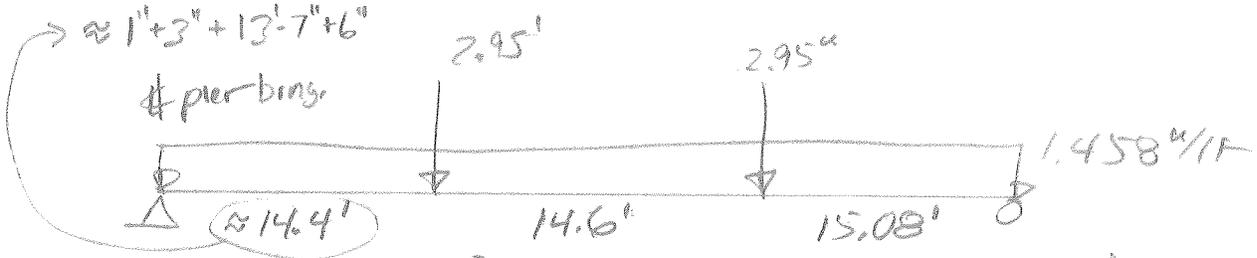
$$\text{WS} \text{ e } \left(\frac{99.5 \times 4.3}{144} \right) \times 0.150 = 0.446 \text{ k/ft}$$

$$2 \text{ Diaphragms e } \frac{81 \times 12 \times 35}{123} \times \frac{150 \text{ pcf}}{1000 \#/\text{ft}^3} = 2.95 \text{ k/ft ea.}$$

REVISED HL-93 OPERATING MOMENT, INTERIOR

W. Daniels
10/16/13

DEAD LOADS



$$M_{DC} = \frac{1.458 \times 44.08^2}{8} + \frac{2.95 \times 14.4 \times 29.7 \times \left(\frac{22.04}{29.7}\right)}{44.08}$$

$$+ \frac{2.95 \times 15.08 \times 29}{44.08} \times \left(\frac{22.04}{29}\right)$$

$$= 354 + 21 + 22.2 = 397 \text{ ft}\cdot\text{w}$$

$$M_{DW} = \frac{0.446 \times 44.08^2}{8} = 108 \text{ ft}\cdot\text{w}$$

LL DF: ~~we~~ here we can use $6\frac{1}{4}'' + 4.3'' = 10.55''$ deck, since full deck affected LL dist.

per A1, 2 or more lanes centered

$$m_g = 0.06 + \left(\frac{s}{9.5}\right)^{0.6} + \left(\frac{s}{L}\right)^{0.2} \left(\frac{K_g}{12 L t_s^3}\right)^{0.1}$$

$$K_g = n (I + A e_g^2)$$

$$n = 1.0$$

$$I = \frac{1}{12} \times 19'' \times 37^3 = 80201 \text{ in}^4$$

$$e_g = \frac{1}{2}(37'') + \frac{1}{2}(10.55'') = 23.78$$

$$s = 99.5''$$

$$L = 44.08'$$

$$K_g = 1(80201 + 19'' \times 37'' \times 23.78^2) = 477572$$

$$\left(\frac{K_g}{12 L t_s^3}\right)^{0.1} = \left(\frac{477572}{12 \times 44.08 \times 10.55^3}\right)^{0.1} = 0.974$$

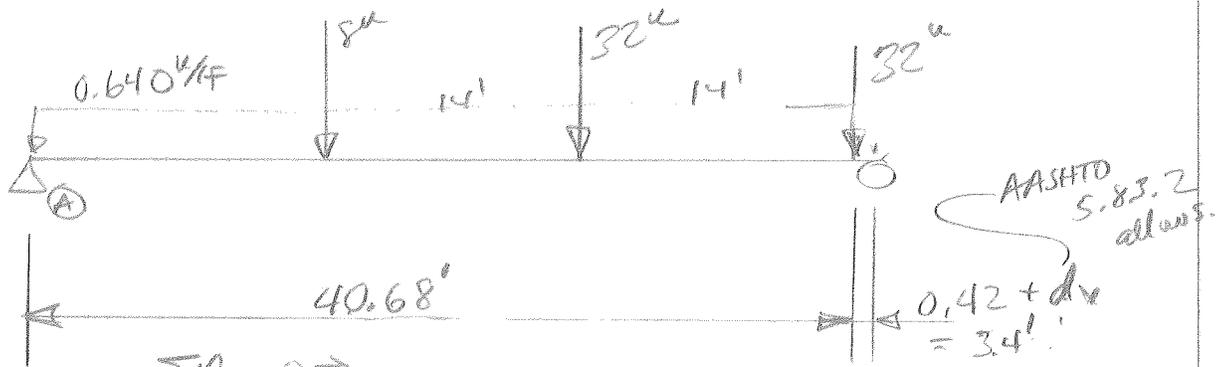
$$m_g = 0.06 + \left(\frac{8.292}{9.5}\right)^{0.6} \left(\frac{8.292}{44.08}\right)^{0.2} 0.974 = 0.703$$

W. David
10/16/13

REVISED HL-93 OPERATING SHEAR, INTERIOR

LIVE LOAD

- Critical location for shear will be face of spri. @ ABUTMENT where bearing length is smallest + dv
- Expansion plate details 10'8" bearing lb, say 5" from \pm spri.



$$\sum M_A = 0 \Rightarrow \frac{0.64 \times 44.08^2}{2} + 8 \times 12.68 + 32 \times 26.68 + 32 \times 40.68 - R_B \times 44.08 = 0$$

$$\Rightarrow R_B = \frac{622 + 2257}{44.08} = 65.3^k \leftarrow \text{per lane, No IM}$$

$$R_B \text{ w/im} = 1.33 \times 51.2 + 14.1 = 82.2^k$$

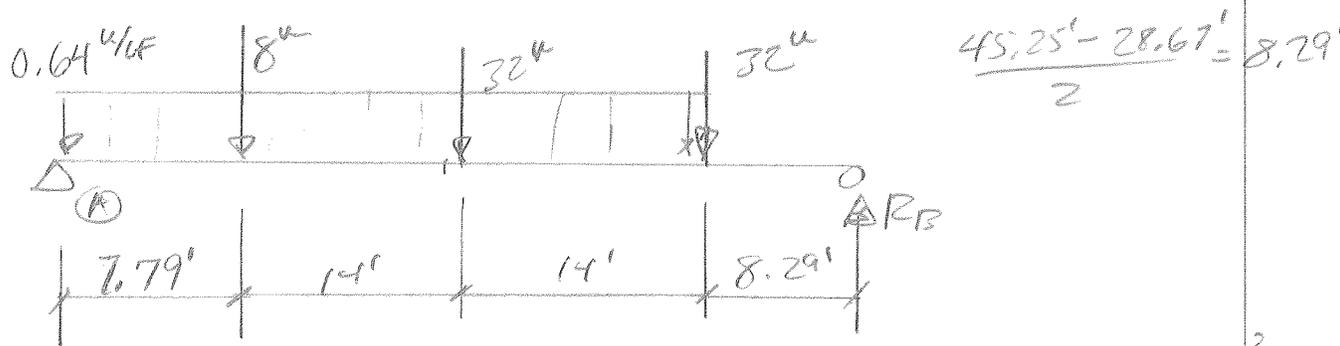
$$* \text{ tandem rule: } R_B = \frac{25 \times (40.68 + 26.68)}{44.08} = 459^k \text{ vs. } 51.2^k \text{ truck}$$

05 588 000 QUANTITY CASE SOLUTION
 02 706 000 SUPPLY RELEASE SOLUTION
 03 000 000 SUPPLY RELEASE SOLUTION
 04 000 000 SUPPLY RELEASE SOLUTION
 05 000 000 SUPPLY RELEASE SOLUTION
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 100 000 000 SUPPLY RELEASE SOLUTION

REVISED HL-93 OPERATING SHEAR, INTERIOR

W. DAVIES
10/16/13

CHECK SHEAR AFTER LAST BENT-UP BAR



$$\sum M_A = 0: \quad 8 \times 7.79 + 32 \times 21.79 + 32 \times 35.79 + 0.64 \times \frac{35.79^2}{2} - R_B \times 44.08 = 0$$

$$\Rightarrow R_B = \frac{1905 + 410}{44.08} = 52.5 \text{ k NO in}$$

$$R_B = 1905 / 44.08 \times 1.33 + \frac{410}{44.08} = 66.8 \text{ w/in}$$

CAPACITY: $V_s = \frac{0.22 \times 33 \times 35.5}{18''} = 143 \text{ k}$

USE 5.8.24.2 - we don't have min stirrup.

min shear reinf. = $0.0316 \sqrt{25} \frac{19 \times 18''}{23} = 0.52 < 0.22 \text{ NG}$

$$\beta = \left(\frac{4.8}{1 + 750E_s} \right) \left(\frac{51}{29 + S_{xe}} \right); \quad S_{xe} = S_x \frac{1.38}{g_y + 0.63}$$

$S_x = \min \left\{ \begin{array}{l} 35.5'' \text{ top lens bar} \\ \approx 37'' - (3 \times \frac{1}{8} + 4 \times \frac{1}{4} + 4 \times \frac{1}{4}) + \text{say } 3'' = 28.4'' \text{ stirrup hanger} \end{array} \right.$

min Area = $0.002 \times 19 \times 28.4 = 1.62 \text{ in}^2 \text{ NG}$

$S_x = 35.5''$

$$S_{xe} = 35.5 \left(\frac{1.38}{31.4 + 0.63} \right) = 35.5$$

$$\epsilon_s = \frac{M_u/d_v + V_u}{E_s A_s}$$

Spandag 9: $V_u = 1.35 \times 66.8 + 1.25(24.5 + 6.13) = 113.6 \text{ k}$

REVISED HL-93 OPERATING SHEAR, INTERIOR

W Dennis
10/16/13

CHECK SHEAR AFTER LAST BENT-UP BAR

$$M_{u+m} = 66.8^{\text{in}} \times 8.29' = 554^{\text{in}}$$

$$M_{DW} \neq M_{DC} \sim$$

for general w , $m = \frac{wl}{2} \times 8.29 - \frac{w \times 8.29^2}{2}$

$$\Rightarrow M_{DW} = \frac{0.446}{2} (44.08 \times 8.29 - 8.29^2) = 66.2$$

$$M_{DC} \approx \frac{1.458}{2} (296.7) + 1.5 \times 3^{\text{in}} \times 8.29'$$

↳ diaphragms

$$= 216.3 + 37.3 = 253.6^{\text{in}}$$

$$M_u = 1.35 \times 554 \times 0.703 + 1.25 (66.2 + 253.6) =$$

$$526 + 400 = 926 \text{ ft}\cdot\text{lb}$$

$$\epsilon_s = \left(\frac{926 \times 12 + 114}{35.5} \right) \frac{1}{29000 \times 15.63} = 9.42 \times 10^{-4}$$

$$\beta = \left(\frac{4.8}{1 + 750 \times 9.42 \times 10^{-4}} \right) \left(\frac{51}{39 + 35.5} \right) = 1.92$$

$$\theta = 29 + 2500 \times 9.42 \times 10^{-4} = 32.3^\circ$$

so... $V_c = 0.0316 \times 1.92 \sqrt{2.5} \times 19 \times 35.5 = 64.7^{\text{in}}$

$$V_s = \frac{0.22 \times 33 \times 35.5 \times (10 \pm 32.3^\circ)}{18''} = 22.6^{\text{in}}$$

$$\phi V_n = 0.95 \times 0.90 \times (64.7 + 22.6) = 74.6^{\text{in}}$$

REVIEW OF EA RATINGS - CAMBRIDGE

W. Daniels
10/23/13

- EA Ratings used out-to-out span of 29.17', but actual span is 25' clear + brg length, say 25'-6"
- EA shear calculations neglected bent-up bars & spt
- EA moment capacity calc use round bars, not square as on plans

REVISED HL-93 OPERATING MOMENT, INTERIOR

$$M_u = PL/4 + \frac{P \times 8.75 \times 16.75}{25.5} \left(\frac{12.75}{16.75} \right) + \frac{0.640 \times 25.5^2}{8} \quad (P=25^k)$$

= 159.4 + 109.4 + 52.0 = 320.8 ft-k @ 0 m, 410 w/m
 RISA 3D indicates 322 ft-k - OK tandem controls.

$$M_{DC} = \frac{0.935 \times 25.5^2}{8} + \frac{1.317 \times 25.5}{4} = 76.0 + 8.4 = 84.4 \text{ ft-k}$$

deck + brg + rail (see EA calc) diaphragm (see EA calc)

$$M_{DW} = \frac{0.251 \times 25.5^2}{8} = 28.5 \text{ ft-k}$$

$$g_m = 0.564 \quad (2 \text{ lanes} - \text{see EA})$$

$M_n = 480.6 \text{ ft-k per EA}$, but used $A_s = 9.18 \text{ in}^2$ (12-#8 bars)

plans call out 12 - 1" square bars; $A_s = 12 \text{ in}^2$; $b = 5'-9"$

$$0.85 f'_c a b = A_s f_y \Rightarrow a = \frac{12 \times 33}{0.85 \times 2.5 \times 5.75 \times 12} = 2.070"$$

$$M_n = A_s f_y (d - a/2) = 33 \times 12 (19.5 - 2.07/2) = 7187 \text{ in-k} = 599 \text{ ft-k}$$

$$R = \frac{0.9 \times 0.95 \times 599 - 1.25 (84.4 + 28.5)}{1.35 (0.564 \times 410)} = 1.19 \text{ vs. } 0.62 \text{ M EA report.}$$

