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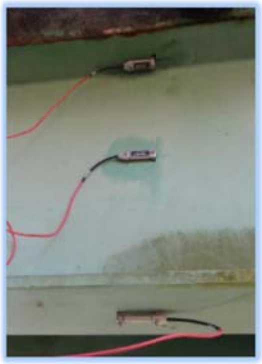

Technical Report 14-11

Advanced Bridge Safety Initiative

Live Load Testing and Load Rating of the Evans Brook Bridge (#5506) and the Hastings Bridge (#5507) in Batchelders Grant, Maine

January 2014

Technical Report Documentation Page

1. Report No.
ME 14-11

2.

3. Recipient's Accession No.

4. Title and Subtitle
Live Load Testing and Load Rating of the Evans Brook Bridge (#5506) and the Hastings Bridge (#5507) in Batchelders Grant, Maine

5. Report Date
January 2014

6.

7. Author(s)
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8. Performing Organization Report No.
14-13-979 MDOT

9. Performing Organization Name and Address
Advanced Structures and Composites Center
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35 Flagstaff Rd.
Orono, ME 04469

10. Project/Task/Work Unit No.
Project 017666.00

11. Contract © or Grant (G) No.
Contract # 20100506*5729

12. Sponsoring Organization Name and Address
Maine Department of Transportation

13. Type of Report and Period Covered

14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract (Limit 200 words)

The Advanced Structures and Composites Center at the University of Maine (UMaine) performed live load testing and rating factor analysis for two bridges (No. 5506 and No. 5507) in Batchelders Grant, Maine. The bridge load rating performed by consultants to the Maine Department of Transportation (DOT) indicated that the girders are not sufficient for carrying the legal loads for these bridges. Each bridge is built with four rolled steel girders (W36x170) with a variable depth concrete deck. Live load testing was conducted on October 17 and 18, 2013 in collaboration with Maine DOT personnel to evaluate the performance of the girder with potential gains expected due to unintended composite action.

The strain measurements were consistent, and the results appear reliable. Measured strains clearly indicated partial composite action between the girder and slab for Bridge No. 5506 and full composite action for Bridge No. 5507. Given that partial composite action was observed in Bridge No. 5506 and the flange is not partially embedded, the interior girder HL-93 operating rating factor for flexure can only be increased from 0.56 to 0.88 based on measured strains. However, the interior girder moment rating factor for Bridge No. 5507 can be increased to 0.92 based on analysis alone if the MaineDOT agrees that the girder will be able to develop its noncomposite plastic moment capacity given its partial embedment in the slab. The measured strains indicate that other sources of structural stiffness and capacity may justify a further increase in HL-93 operating rating factor to 1.18 for interior girder flexure in Bridge No. 5507.

17. Document Analysis/Descriptors
Bridge load rating, live load testing, girder and slab composite action

18. Availability Statement

19. Security Class (this report)

20. Security Class (this page)

21. No. of Pages
87

22. Price

1865 THE UNIVERSITY OF MAINE

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Live Load Testing and Load Rating of the Evans Brook Bridge (No. 5506) and Hastings Bridge (No. 5507) in Batchelders Grant, Maine

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Report Number: 14-13-979 MDOT

January 31, 2014

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Introduction

The Advanced Structures and Composites Center at the University of Maine (UMaine) performed live load testing and rating factor analysis for two bridges in Batchelders Grant, Maine. The bridge load rating performed by consultants to the Maine Department of Transportation (DOT) indicated that the girders are not sufficient for carrying the legal loads for these bridges. Each bridge is built with four rolled steel girders (W36x170) with a variable depth concrete deck. Live load testing was conducted on October 17 and 18, 2013 in collaboration with Maine DOT personnel to evaluate the performance of the girder with potential gains expected due to unintended composite action.

Test Setup

Each girder was instrumented with strain gages at its nominal mid-span with offsets away from diaphragms. Three of the four girders were also instrumented at the approximate quarter points of the bridge. One interior girder was also instrumented near the abutment to investigate end fixity. The Bridge Diagnostics Incorporated (BDI) Wireless Structural Testing System (STS WiFi) (Equipment number AEWC 1069) was used to instrument and collect the load position and strain data. A typical cross section can be seen in Figure 1 and Figure 2 where three gages are mounted on the bottom, near mid-height and near the top of the steel member. Detailed locations can be seen in Figure 3 and Figure 4. The skew on bridge 5507 was not measured and is not shown in Figure 4.

Bridge 5507 also has a flat concrete slab approach span that was instrumented but no analysis was completed for this report. Two gages were placed at mid-span on the underside of the slab and parallel to traffic as shown in Figure 5.