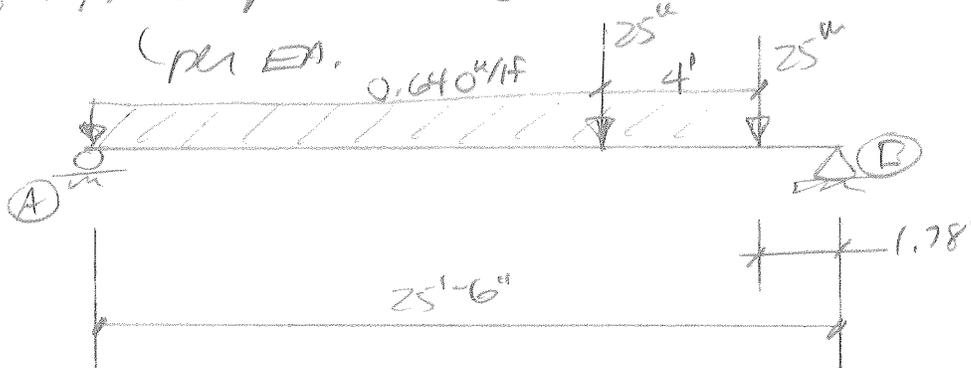
OK for HL-93 operating, moment

REVIEW OF EA RATINGS - CAMBRIDGE

W. Davids
10/23/13

REVISED HZ-93 OPERATING SHEAR, INTERIOR, @ SPRT

Check shear @ dv from face of split, or
3" + 18.36" from bearing



$$V_{LL} = R_B; \left(\sum M_A^+ = \frac{0.64 \times 23.72^2}{2} + 25 \times (19.72 + 23.72) - R_B \times 25.5 = 0 \Rightarrow R_B = \frac{180 + 1086}{25.5} \right)$$

w/m: $V_{LL} = \frac{180}{25.5} + 1.33 \times \frac{1086}{25.5} = 63.7^k = 49.64^k \leftarrow \text{controls (transverse)}$

w/ truck:

$$V_{LL} = \frac{180 + 32(23.72 + 9.72)}{25.5} = 49.02^k \text{ (truck + lane)}$$

$g_u = 0.737$
(see EA report)

DC: girder cut = 0.935 k/ft (incl. rails)
diaphragm: 1.317 k (e center)

DW: 0.351 k/ft

$$V_{DC} = \frac{0.935 \times 23.72^2 / 2 + 1.317 / 2}{25.5} = 11.0^k$$

$$V_{DW} = \frac{0.351 \times 23.72^2 / 2}{25.5} = 3.87^k$$

Capacity: $V_c = 0.0316 \times 2 \times \sqrt{2.5} \times 18'' \times 18.36'' = 33^k$

$$V_s = \frac{A_v f_y d_v}{s} + A_v f_y \sin^2(\alpha) \quad \alpha = \cos^{-1} \left(\frac{15.25}{18.5} \right) = 34.5^\circ$$

$$V_s = \frac{2 \times 0.70 \times 33 \times 18.36''}{4.5} + 2 \times 10''^2 \times 33 \times \sin^2 34.5^\circ$$

(bars G2; see drawings.)

$$= 53.9^k + 37.4^k = 91.3^k$$

REVIEW OF EA CALCS - CAMBRIDGE

W. Davids
10/23/13

CHECK SHEAR @ LAST BENT-UP BAR

Need M_u corresp to V_u
 $M_{u+M} = (41.0 \times 1.33 \times 2.59' + 6.59 \times 2.59') = 158.3^k$ (imp on load)
 $M_{u1} = 0.564 \times 158.3 = 89.3^k$ / girder w/im

$M_{DC} = \frac{0.935}{2} (25.5' \times 2.59' - 2.59'^2) + \frac{1.317}{2} \times 2.59' = 27.7 + 1.71$
 $= 29.4^k$

$M_{DW} = \frac{0.351}{2} (59.3) = 10.4^k$

$M_u = 1.25 (29.4 + 10.4) + 1.35 \times 89.3 = 170.3^k$

$\epsilon_s = \left(\frac{170.3 \times 12 + 78}{18.36} \right) / (29000 \times 12 \text{ in}^2) = 5.44 \times 10^{-4}$

$\beta = \left(\frac{4.8}{1.750 \times 5.44 \times 10^{-4}} \right) = 3.41$

$\theta = 29 + 3500 \epsilon_s = 30.9^\circ$

So... $V_c = 0.0316 \times 3.41 \sqrt{7.5} \times 18 \times 18.36 = 56.3$

$V_s = \frac{0.4 \times 32 \times 18.36 \cot \theta}{5.25} = 77.1^k$
 $\Sigma = V_u = 133^k$

$R = \frac{0.9 \times 0.95 \times 133 - 1.25 \times (10.8 + 3.62)}{1.35 \times 61.1 \times 0.737} = \frac{114.1 - 17.4}{60.8}$
 $= \underline{1.59} > 1 \text{ OK}$

REVIEW OF EA RATINGS - EUSTIS

- EA Codes for shear do not account for bent-up bars in shear capacity
- Must also check shear @ termination of last bent-up bar

SHEAR SUPPORT:

per EA, $V_{ult} = 73.5^k$ (includes DF = 0.812)
 $V_{DC} = 37.09^k$
 $V_{DW} = 9.68^k$

$V_c = 66.71^k$ ($0.9d_c = 31.05''$) $\phi \psi_c = 0.9 \times 0.95$
 $V_s = 50.30^k$ (vertical bars only)

bent-up bars: 2 - #11 bent @ 45 degree, 33ksi steel

$A_s^{bent} = 2 \times 1.56 \text{ in}^2 = 3.12 \text{ in}^2$

$V_s^{bent} = 3.12 \text{ in}^2 \times 33 \text{ ksi} \sin 45^\circ = 72.8^k$

$V_s = 50.3 + 72.8 = 123.1^k$

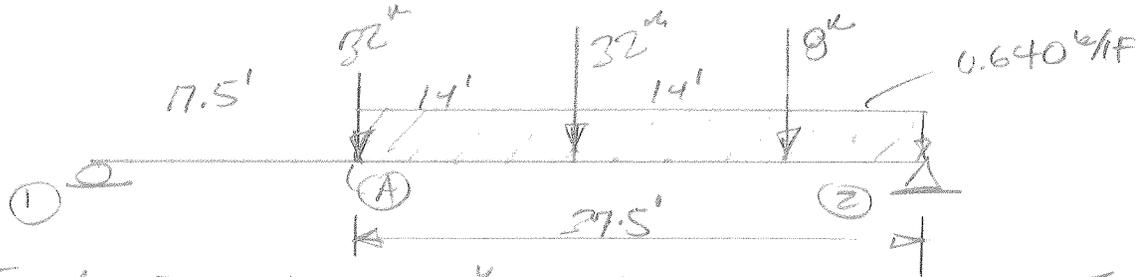
$V_n = 66.71 + 123.1 = 190^k$

$RF = \frac{0.9 \times 0.95 \times 190 - 1.25 (37.09 + 9.68)}{1.35 \times 73.5^k} = 1.05$
 (operating)

SHEAR LAST BENT-UP BAR

Stirrup spacing = 17"

Bend @ 17'-6" from E span



Truck Panel 1: $\frac{52 \text{ k} \times 37.5}{55} + \frac{32 \times 22.5}{55} + \frac{8 \times 9.5}{55}$
 $= 21.82 + 13.7 + 1.38 = 36.9 \text{ k}$

Lane Panel 1: $\frac{0.640 \times 27.5 \times (27.5/2)}{55} = 8.2 \text{ k}$

$V_{LLIM} = (1.33 \times 36.9 + 8.2) \times 0.812 = 46.5 \text{ k}$

$V_{DC} = 1.439 \times 27.5 - 1.439 \times 17.5 + 3.086/2 = 15.9 \text{ k}$
 $V_{DW} = 0.392 \times (27.5 - 17.5) = 3.92 \text{ k}$

Capacity: $V_c = 66.71 \text{ k}$

$V_s = \frac{2 \times 0.20 \times 33 \times 31.05}{17"} = 24.1 \text{ k}$

$V_n = 66.71 + 24.1 = 90.81$

$RF = \frac{0.9 \times 0.95 \times 90.81 - 1.25(15.9 + 3.92)}{1.33 \times 46.5} = 0.85 < 1.0$

May be an issue w/ shear termination.

REVIEW OF RATINGS - EUSTIS

Appendix A

SHEAR LAST BENT-UP BAR

$$Min A_v \geq 0.0316 \sqrt{f'_c} \frac{b_1 s}{f_y} = 0.0316 \sqrt{2.5} \left(\frac{21.5'' \times 17''}{33} \right) = 0.55'' < 0.40''$$

Use general procedure, 5.8.3.4.2 to determine β ALG previous calc Not valid

$$\beta = \frac{4.8}{(1 + 750 \epsilon_s)} \left(\frac{51}{39 + 5x_e} \right); \quad \theta = 29 + 3500 \epsilon_s$$

$$\epsilon_s = \frac{\frac{1 M_u}{d_v} + V_u}{E_s A_s}$$

middle depth - per EA

$$M_{DG} = 1.439 \times \frac{55}{2} \times 17.5 - \frac{1.439 \times 17.5^2}{2} + \frac{17.5}{27.5} \times \frac{3.086 \times 55}{4}$$

$$= 692.5 - 220.3 + 27.0 = 499 \text{ k}$$

$$M_{DW} = \frac{0.392}{1.439} (692.5 - 220.3) = 128.6 \text{ k}$$

$$M_{u,m} = 46.5^k \times 17.5^u = 813.8 \text{ k}$$

$$M_u = 1.35 \times 813.8 + 1.25 (499 + 128.6) = 1883 \text{ k} > V_u d_v$$

$$V_u = 1.35 \times 46.5 + 1.25 (15.9 + 3.92) = 87.6 \text{ k}$$

$$\epsilon_s = \frac{\frac{1883 \times 12}{31.05} + 87.6}{29000 \times 15 \times 16.56 \text{ in}^2} = 1.20 \times 10^{-3}$$

$$S_x = d_v \Rightarrow S_{x_e} = 31.05 \times \frac{1.38}{0.75 + 0.63} = 31.05''$$

$$\beta = \left(\frac{4.8}{1 + 750 \times 1.20 \times 10^{-3}} \right) \frac{51}{(39 + 31.05)} = \underline{1.84}$$

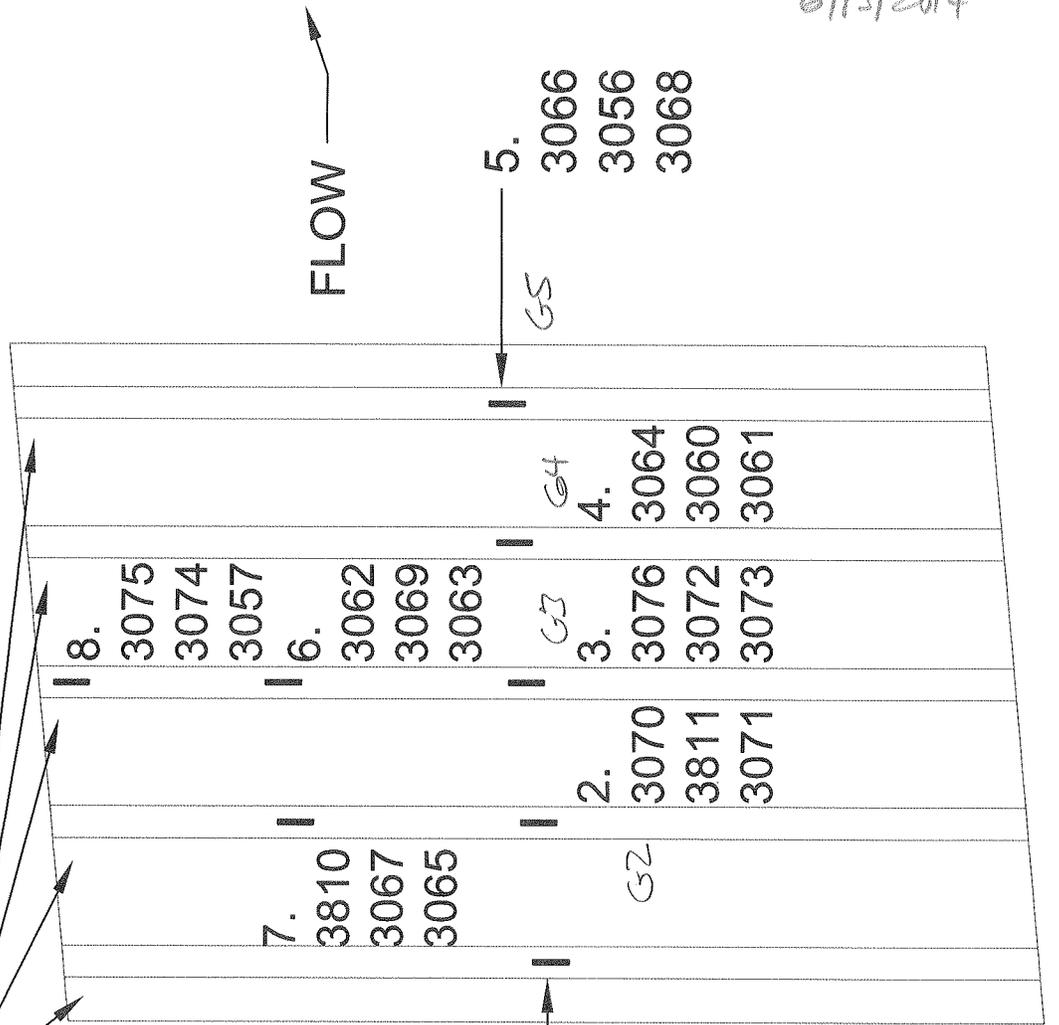
say 34" aggregate

$$\theta = 29 + 3500 \times 1.2 \times 10^{-3} = 33.2^\circ$$

6/13/2014

NOTE: EACH GAUGE POSITION CONSISTS OF 3 GAUGES, AS SHOWN IN THE TYPICAL T-BEAM CROSS SECTION.

GIRDERS

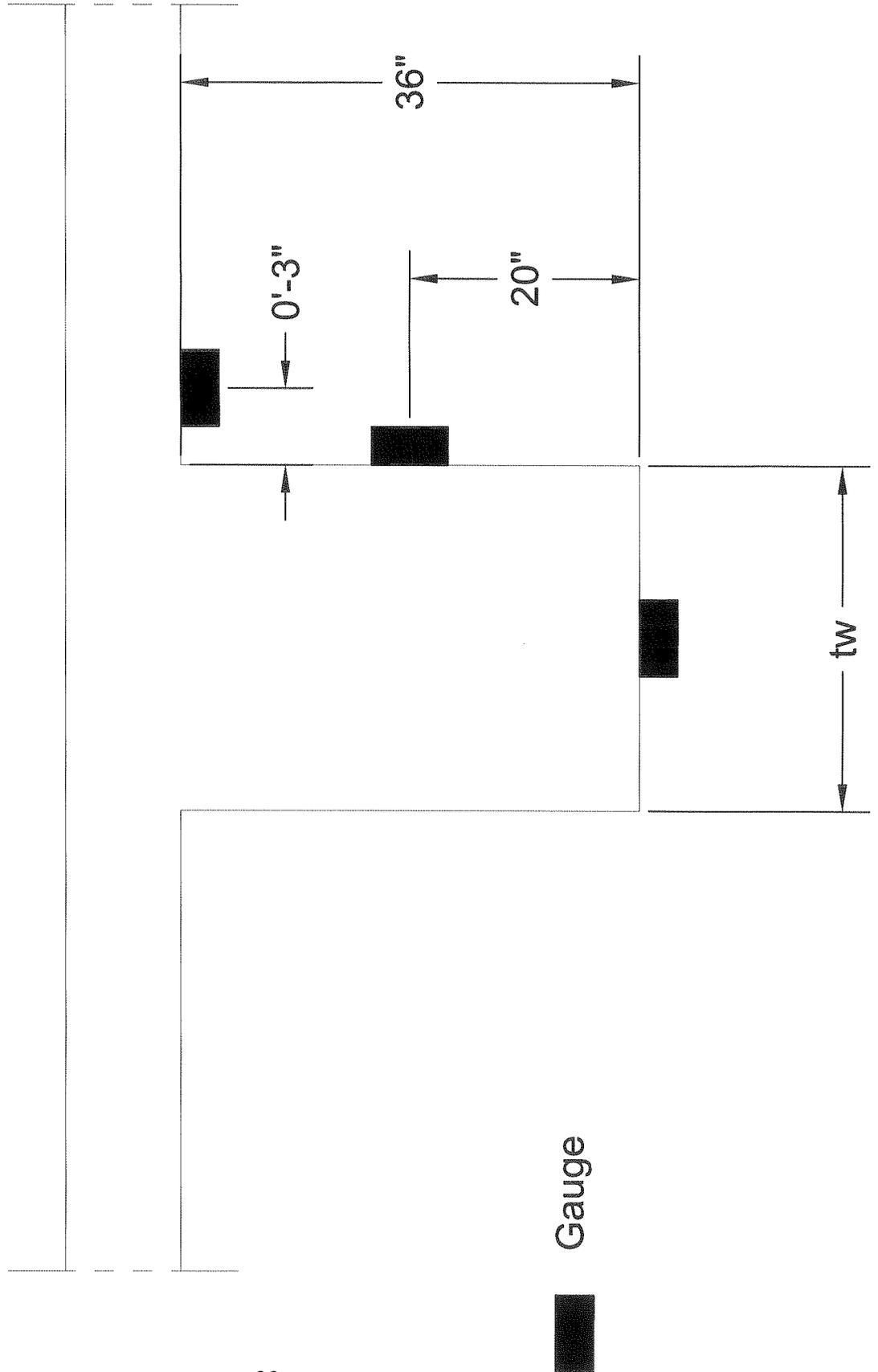


Section Number

Sensor Number

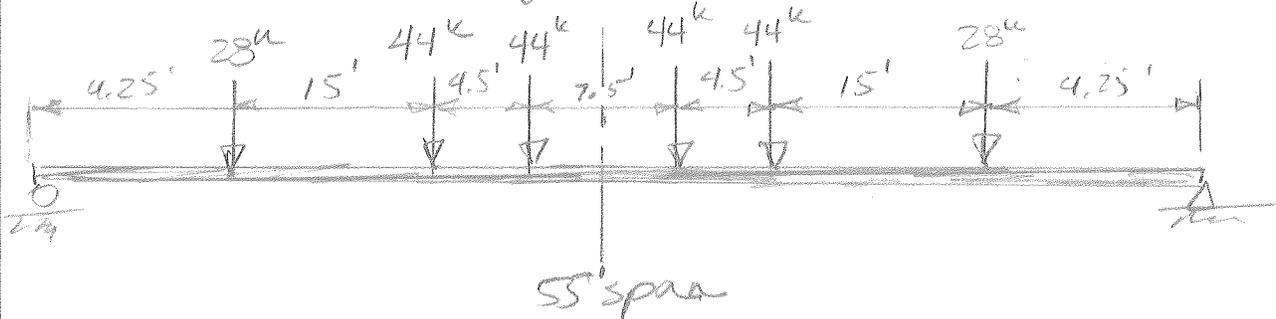
TOWARD SUGARLOAF

Typical T-Beam Cross Section with 3 Gauge Configuration for Each Gauge Position



ASSESS CRACKED VS. UNCRACKED

Approximate loading on full bridge, 4 trucks:



$$M_d \approx 2 \times \left(28 \times \frac{4.25 \times 50.75}{55} \times \frac{27.5}{50.75} + 44 \times \frac{19.25 \times 35.75}{55} \times \frac{27.5}{35.75} + 44 \times \frac{31.25 \times 23.75}{55} \times \frac{27.5}{31.25} \right)$$

$$M_d \approx 2 \times (59.5 + 423.5 + 522.5) = 2011 \text{ ft}\cdot\text{k}$$

Interior girder props: $I_{cr} = 166000 \text{ in}^4$ $S_{cr} = 5632 \text{ in}^3$
(spreadsheet) $y_{bot} = 29.5''$

Exterior girder props: $I_{cr} = 253500 \text{ in}^4$ $S_{cr} = 6836 \text{ in}^3$
(spreadsheet) $y_{bot} = 37.1 \text{ in}$

$$f'_c = 2500 \text{ psi} \Rightarrow E_c \approx 57\sqrt{2500} = 2850 \text{ ksi}$$

$$\epsilon \approx \frac{M}{S \times E_c} = \left(\frac{2011 \times 12}{3 \times 5632 + 2 \times 6836} \right) \times \frac{10^6}{2850} = \underline{\underline{277 \mu\epsilon}}$$

$$\text{Max measured } \epsilon = 87.8 \mu\epsilon \ll 277 \mu\epsilon$$

AGENTS LOAD DISTRIBUTION

Moment distribution inferred from measured strains:

$$M_{total} = \sum_{i=1}^5 E_c S_i \epsilon_i \quad \text{--- measured} \quad \text{let } E_c \text{ vary}$$

$$\Rightarrow 2011 \times 12 = \frac{E_c}{10^6} [17161 \times 52.4 + 12112 \times (87.8 + 87.3 + 75.9) + 17161 \times 44.6]$$

$$\Rightarrow 24132 = \frac{E_c}{10^6} (1664612 + 3040112) \Rightarrow E_c = 5129$$

$$M_1 = \frac{5129 \times 17161 \times 52.4}{10^6 \times 12} = 384 \text{ ft}\cdot\text{k} \quad \text{DF} \quad 384/2011 \times 2 = 0.382$$

$$M_2 = \frac{5129 \times 12112 \times 87.8}{10^6 \times 12} = 455 \text{ ft}\cdot\text{k} \quad 455/2011 \times 2 = 0.452$$

$$M_3 = \frac{5129 \times 12112 \times 87.3}{10^6 \times 12} = 452 \text{ ft}\cdot\text{k} \quad 452/2011 \times 2 = 0.450$$

$$M_4 = \frac{5129 \times 12112 \times 75.9}{10^6 \times 12} = 393 \text{ ft}\cdot\text{k} \quad 393/2011 \times 2 = 0.390$$

$$M_5 = \frac{5129 \times 17161 \times 44.6}{10^6 \times 12} = 327 \text{ ft}\cdot\text{k} \quad 327/2011 \times 2 = 0.325$$

$\Sigma = 2011 \text{ ft}\cdot\text{k} \quad \checkmark \text{ OK}$

$\Sigma 2.00 \text{ loads} \quad \checkmark \text{ OK}$

Max interior DF = 0.452

AASHTO value (per EA report) = 0.764 (2 lanes, m=1)

AASHTO DF appears to be quite conservative

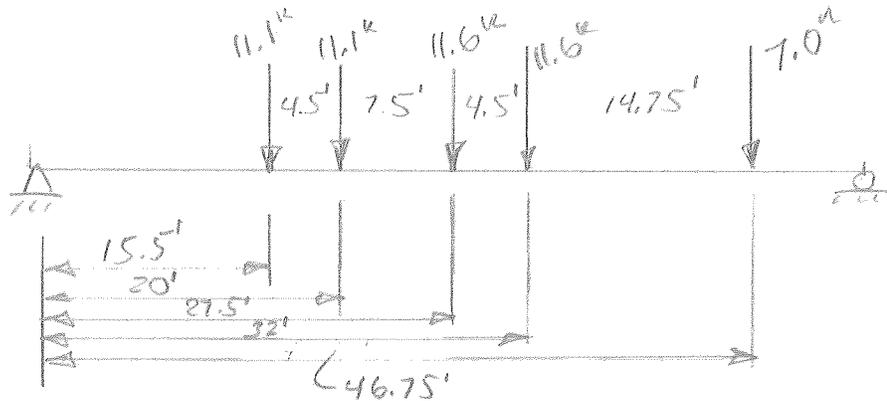
Bridge appears to exhibit significantly more load sharing than expected

COMPARE TEST MOMENTS W/HR-93+1M

6/26/2014

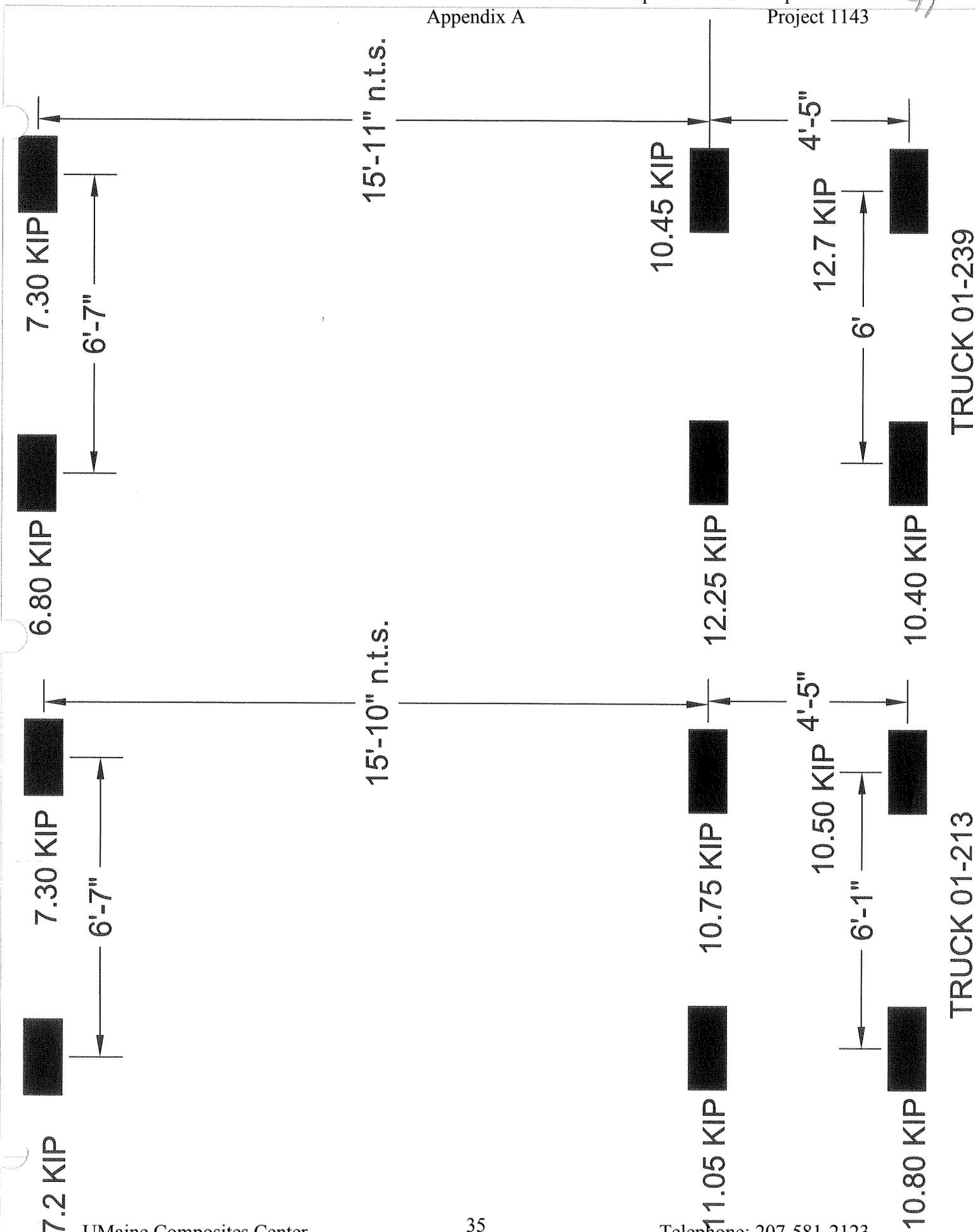
- Truck dimensions + wheel wts given on next 2 pp.
- 2-truck & 4-truck load positions summarized on pp. 11-12
- Will base analyses on avg wheel wts & dimension of pairs of similar trucks.