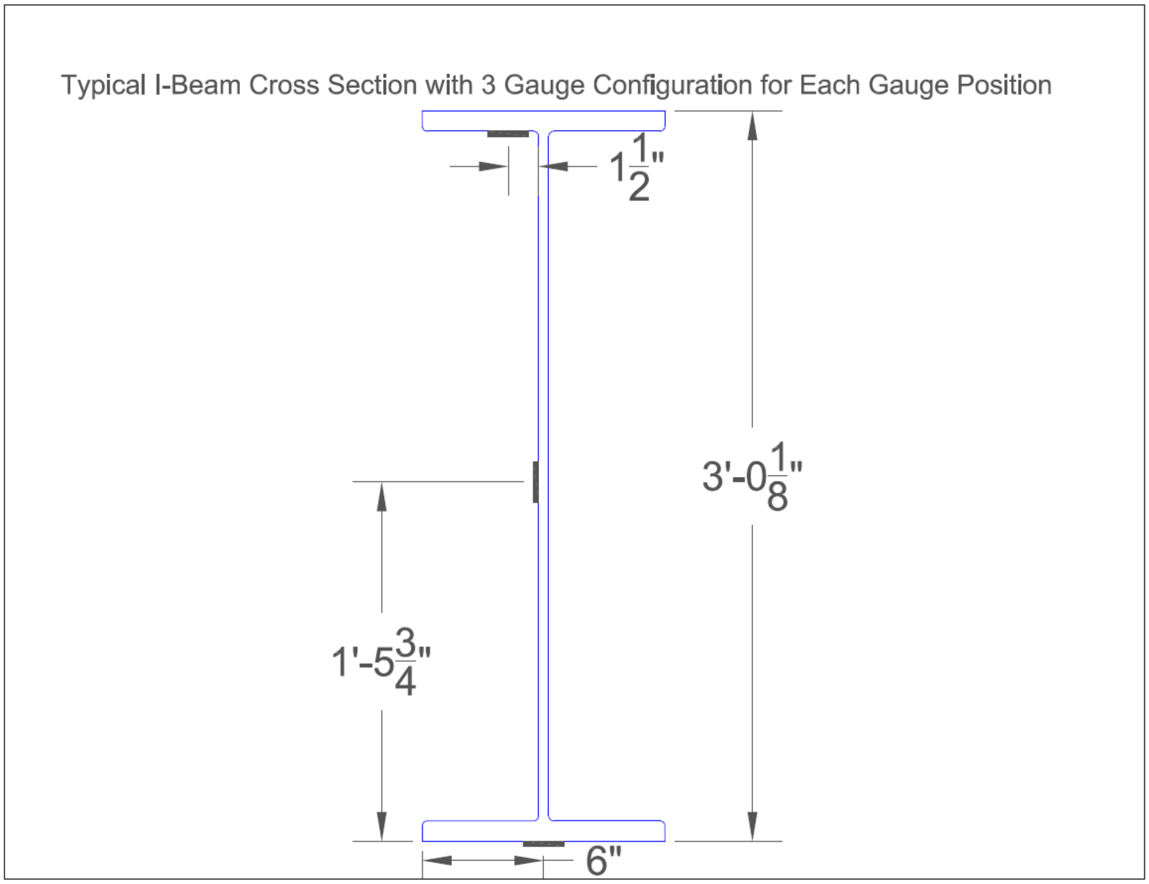


Figure 1 - Typical cross section with 3 strain gages and nodes.

Figure 2 - Cross-section view of interior girder.