Approximate loading per lane w/ 4 trucks: (55' span)



Wheel	M_{max}	M_E
11.1 ^k	$\frac{11.1 \times 15.5 \times 39.5}{55} = 123.6^{k}$	$123.6 \times \frac{27.5}{39.5} = 86.0^{k}$
11.1 ^k	$\frac{11.1 \times 20 \times 35}{55} = 141.3^{k}$	$141.3 \times \frac{27.5}{35} = 111.0^{k}$
11.6 ^k	$\frac{11.6 \times 55}{4} = 159.5^{k}$	159.5 ^k
11.6 ^k	$\frac{11.6 \times 32 \times 23}{55} = 155.2^{k}$	$155.2 \times \frac{27.5}{32} = 133.4^{k}$
7.0 ^k	$\frac{7.0 \times 46.75 \times 8.25}{55} = 49.1^{k}$	$49.1 \times \frac{27.5}{46.75} = 28.9^{k}$
		$\sum 518.8^{k}$
		$\times 2 \text{ wheels}$
		<u><u>1037.6^k</u></u>

Appendix A



7.2 KIP

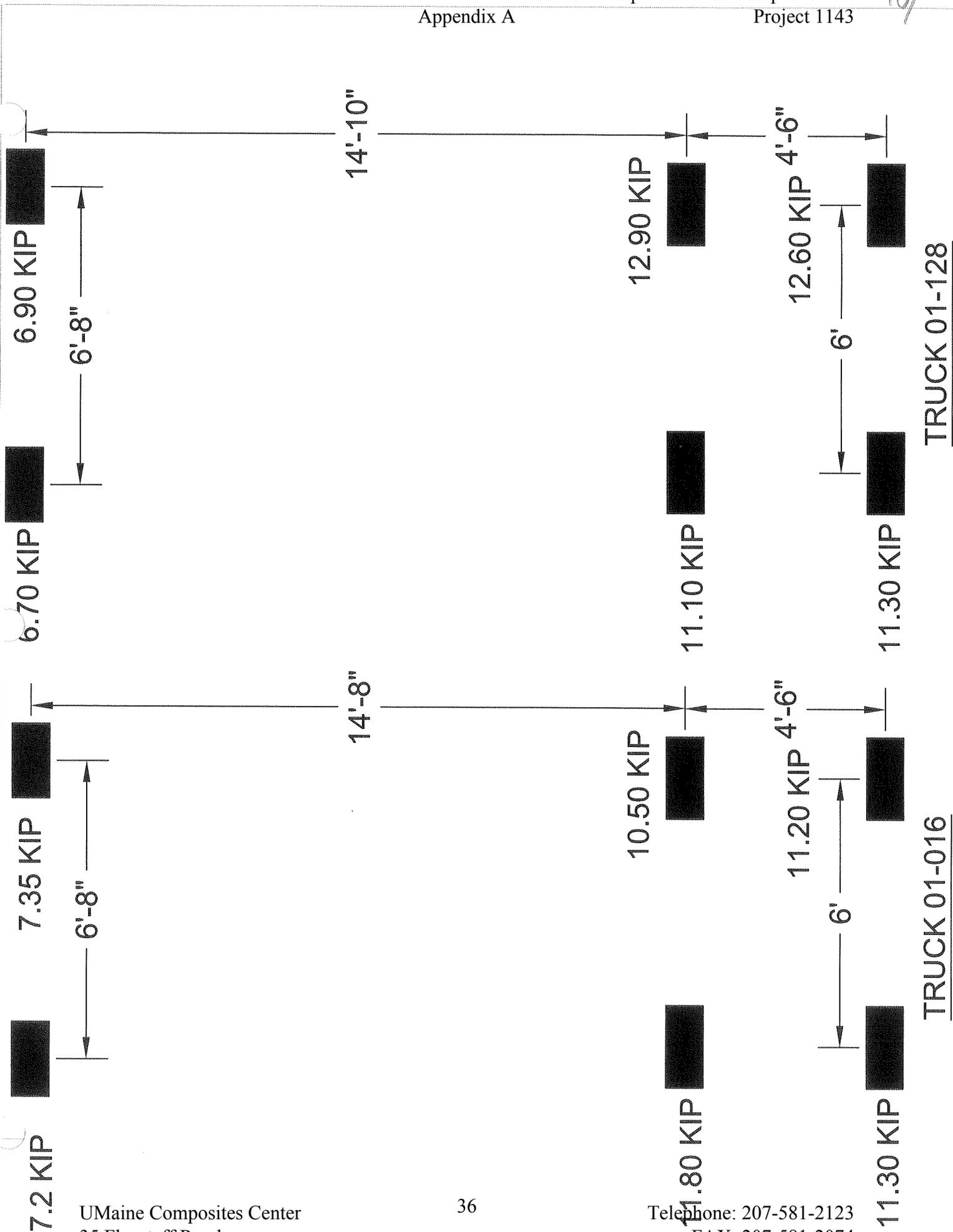
7.30 KIP

6.80 KIP

7.30 KIP

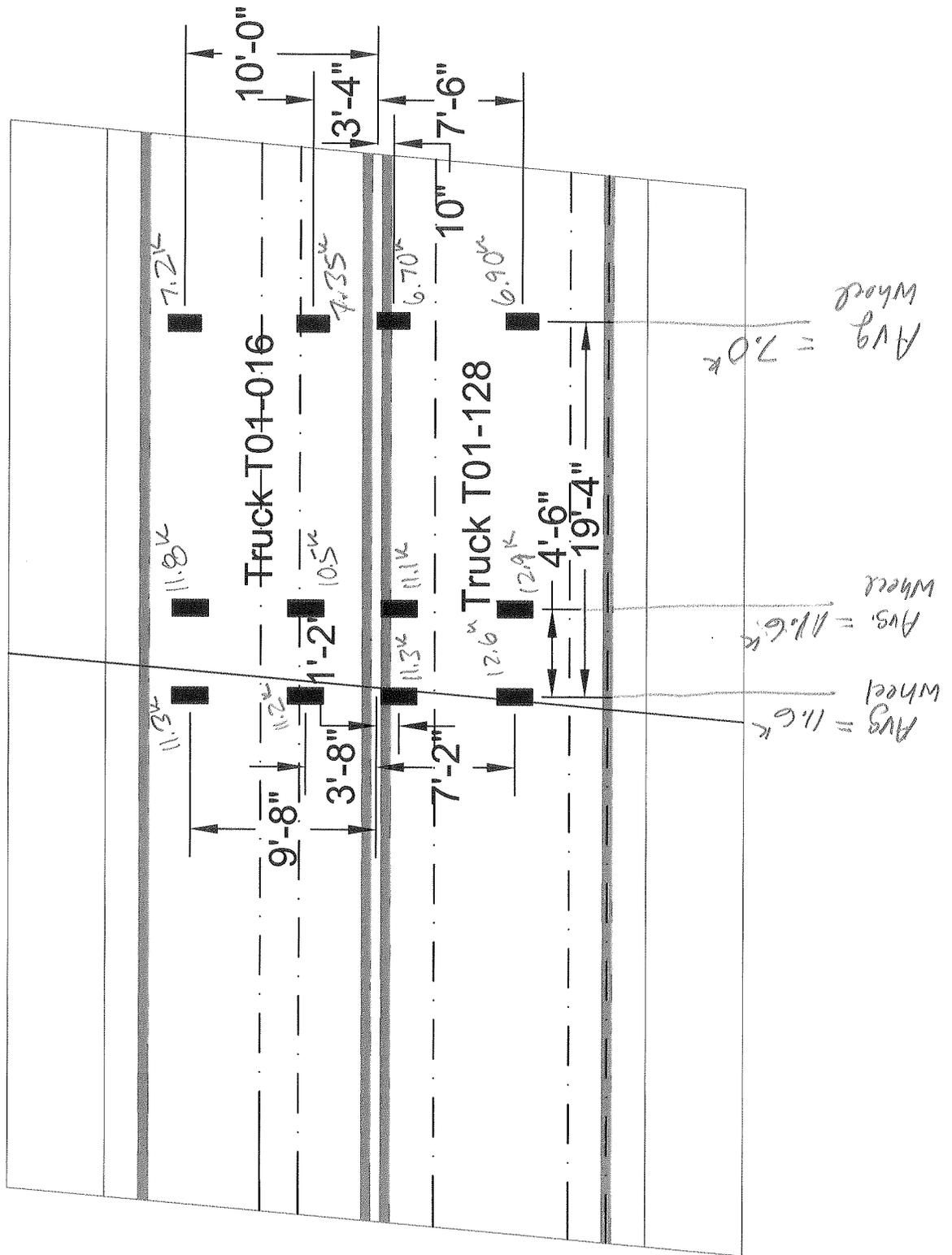
TRUCK 01-213

TRUCK 01-239



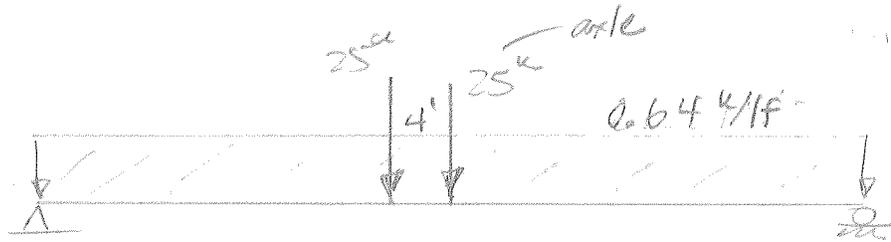
Lane Position for Single Truck Test

TWO



COMPARE TEST MOMENTS w/ HL-93+1M

LOADING per lane due to HL-93+1M



Tandem: $M_L = \frac{25 \times 55}{4} + \frac{25 \times 23.5 \times 31.5}{55} \times \frac{27.5}{31.5} = 638^{k}$

Truck: $M_L = 32 \times 55/4 + (32+8) \times \frac{13.5 \times 41.5}{55} \times \frac{27.5}{41.5} = 440 + 270 = 710^{k}$

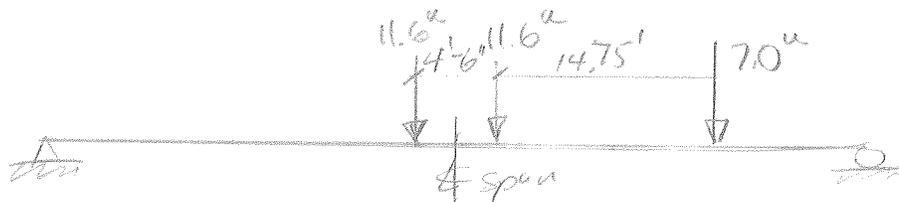
lane: $\frac{0.64 \times 55^2}{8} = 242^{k}$

$M_{U+M} = 1.33 \times 710 + 242 = 1186^{k}$

We produced $\approx \frac{1038}{1186} \times 100\% = 87.5\%$ of HL-93+1M

ASSESS RATING FACTORS

- Compare 2 truck + 4 truck loading
- two truck loads:



$M_L = \left[\frac{11.6 \times 25.25 \times 29.75}{55} \times \frac{27.5}{29.75} \times 2 + \frac{7 \times 44.5 \times 10.5}{55} \times \frac{27.5}{41.5} \right] \times 2$ trucks
 $= (316.9 + 96.75) \times 2 = 707.3^{k}$

ASSESS RATING FACTORS

Max $\mu\epsilon$ w/ 4 trucks = 87.8 $\mu\epsilon$ (section 2)

Max $\mu\epsilon$ w/ 2 trucks = 425.4/8 = 53.1 $\mu\epsilon$ (section 2)

$$\frac{707.2}{53.1} = 13.32 \frac{\mu\epsilon}{\mu\epsilon} \quad \frac{1037.6}{87.8} = 11.8 \frac{\mu\epsilon}{\mu\epsilon}$$

\therefore reasonably linear increase in strain w/ load

Compare predicted strain w/ measured:

DF for 2 lanes loaded, interior side = 0.764 (EA report)

$$\text{predicted strain} = \frac{M}{ES_{uncr}} \left(\frac{1038 \text{ ft} \cdot \text{lb} \times 12}{2850 \text{ ksi} \times 12112 \text{ in}^3} \right) \times 10^6 \times 0.764 = 276 \mu\epsilon$$

use uncracked section, consistent w/ observed response

based on nominal values... probably low.

previous analysis indicates higher E_c is appropriate, up to 5129 ksi. Say $E_c \approx 57 \sqrt{5000} = 4030 \text{ ksi}$.
conservatively say $f_t = 2 \times \text{nominal}$

$$\text{predicted strain} = \left(\frac{1038 \times 12}{4030 \times 1212} \right) \times 10^6 \times 0.764 = 195 \mu\epsilon$$

$$K_a = \frac{E_c}{E_T} - 1 = \frac{195}{87.8} - 1 = 1.221 \text{ --- conservative}$$

$T/W = 1 > 0.7$, but cannot necessarily extrapolated due to uncracked response $\Rightarrow K_b = 0.5$

$$K = 1 + 1.221 \times 0.5 = 1.61$$

Operating RF for bending = 0.71 (interior)

$$\Rightarrow \text{New operating RF} = 1.61 \times 0.71 = \underline{1.14} > 1.0 \checkmark \text{ OK}$$

✓
WGD
6/27/11

WHITEFIELD - LL TEST ANALYSIS

Assumptions & Properties:

- $f'_c = 2500 \text{ psi}$, $n = 10$
 - Concrete wearing surface is intact & contribute to girder stiffness, effective deck $t = 10.55"$. Conservative, since this reduces calculated strain.
 - Analysis will be done assuming an average measured truck weight
 - Analyze for max measured strain, loading 47 w/ 2 trucks located close to ξ , gauge B3067 recorded $415/8 = 51.9 \mu\epsilon$
 - $I_{uncr} = 391030 \text{ in}^4$; $I_{cr} = 172438 \text{ in}^4$
 $\bar{y} = 31.4"$ (bottom); $\bar{y} = 32.5"$ (bottom)
 $S_{bot} = 12467 \text{ in}^3$; $S_{bot} = 5144 \text{ in}^3$
- Assumes $n = 8$, actual concrete strength taken as 5 ksi (see next page)

Live load analysis:

LL DF = 0.703 (see RF analysis)

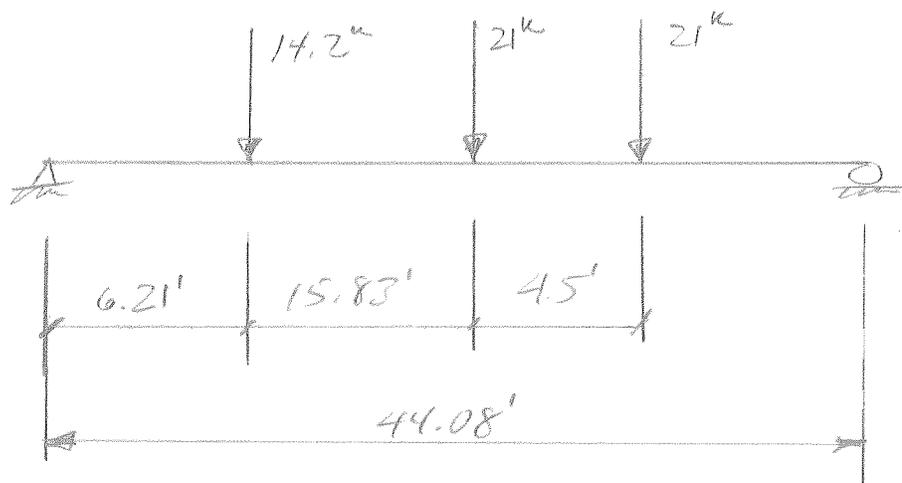
Avg truck tandem = $(43.95 + 40.25)/4 = 21^k \text{ ea}$

Avg truck steer = $(19.45 + 13.85)/2 = 14.2^k$

Avg truckst = $2 \times 21 + 14.2 = 56.2$

$= (54.7 + 57.8)/2 = 56.2^k$
OK

span = 44.08' (see RF analysis)



2/
WGP
6/27/11

WHITEFIELD - LL TEST ANALYSIS

LIVE LOAD Analysis:

$$M_{LL} = \frac{21 \times 44.08}{4} + \frac{21 \times 26.54 \times 17.54}{44.08} \left(\frac{22.04}{26.54} \right)$$

conservatively increase fix 2 \rightarrow

$$+ \frac{14.2 \times 6.21 \times 37.87}{44.08} \left(\frac{22.04}{37.89} \right) = 231.4 + 184.2 + 44.1$$

$$E_c \approx 57 \sqrt{5000} = 4030 \text{ ksi}$$

$$= \frac{460 \text{ Hole}}{\text{lane}} = 0.703 \times 460 \text{ Hole} = 323 \text{ gorden}$$

$$\epsilon_{uncr} = \frac{M}{S_{uncr} E} = \frac{(323 \times 12)}{12467 \times 4030} \times 10^6 = 77.1 \mu\epsilon \text{ (uncracked)}$$

$$\epsilon_{cr} = \frac{M_y}{E I_{cr}} = \frac{(323 \times 12)}{5144 \times 4030} \times 10^6 = 187.1 \mu\epsilon \text{ (cracked)}$$

Max measured $\epsilon = 51.9 \mu\epsilon$... section is behaving much like an uncracked beam.

Note that when adjusting the RF per the AASHTO-MBE, it is conservative to assume a stiffer section.

4/
W/G
6/27/11

WHITEFIELD - LL TEST ANALYSIS

Closer look @ LL Dist:

Based on measured strains, we can assess LL distribution knowing girder section props.

$$M = \sum S_{int} \epsilon_{int} E + \sum S_{ext} \epsilon_{ext} E = E \sum S \epsilon$$

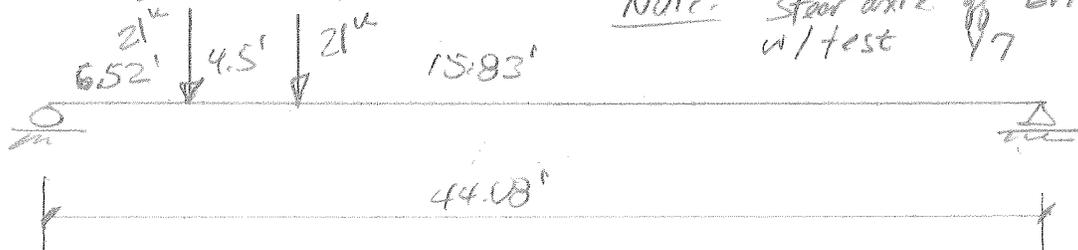
@ midspan: interior girder strains = 51.9 $\mu\epsilon$ & 40.7 $\mu\epsilon$
exterior girder strains = 12.9 $\mu\epsilon$ (1 failed gauge)
(seems very low)

@ 1/4 point: interior girder strains = 46.6 $\mu\epsilon$
exterior girder strain = 24.0 $\mu\epsilon$ } conservative - discrepancy in load in span 2

Will use 1/4 pt strains to assess load distribution.

$$M = \sum_{i=1}^{4 \text{ girders}} E_c S_i \epsilon_i$$

Note: Star axle of bridge w/ test



$$M_{1/4pt} = \frac{21 \times 11.02 \times 33.06}{44.08} + \frac{21 \times 6.52 \times 37.56 \times 11.02}{44.08 \times 37.56}$$

$$= 173.6 + 34.23 = 208 \text{ k-ft/lane}$$

$$\Rightarrow (208 \times 12 \times 7) = \frac{E}{10^6} \left[\frac{1}{2} \times 12467 \times 46.6 + \frac{1}{2} \times 14218 \times 24.0 \right]$$

$$\Rightarrow E = 2709 \text{ ksi} \leftarrow \text{inconsistent w/ } f'_c \approx 2300 \text{ psi}$$

$$M_{interior} = \frac{2709 \times 12467 \times 46.6}{1000000 \times 12} = 131 \text{ ft-k}; DF = \frac{131}{208} = 0.630$$

$$M_{exterior} = \frac{2709 \times 14218 \times 24}{1000000 \times 12} = 77 \text{ ft-k}; DF = \frac{77}{208} = 0.370$$

1.00 ✓

APPENDIX B – EUSTIS BRIDGE TESTING

The Eustis bridge was tested in 2013 with one truck and then retested in 2014 with up to four trucks. A summary of the loading, instrumentation and representative data is presented for the 2014 test only.

TEST PLAN

Three series of tests were conducted with one, two and four trucks with trucks either rolling across the bridge at a slow speed or static positions (4 trucks). The bridge was closed to all other traffic when data was collected. Truck weights, dimensions, and positions during the four truck loading case are presented in Appendix A. Lane positioning for the single and two truck load cases is given in Figure B1 and Figure B2. These two load cases had the trucks starting off the bridge and then travelling across the bridge at a slow speed (approximately 3-5 mph).

Lane Position for Single Truck Test

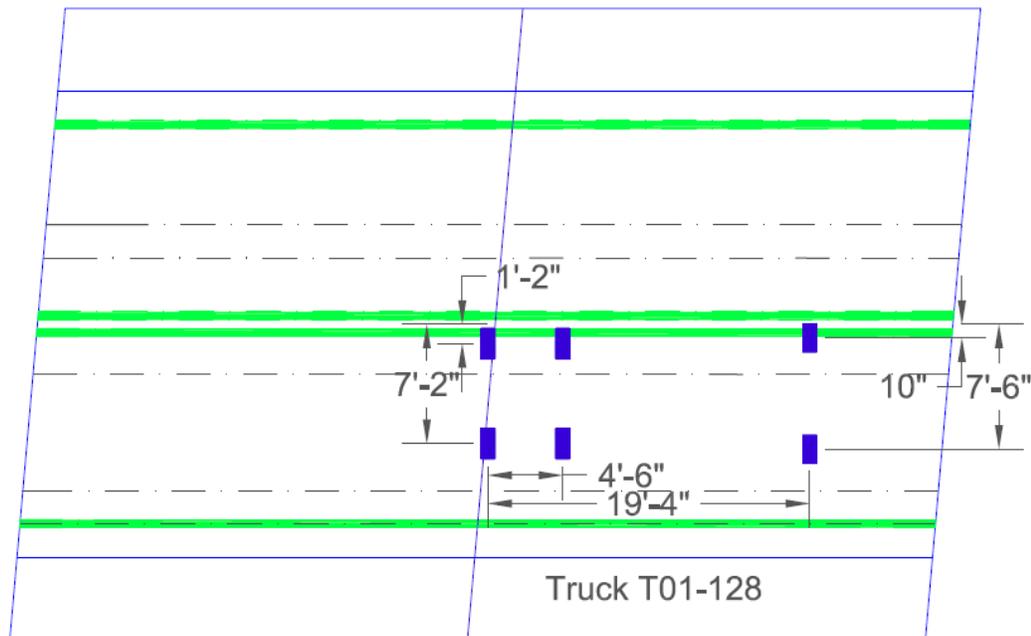


Figure B1 – Truck lane position during single truck loading

Lane Position for Single Truck Test

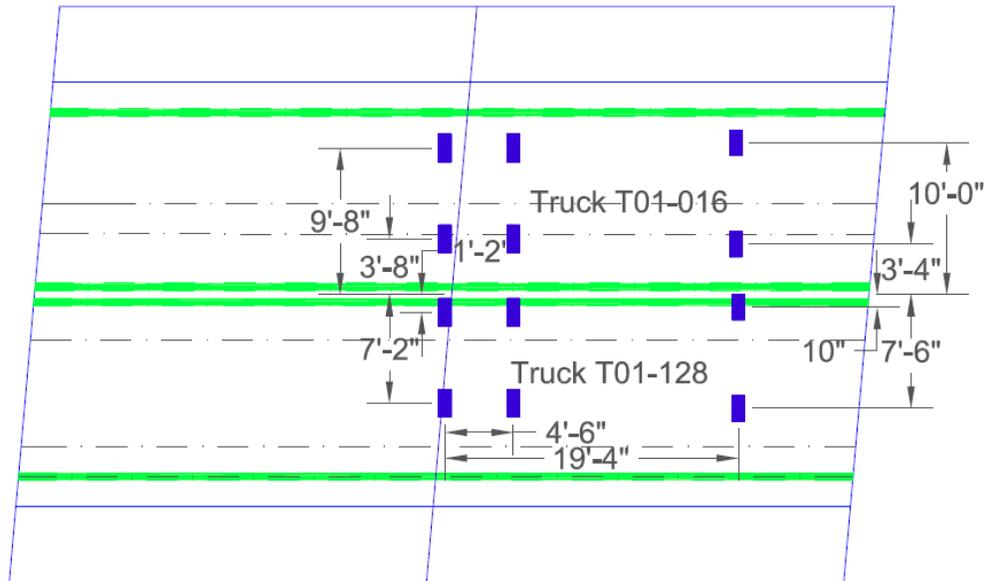


Figure B2 – Lane position for two truck loading

INSTRUMENTATION

Twenty-four strain transducers were used during the testing of the Eustis bridge. They were located in groups of three at a given cross section and are shown in Appendix A. Gages used extensions as shown in Figure B3 to average any effects of cracks in the concrete.



Figure B3 – Installed gage on bottom of T-beam with 21" extension

REPRESENTATIVE DATA

Plot of strain data from the two and four truck loading can be seen in Figures B4 and B5 where strain (microstrain) is plotted versus time from the start of the test. The test software automatically zeroes the data.