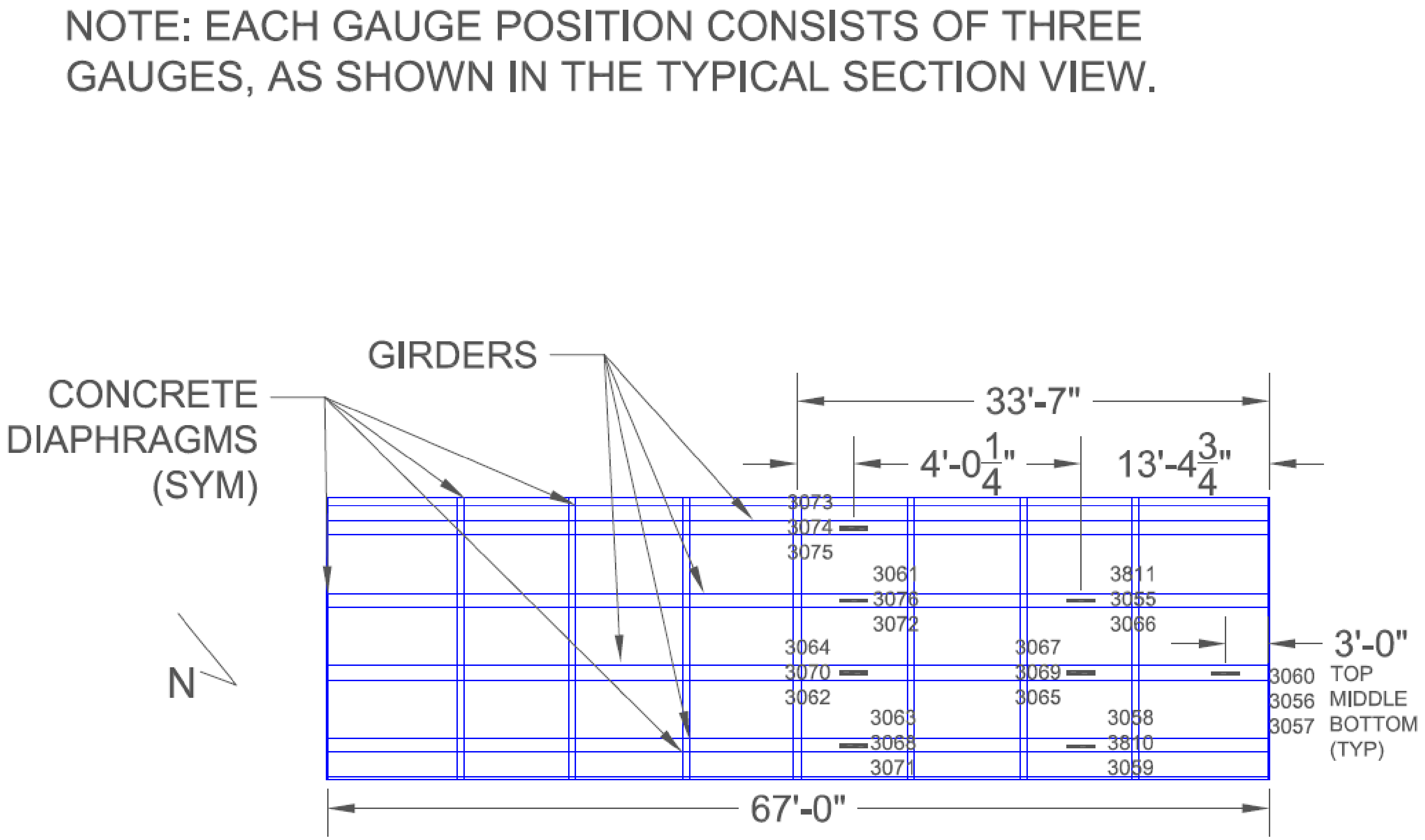


Figure 3 - Instrumentation plan view for Bridge No. 5506.