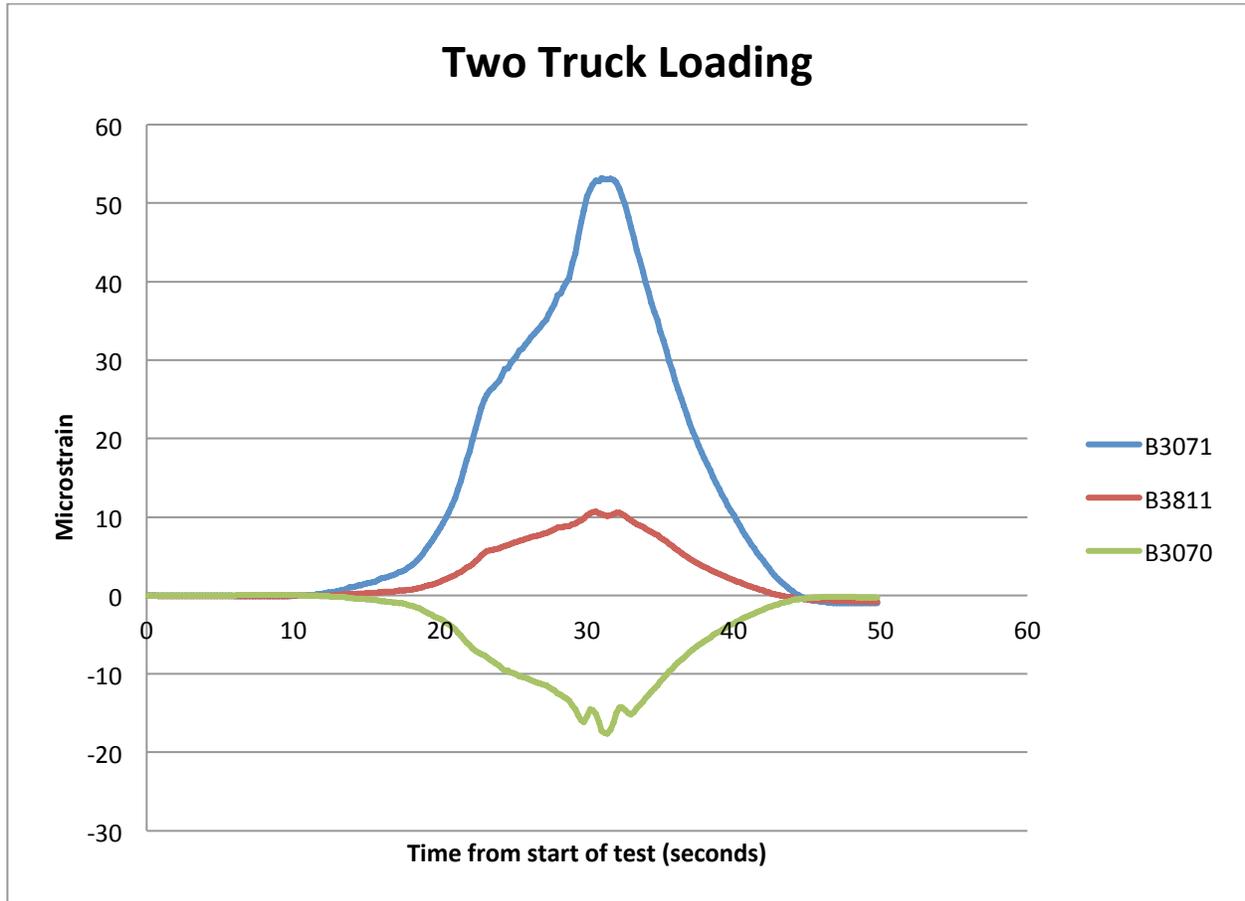


Figure B4 – Chart of strain during two truck loading

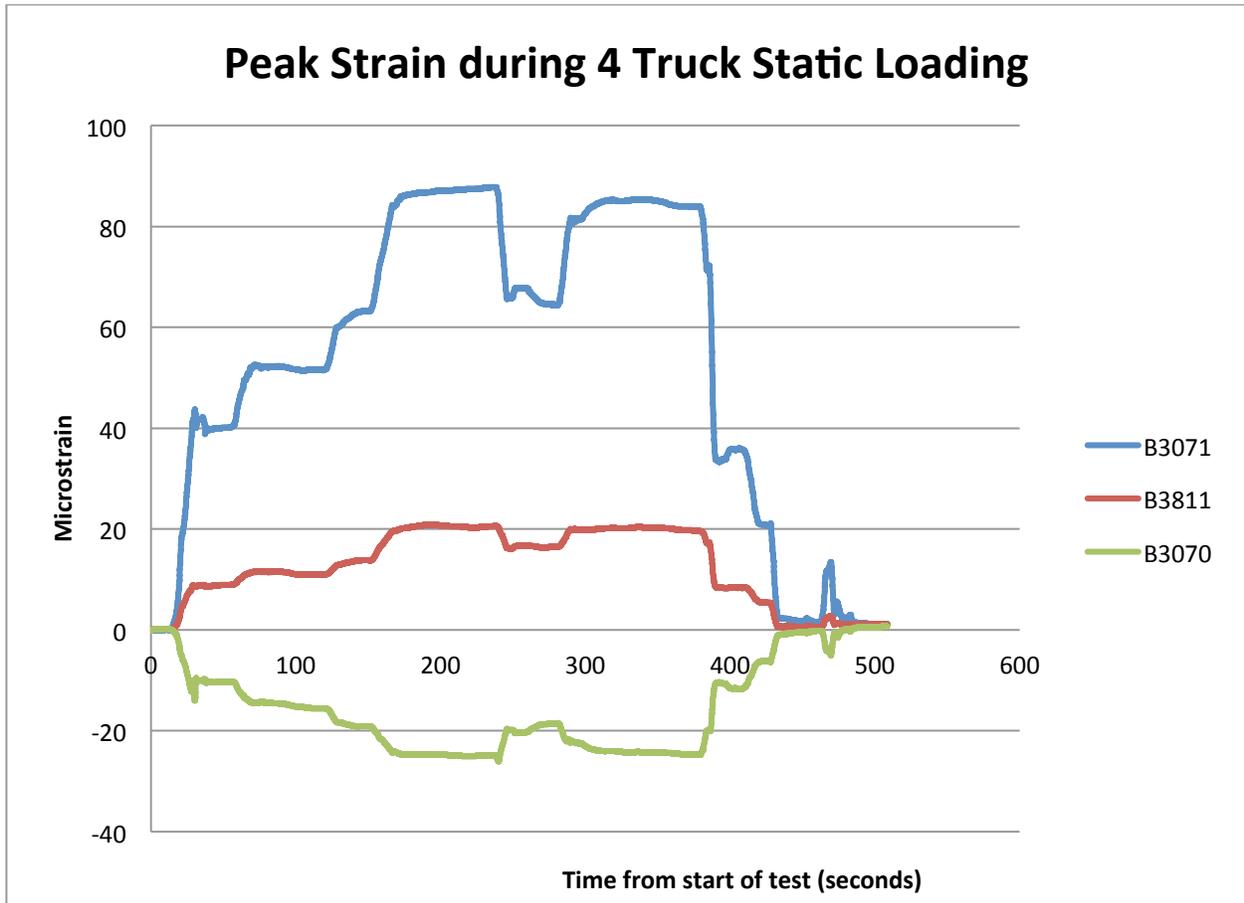


Figure B5 – Chart of strain during four truck (static) loadings

APPENDIX C – WHITEFIELD BRIDGE TESTING

TEST PLAN

The bridge in Whitefield was tested in 2013 with one and two trucks similar to previous bridges. Loading and instrumentation was directed to evaluate load distribution. Test vehicle information can be seen in Figures C1 and C2.

License Plate Number- T01-201

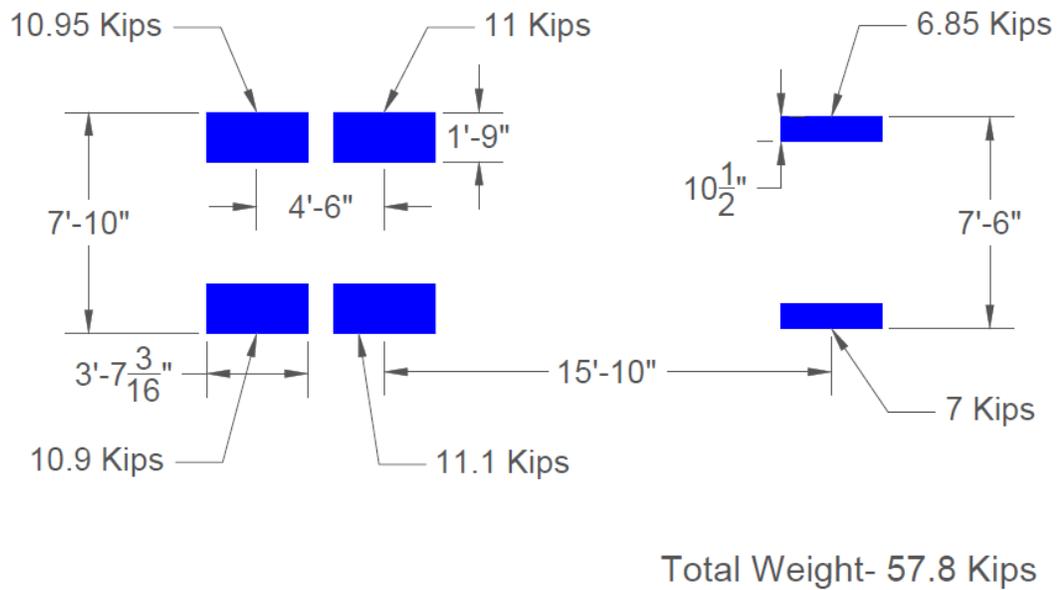


Figure C1 – Truck weight and dimensions

License Plate Number- T01-210

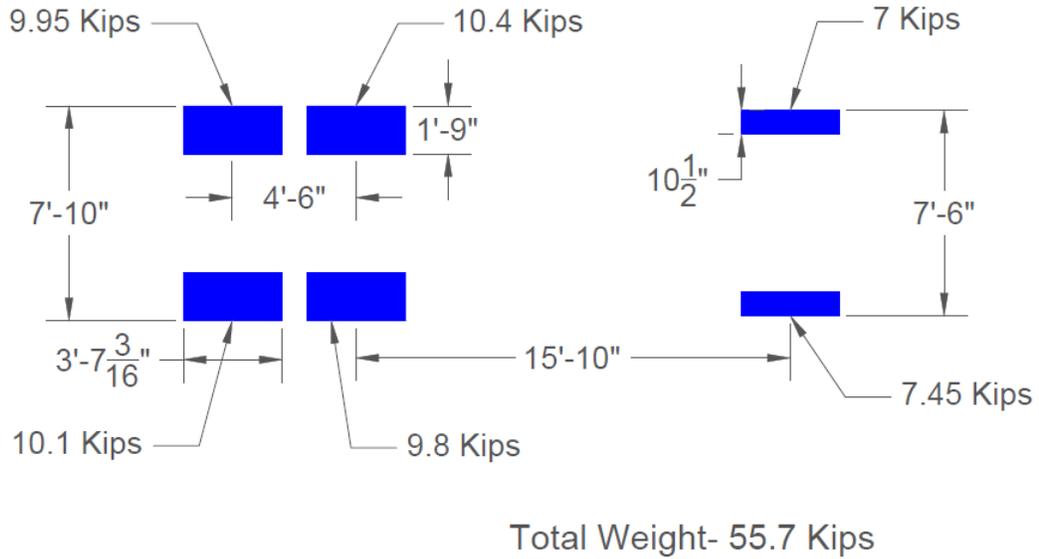


Figure C2 – Truck weight and dimensions

Single truck lane positions for five tests are shown in Figure C3. Two truck load cases were symmetric about the centerline of the bridge and have the trucks on the outer and inner positions of each lane as shown in Figure C4. The second two truck load case had both trucks side by side as shown in Figure C4 except they were 2 ft outward from the centerline to the inside of the rear dual wheels.

NOTE: ALL TRUCK POSITIONS ARE MEASURED FROM THE INSIDE OF THE UPSTREAM CURB TO THE MIDDLE OF THE DRIVER SIDE REAR DUALY AXLE.

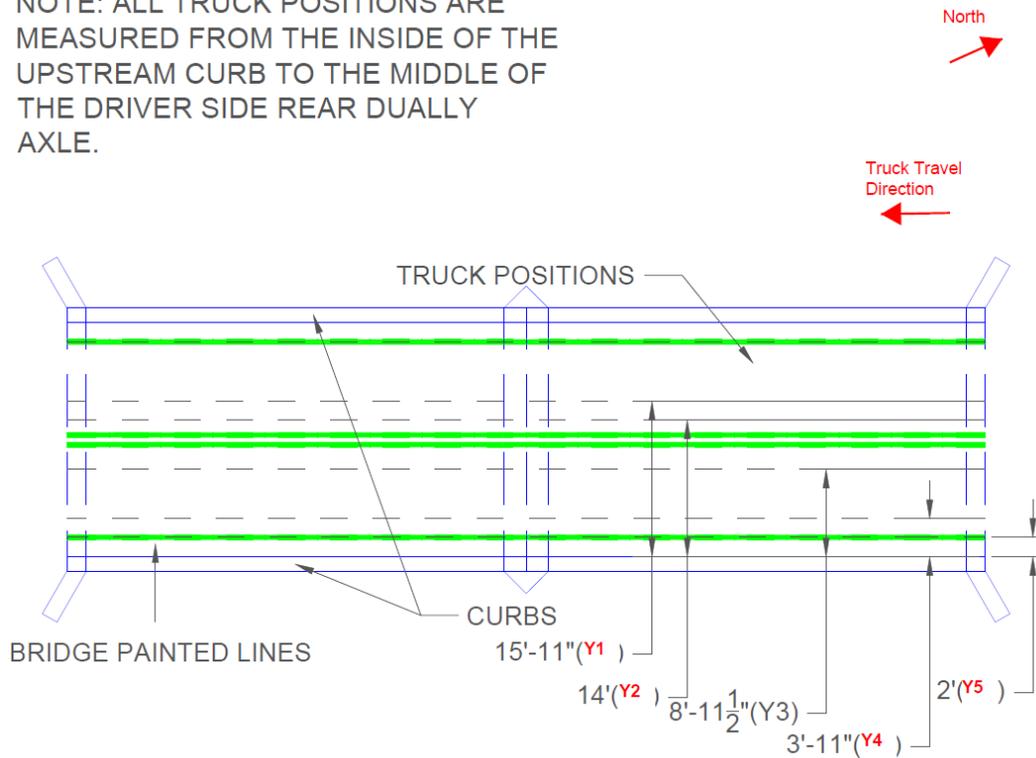


Figure C3 – Single truck truck positions during tests 1-5

Y6 Truck Position- Each truck is two feet from the curb

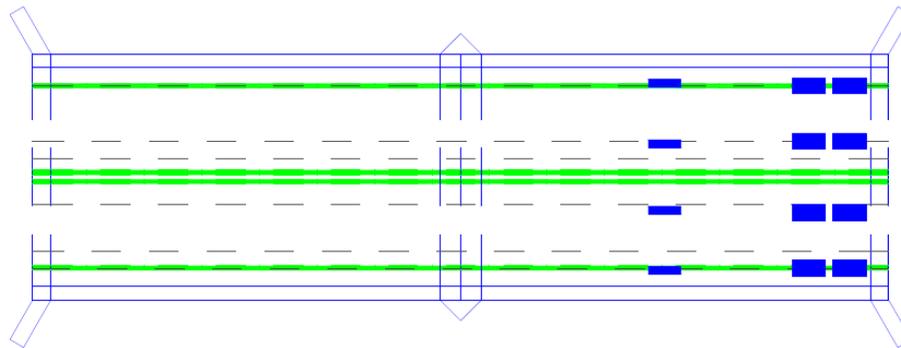


Figure C4 – Two truck lane position during test Y6 (2 ft from curb).

INSTRUMENTATION

Instrumentation was concentrated in the northerly span. All four girders were instrumented at midspan with three gages across the height of the cross section. Similarly two girders were instrumented at the quarter points of the eastern exterior and first interior girder. Two girders were instrumented at their quarter points in the southerly span as seen in Figure C5. Gages were located on the bottom of the section, 23 inches up the web and on the underside of the deck.

NOTE: EACH GAUGE POSITION CONSISTS OF THREE GAUGES, AS SHOWN IN THE TYPICAL SECTION VIEW.