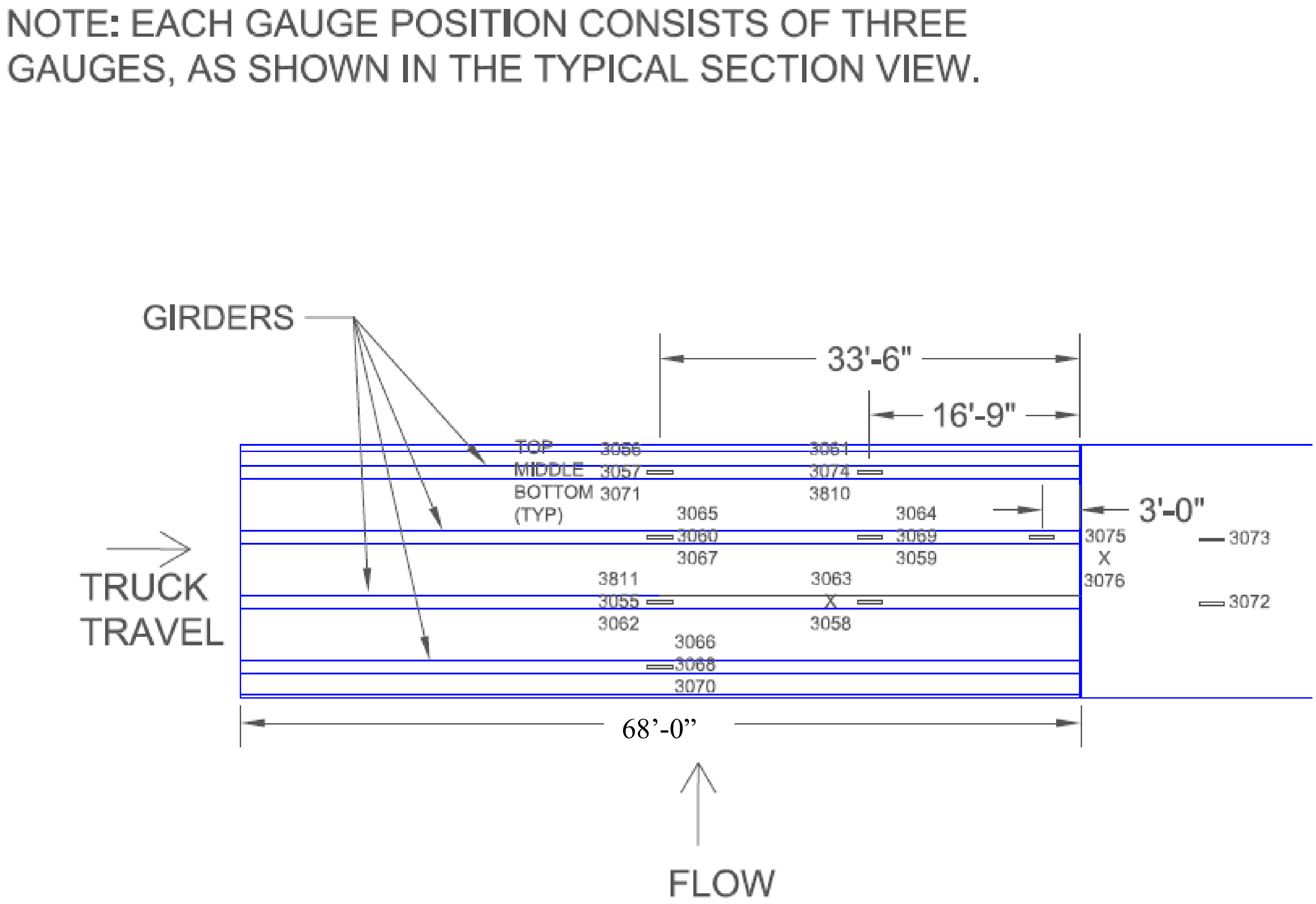


Figure 4 - Instrumentation plan view for Bridge No. 5507.

Figure 5 - One of two gages with extension on approach slab.

Test Vehicles

Two loaded tandem rear axle dump trucks provided by the DOT were used as test vehicles. Their total weights as measured by DOT scales and personnel were 57,600 lbs. (Truck 01-248) and 55,050 lbs. (Truck 01-853). The wheel line and axle spacings were measured on site for truck 01-248. Truck 01-248 was outfitted with the BDI Autoclicker to measure load position. The wheel circumference was also measured for this truck to be 10.90ft.

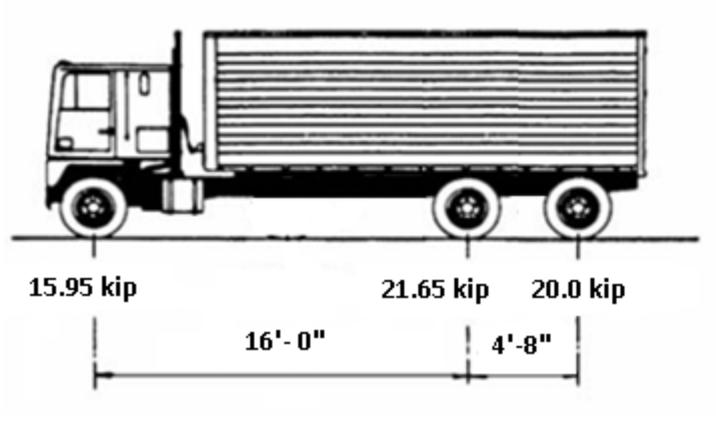


Figure 6 - Axle weights and spacings for Truck 01-248.

Testing

Seven tests were conducted at each bridge with either one or two test vehicles. Tests one, two and three were conducted with one truck in three different lane positions across each bridge. Test four used both trucks side-by-side traveling across the bridge as seen in Figure 11. During tests five, six and seven the trucks were in series and chained together as seen in Figure 12. At bridge 5507 the trucks traveled southerly during the test across the main span then concrete slab approach span. The trucks traveled northerly on bridge 5506 during the test.

Truck 01-248 was used for all single truck tests and was outfitted with the AutoClicker. This truck was also in the lead for tests 5, 6, and 7. During test 4 truck 01-248 was on the left when facing in the direction of travel. Figure 7, Figure 8 and Figure 9 show the wheel line alignment for each of the first 3 tests for a single truck as well as for tests 5, 6, and 7 with two trucks in series. Figure 10 shows the load lines for test 4.

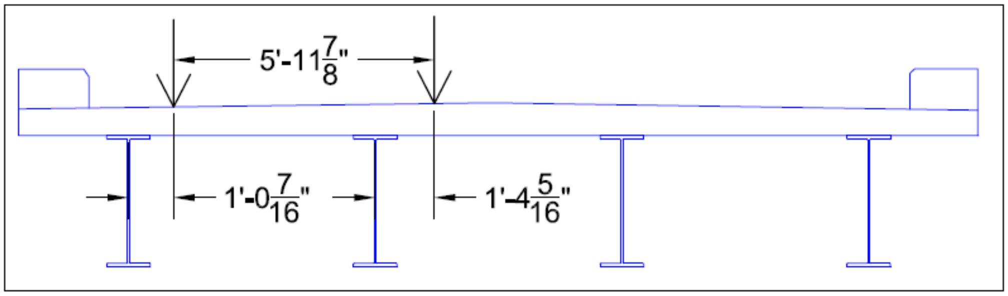


Figure 7 - Test 1 load configuration.

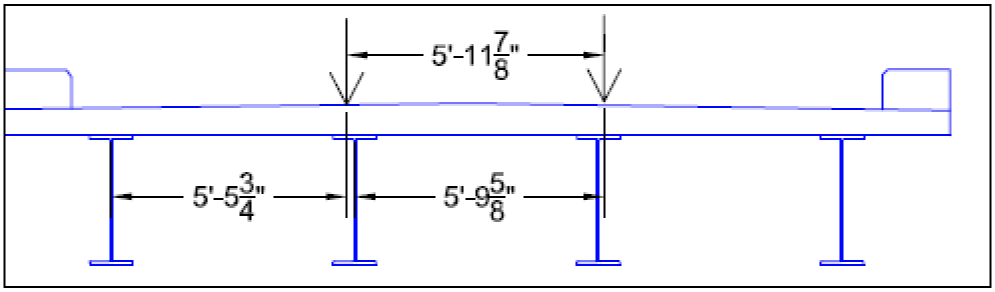


Figure 8 - Test 2 load configuration.

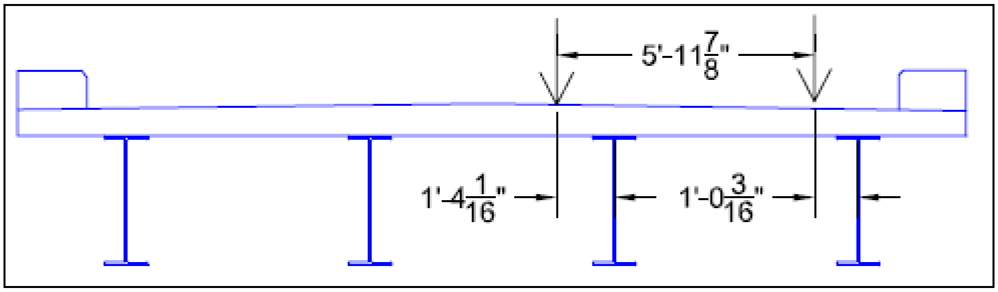


Figure 9 - Test 3 load configuration.

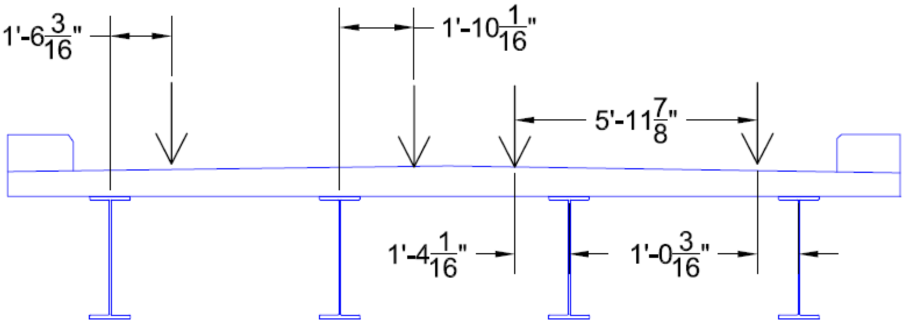


Figure 10 - Cross section with wheel line locations during Test 4.

Figure 11 - Trucks side by side for Test 4.

Figure 12 - Trucks chained together for Tests 5, 6, and 7.

Results

Peak strain values and graphs are presented here for Test 4 where data was used for the load rating analysis. Additional tables and graphs for all tests are included in Appendix A and Appendix C. Strains for mid-height and top flange in calculations may be negligibly different than those in Table 1 and Table 2. The values for those gages corresponding to the peak tensile strain were used in the calculations, though overall peak strains are reported in Table 1 and Table 2.