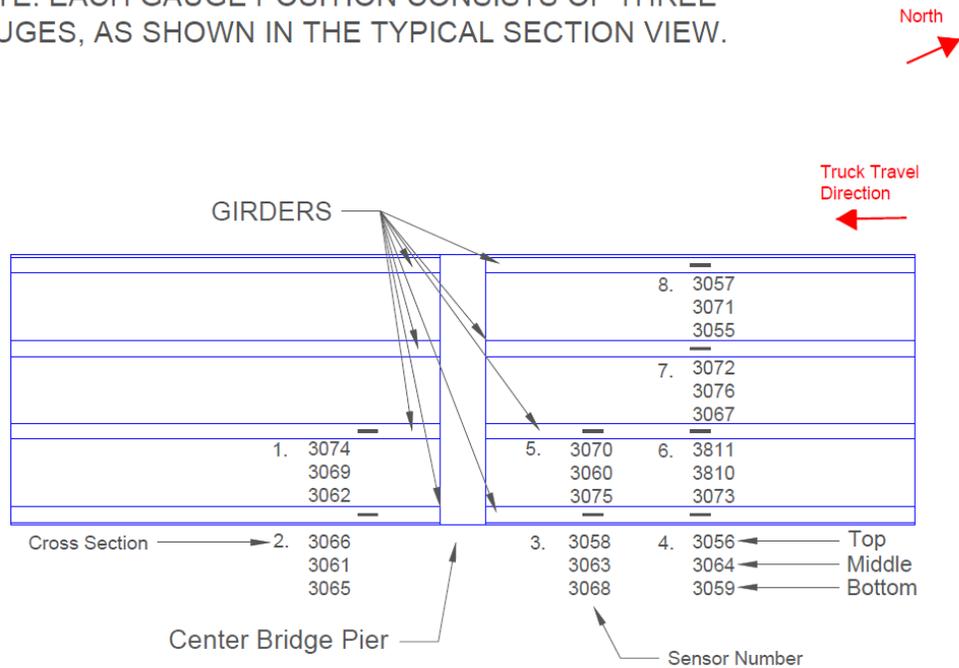


Figure C5 – Gage locations during Whitefield bridge test

REPRESENTATIVE DATA PLOTS

Representative plots of strain versus front tire position on the bridge are given in Figure C6 and C7. Similar plots are available for all other gages.

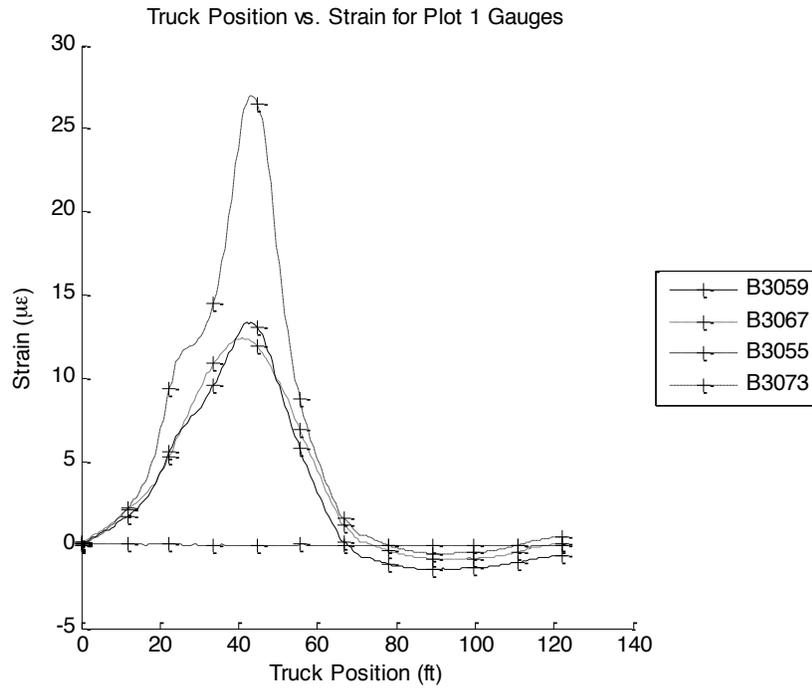


Figure C6-Test Y1 Bottom Sensors (Note – Gage B3055 was found debonded when removed after test. It appears to not record any measurement during the testing).

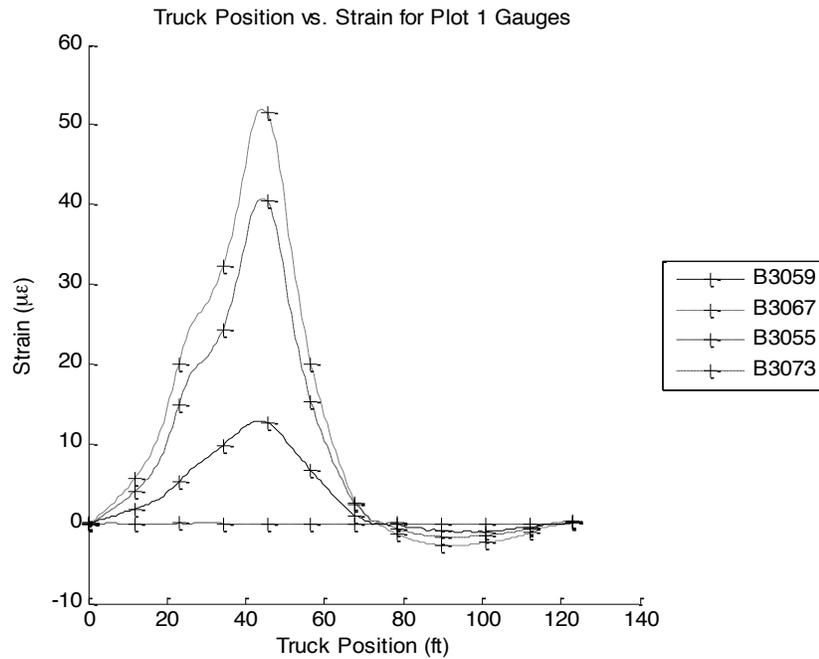


Figure C7-Test Y7 (two trucks) Bottom gages

APPENDIX D – EDDINGTON BRIDGE CALCULATIONS AND TEST PLAN

CALCULATIONS

$E_c := 1820\sqrt{5} \cdot \text{ksi}$	Concrete MOE	$E_c = 4.07 \times 10^3 \text{ ksi}$
$\epsilon_{\text{ext}} := 9.0 \cdot 10^{-6}$	Strain in exterior girder	
$\epsilon_{\text{int}_1} := 13.6 \cdot 10^{-6}$	Strain in first interior girder	
$\epsilon_{\text{int}_2} := 13.5 \cdot 10^{-6}$	Strain in second interior girder	
$S_{\text{int}} := 5169 \text{ in}^3$	Section modulus of interior girder (5ksi - spreadsheet)	
$S_{\text{ext}} := 17689 \text{ in}^3$	Section modulus of exterior girder (5ksi - spreadsheet)	
$M_{\text{ext}} := E_c \cdot \epsilon_{\text{ext}} \cdot S_{\text{ext}}$	Moment in ext girder inferred from strain data for uncracked section	
$M_{\text{ext}} = 53.99 \text{ kip-ft}$		
$M_{\text{int}_1} := E_c \cdot \epsilon_{\text{int}_1} \cdot S_{\text{int}}$	Moment in 1st interior girder inferred from strain data for uncracked section	
$M_{\text{int}_1} = 23.84 \text{ kip-ft}$		
$M_{\text{int}_2} := E_c \cdot \epsilon_{\text{int}_2} \cdot S_{\text{int}}$	Moment in 2nd interior girder inferred from strain data for uncracked section	
$M_{\text{int}_2} = 23.66 \text{ kip-ft}$		

Analytical Solution for Moment to Girders

$\text{span} := 357 \text{ in}$	Measured span of bridge	$\text{span} \cdot 5 = 14.875 \text{ ft}$
$M_{\text{test}} := 21.3 \text{ kip} \cdot \left(\frac{\text{span}}{2} - \frac{54 \text{ in}}{2} \right)$	Total moment due to test vehicle (2 rear axles centered over midspan of widened section, puts front tire off bridge)	
$M_{\text{test}} = 268.91 \text{ kip-ft}$		
$S_t := S_{\text{ext}} + S_{\text{int}_1} + S_{\text{int}_2}$	Total of all section moduli for widened section	
$S_t = 2.803 \times 10^4 \text{ in}^3$		
$\frac{M_{\text{ext}}}{M_{\text{test}} \cdot \left(\frac{S_{\text{ext}}}{S_t} \right)} = 31.811\%$	$\frac{M_{\text{int}_1}}{M_{\text{test}} \cdot \left(\frac{S_{\text{int}_1}}{S_t} \right)} = 48.071\%$	$\frac{M_{\text{int}_2}}{M_{\text{test}} \cdot \left(\frac{S_{\text{int}_2}}{S_t} \right)} = 47.717\%$

TEST PLAN

One truck was used to load the available portion of the Eddington Bridge as construction was being done on headwalls and railings on the southerly side of the bridge. The instrumentation and loading plan was adjusted the day of the test to concentrate on the northerly half of the bridge.

The single truck on site was weighed and measured as shown in Figure D1.

License Plate Number- T01-229

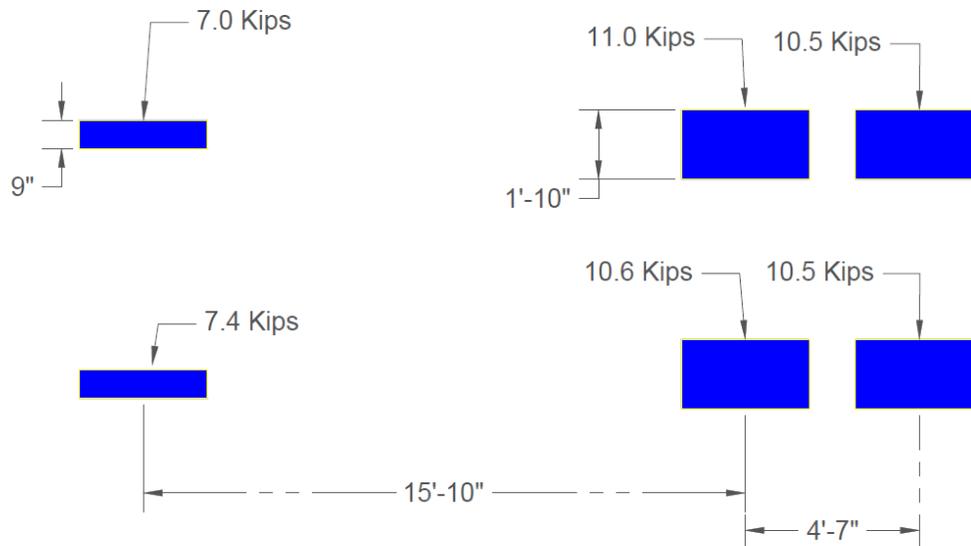


Figure D1 – Truck weight and dimensions

Truck lane positions during the test are shown in Figure D2.

NOTE: ALL OF THE TRUCK POSITIONS ARE MEASURED FROM THE INSIDE FACE OF THE DOWNSTREAM CURB TO THE MIDDLE OF THE REAR DUALY

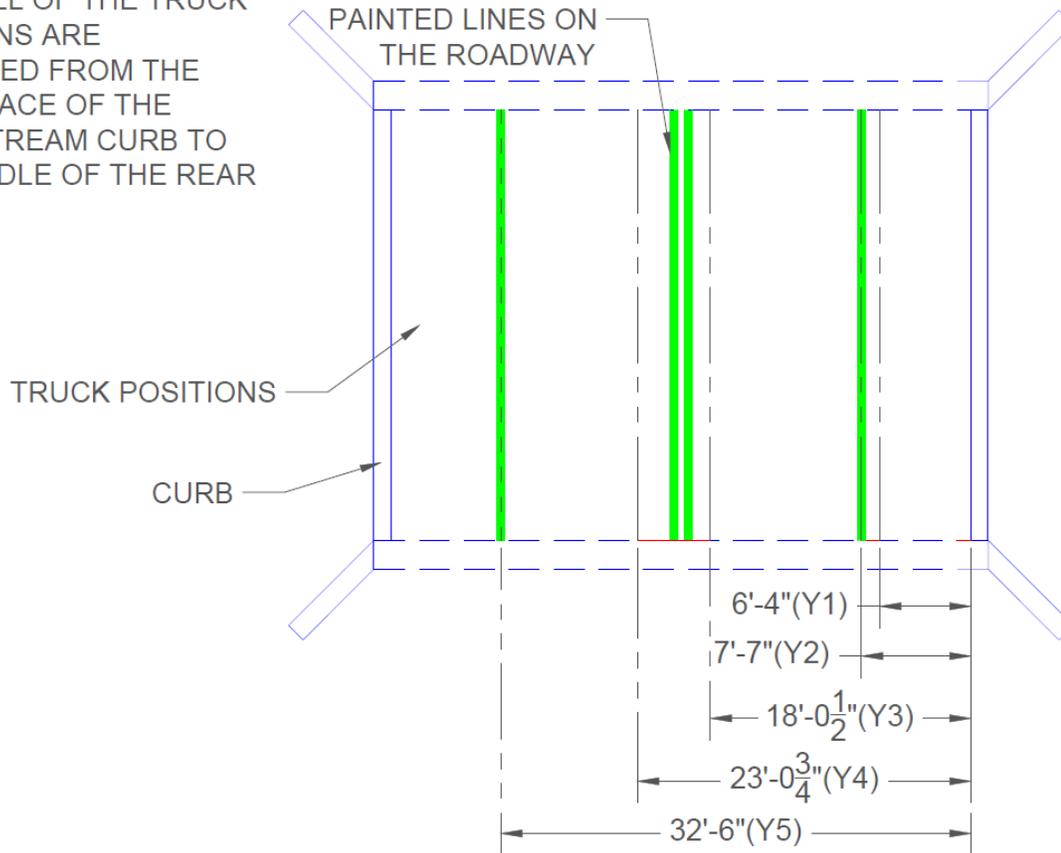


Figure D2 – Truck lane positions during tests

INSTRUMENTATION

Twenty-four strain transducers were used during the test of the Eddington Bridge. They were concentrated toward the northerly half of the bridge as shown in Figure D3 where each cross section shown has three gages, one on the bottom of the web and deck and one at mid height of the web.

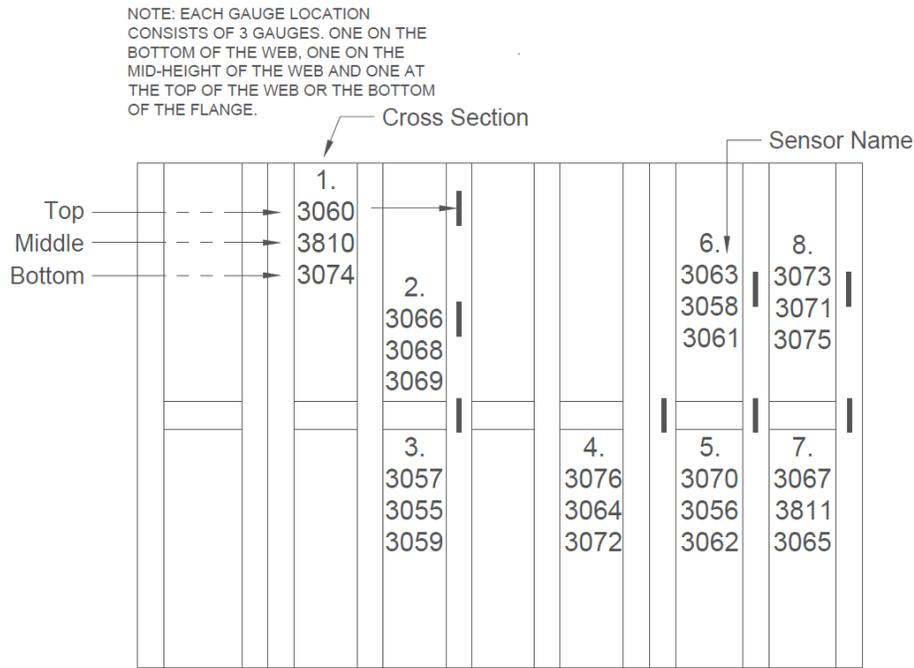


Figure D3 – Gage locations

REPRESENTATIVE DATA PLOTS

A representative plot of strain versus front tire position on the bridge is given in Figure D4. Similar plots are available for all other gages.

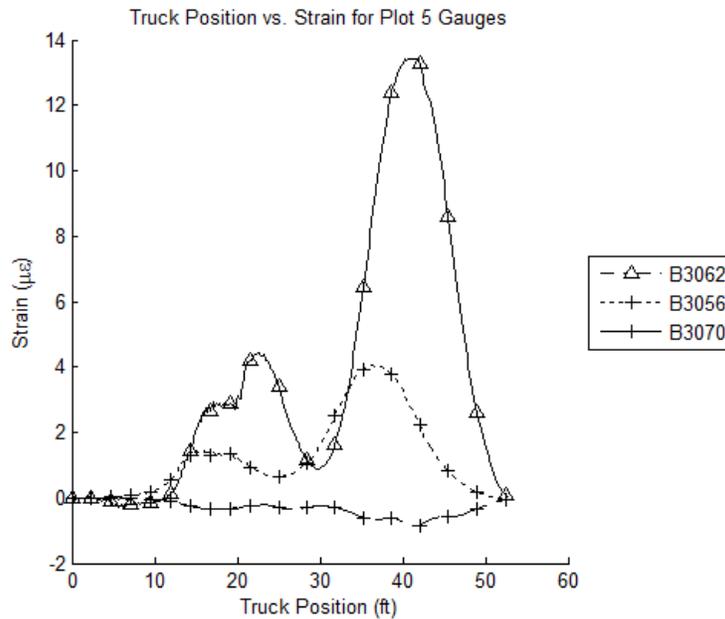


Figure D4 – Plot of strain during 1st test versus front tire position.

APPENDIX E – ALBION BRIDGE CALCULATIONS AND TEST PLAN

TEST PLAN

One truck was used to load the Albion bridge in lane positions parallel to the road alignment. The truck weight and dimensions and lane positions during the test can be seen in Figures E1 and E2.

License Plate Number T01-845

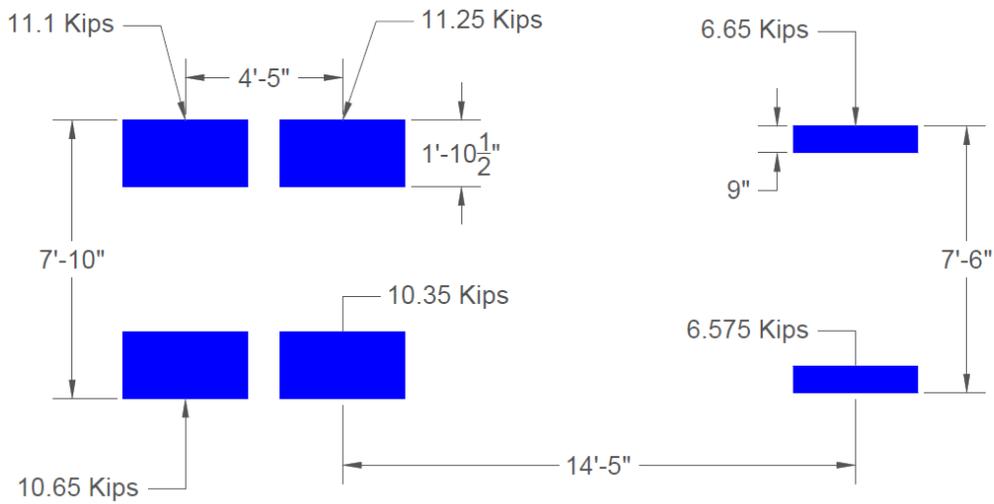


Figure E1 – Truck weight and dimensions

NOTE: ALL OF THE TRUCK POSITIONS ARE MEASURED FROM THE INSIDE FACE OF THE DOWNSTREAM CURB TO THE MIDDLE OF THE DRIVERS SIDE REAR DUALS.

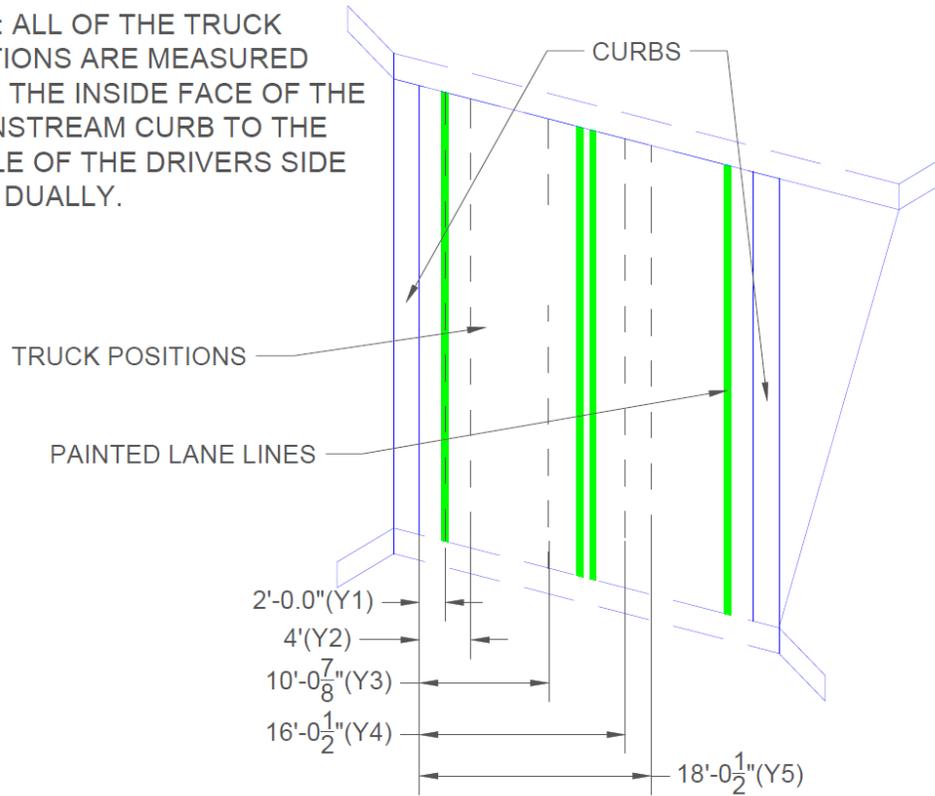


Figure E2 – Truck lane position during each test (Y1-Y5)

INSTRUMENTATION

Twenty-four strain transducers were used to collect strain data during the Albion bridge. They were grouped with three gages across the height of the cross section on the bottom of the T-beam and deck and at 22 inches up the height of the web. Individual gage locations can be seen in Figure E3. Gages were used with 21 inch extensions as seen previously in Figure B3.

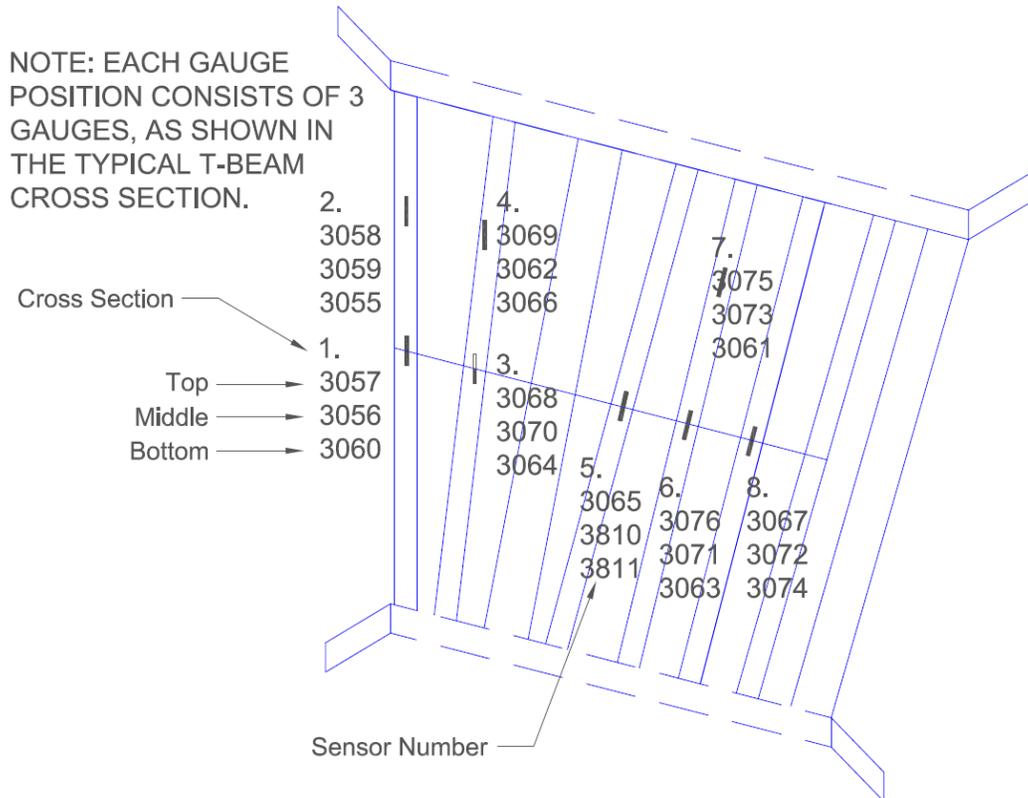


Figure E3 – Gage positions during Albion bridge test

REPRESENTATIVE DATA PLOTS

A representative plot of strain versus front tire position on the bridge is given in Figure E4. Similar plots are available for all other gages.

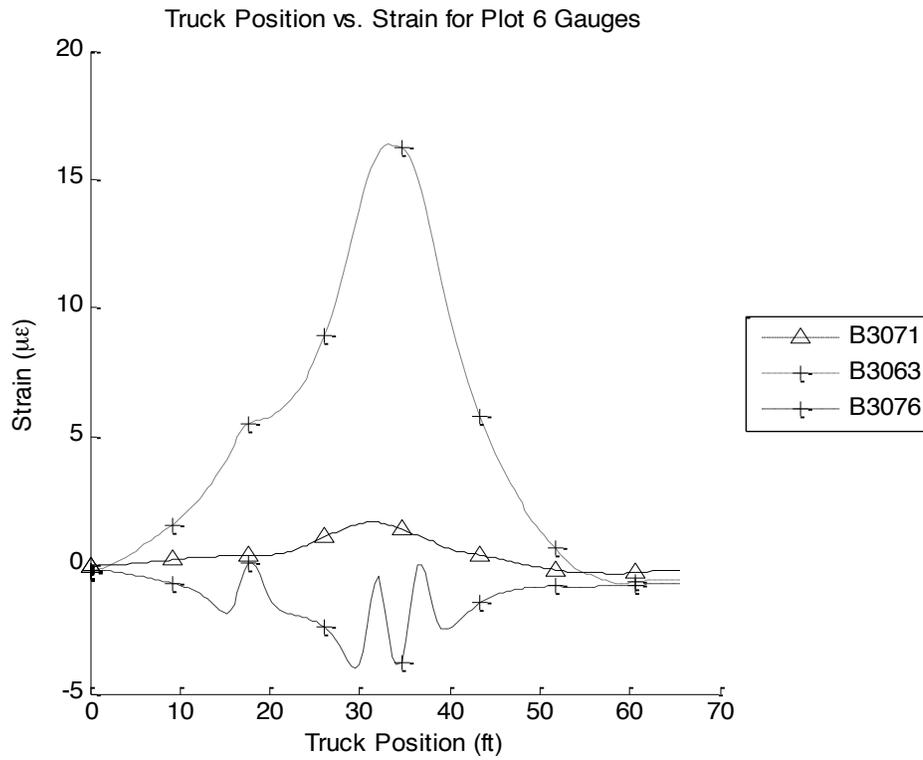


Figure E4 – Strain versus front tire position for Cross section 6