Table 1- Peak Strain (microstrain) Values for Test 4 (Bridge 5506)

	B3067	**B3057**	**B3056**	**B3060**	**B3070**	**B3072**	**B3076**	**B3061**
Max	0.29	5.09	1.60	2.83	50.13	115.08	38.66	1.10
Min	-24.52	-19.83	-8.84	-6.17	0.00	0.02	-0.07	-32.74
	B3069	**B3066**	**B3055**	**B3811**	**B3062**	**B3063**	**B3068**	**B3071**
Max	15.07	78.42	21.87	0.32	116.27	0.07	59.24	120.25
Min	-0.07	-0.04	0.01	-40.32	-0.16	-14.90	-0.21	-0.06
	B3065	**B3059**	**B3810**	**B3058**	**B3064**	**B3075**	**B3074**	**B3073**
Max	74.54	74.26	6.40	0.01	1.30	101.27	35.34	0.01
Min	-0.02	-0.17	-9.22	-54.19	-26.15	-0.17	-0.06	-36.68

Table 2- Peak Strain (microstrain) Values for Test 4 (Bridge 5507)

	B3811	**B3070**	**B3068**	**B3066**	**B3058**	**B3059**	**B3069**	**B3064**
Max	-0.01	119.46	46.35	0.10	80.63	82.14	24.84	-0.05
Min	-21.48	-0.14	-0.30	-14.57	-0.50	-0.39	-0.62	-41.01
	B3063	**B3810**	**B3074**	**B3061**	**B3062**	**B3071**	**B3057**	**B3056**
Max	0.20	108.68	46.81	1.85	124.70	132.55	62.12	0.41
Min	-42.36	-0.31	-0.38	-8.76	-0.08	-0.15	-0.08	-4.55
	B3072	**B3073**	**B3075**	**B3076**	**B3055**	**B3067**	**B3060**	**B3065**
Max	167.89	161.02	45.11	9.31	45.34	119.71	47.07	0.13
Min	-32.53	-46.33	-3.84	-39.32	-1.07	-0.22	-2.38	-26.87

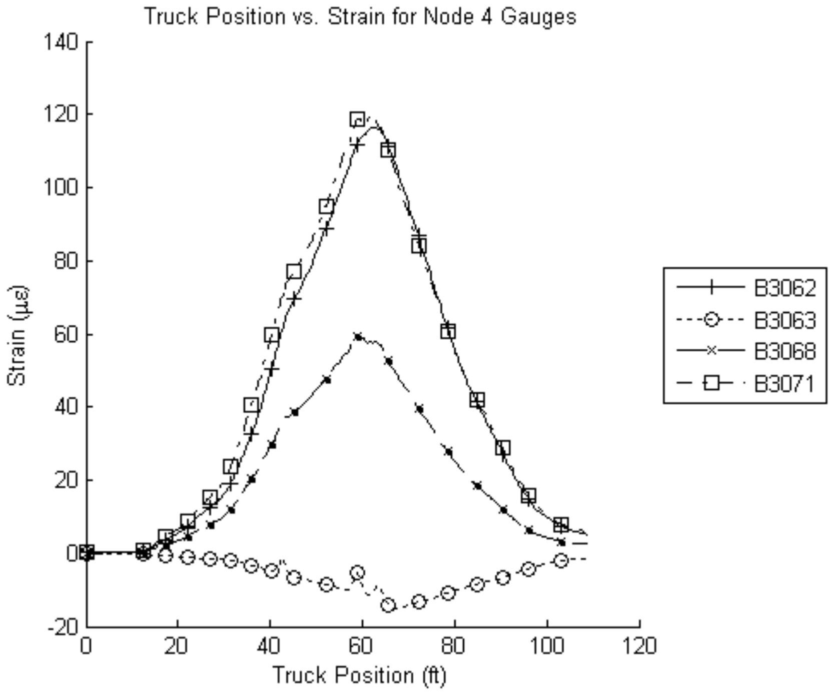


Figure 13 - Selected plots for bridge 5506 Test 4 with peak strain.

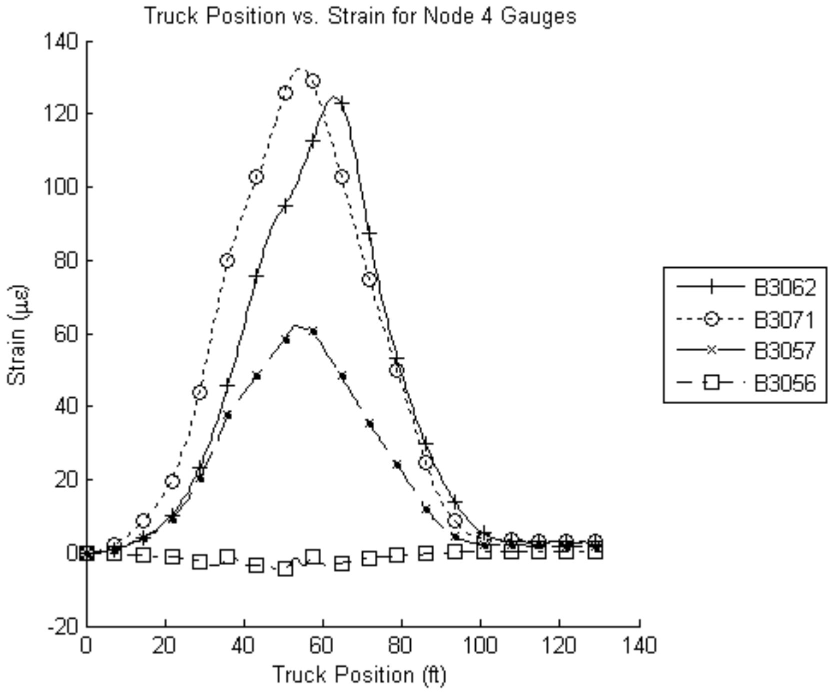


Figure 14 - Selected plots for bridge 5507 Test 4 with peak strains.

Analysis of Strain Data

Calculations in Appendix B for the critical load case with two trucks on each structure compare expected and measured response of an interior girder in flexure for each structure. Salient results are summarized individually for each structure below, since observed response for the two structures was different. The analysis focused on flexure of interior girders, which gave the smallest rating HL-93 operating rating factor for both structures.

Bridge 5506

The maximum measured flexural strain in the critical interior girder on Bridge No. 5506 was 116.3 με. The measured strains indicated partial composite action in all girders, with top flange negative strains as large as 34.3 με. Further, observations made during the test showed that the deck was originally cast on top of the girder top flange, and that there are regions where there are significant gaps between the deck and the girders. The maximum computed strain was 199 με for the same loading. This computed strain reflects the partial composite action observed in the field test as recommended by the AASHTO *Manual for Bridge Evaluation* and the NCHRP *Research Results Digest No. 234*. Given that the test trucks produced a load effect between 40% and 70% of the target HL-93 tandem plus lane (i.e. $0.4 < T/W < 0.7$ where T is the test vehicle load effect and W is the rating vehicle load effect), the AASHTO *Manual for Bridge Evaluation* suggests an increase in rating factor computed as below.

$$K = 1 + \left(\frac{\varepsilon_c}{\varepsilon_t} - 1 \right) K_b = 1 + \left(\frac{199}{116.3} - 1 \right) 0.8 = 1.57$$

The K of 1.57 reflects sources of structural capacity other than composite action such as less-conservative load distribution than predicted by AASHTO, contributions from the integral curbs, etc. Applying this value of K to the HL-93 operating level rating factor of 0.56 reported for an interior girder in the 2012 rating report issued by the Louis Berger Group brings the minimum rating factor up to 0.88. While still less than one, this represents a significant improvement in calculated capacity.

Bridge 5507

As with Bridge No. 5506, measured strains in Bridge No. 5507 were significantly less than those predicted using the AASHTO *Manual for Bridge Evaluation*. However, unlike Bridge No. 5506, strain data for all girders in Bridge No. 5507 indicated full composite action, and the neutral axis location inferred from measured strains agreed very well with the computed neutral axis location in all girders. In addition, field observation showed the girders to be partially encased in the deck, which extends to the bottom of the top flange of each girder. One other observation inferred from strains measured near a support is that the girders exhibited partial rotational fixity at this support. While this partial fixity tends to reduce the maximum positive moment and peak girder strain, it cannot be relied upon at higher loads. Therefore, measured mid-span strains were increased by 26.8 με based on strains measured near the support to remove the effect of partial rotational fixity as detailed in the calculations in Appendix B.

Given that $0.4 < T/W < 0.7$, and increasing measured strains to remove the effect of partial fixity, the rating factor modifier K can be computed as below per the AASHTO *Manual for Bridge Evaluation*:

$$K = 1 + \left(\frac{\varepsilon_c}{\varepsilon_t} - 1 \right) K_b = 1 + \left(\frac{206}{125 + 26.8} - 1 \right) 0.8 = 1.29$$

The AASHTO *Manual for Bridge Evaluation* recommends that composite action be extrapolated to $1.33W$ if composite action is to be relied upon in strength calculations. Based on our analysis of shear flow at the interface between the girder top flange and concrete deck, we do not believe that composite action can be relied upon at $1.33W$. However, given that the flange is partially embedded in the slab and the compression flange is fully laterally braced at all load levels, the relatively stocky rolled girder will be able to develop its full plastic moment capacity M_p. Based on our review of the Louis Berger Group load rating calculations, the RF of 0.70 for interior girder moment is consistent with the section developing only its yield moment M_y. If the section develops M_p, the rating factor increases to 0.92. It is justifiable to increase this rating factor of 0.92 by 1.29, giving an HL-93 operating factor of 1.18 for interior girder flexure.

Conclusions

The strain measurements were consistent, and the results appear reliable. Measured strains clearly indicated partial composite action between the girder and slab for Bridge No. 5506 and full composite action for Bridge No. 5507.

Given that partial composite action was observed in Bridge No. 5506 and the flange is not partially embedded, the interior girder HL-93 operating rating factor for flexure can only be increased from 0.56 to 0.88 based on measured strains. However, the interior girder moment rating factor for Bridge No. 5507 can be increased to 0.92 based on analysis alone if the MaineDOT agrees that the girder will be able to develop its non-composite plastic moment capacity given its partial embedment in the slab. The measured strains indicate that other sources of structural stiffness and capacity may justify a further increase in HL-93 operating rating factor to 1.18 for interior girder flexure in Bridge No. 5507.

References

1. *Manual for Bridge Evaluation 2nd Edition*. AASHTO. Washington D.C. 2011.
2. *LRFD Bridge Design Specifications*. AASHTO. Washington D.C. 5th Edition. 2010.
3. *Bridge Load Rating for Bridge #5506 Dated June 1, 2012*. Louis Berger Group. Provided by James Foster. P.E. Maine DOT.
4. *Bridge Load Rating for Bridge #5507 Dated July 6, 2012*. Louis Berger Group. Provided by James Foster. P.E. Maine DOT.

Appendix A – Summary Tables of Peak Strains for all Tests

Table 3 – Peak Strains for Test 1 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	0.20	24.00	12.15	6.34	24.98	45.64	12.05	-3.27
Minimum Strain	-5.63	1.53	3.65	3.91	-2.58	-1.57	-1.77	-30.80
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	-1.84	84.54	41.32	2.41	57.05	100.20	47.92	-3.45
Minimum Strain	-18.77	0.40	4.99	-6.27	6.60	0.33	1.90	-11.34
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	8.32	15.82	28.67	4.71	21.45	72.14	37.82	-2.06
Minimum Strain	-1.61	-2.48	-0.78	-26.95	2.28	-4.05	6.21	-19.68

Table 4 - Peak Strains for Test 2 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	0.03	54.94	20.56	0.49	48.87	45.66	13.56	-0.02
Minimum Strain	-18.00	-0.04	-0.04	-5.62	-0.31	-0.42	-0.40	-23.07
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	0.14	43.08	18.74	0.07	72.36	55.86	26.14	0.53
Minimum Strain	-29.15	-0.38	-0.08	-3.03	-0.10	-0.01	-0.04	-2.34
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	14.93	13.47	30.68	4.77	26.04	65.96	26.89	0.59
Minimum Strain	-1.27	-2.16	-2.08	-24.17	-1.46	-0.04	-0.17	-16.24

Table 5 - Peak Strains for Test 3 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	0.07	93.90	36.66	0.13	54.86	33.88	10.76	0.11
Minimum Strain	-16.38	-0.20	-0.14	-12.02	-0.60	-0.18	-0.27	-14.58
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	0.18	16.14	6.37	0.08	77.97	23.73	11.14	0.07
Minimum Strain	-29.76	-0.79	-1.21	-1.35	-0.21	-0.56	-1.08	-0.90
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	14.36	6.64	20.16	1.29	28.48	45.39	19.80	0.17
Minimum Strain	-2.05	-2.54	0.03	-19.17	-2.01	-0.68	-0.05	-5.20

Table 6 - Peak Strains for Test 5 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	0.18	31.27	11.82	0.76	32.76	60.54	15.42	0.53
Minimum Strain	-5.57	-1.53	-0.62	-2.93	-0.54	-0.38	-0.22	-34.80
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	0.14	117.94	48.65	-0.03	66.27	123.46	57.30	0.02
Minimum Strain	-20.16	-0.35	-0.61	-12.13	-0.26	-0.25	-0.07	-5.77
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	6.95	14.30	43.70	3.86	24.88	93.16	36.18	-0.04
Minimum Strain	-2.79	-4.73	-1.04	-33.67	-0.18	-0.03	-1.43	-18.82

Table 7 - Peak Strains for Test 6 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	0.02	76.28	28.48	0.64	57.98	58.71	14.93	0.66
Minimum Strain	-20.44	-0.18	-0.19	-8.14	-0.36	-0.24	-0.37	-31.48
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	-0.01	63.11	27.56	0.86	87.97	75.30	36.02	1.21
Minimum Strain	-36.16	-0.16	-0.06	-3.79	0.00	0.06	-0.69	-4.95
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	15.06	13.35	42.41	6.02	30.58	76.71	29.89	0.24
Minimum Strain	-2.01	-3.26	-2.33	-29.14	-1.73	-0.03	-0.31	-18.58

Table 8 - Peak Strains for Test 7 Bridge 5507 (microstrain)

Gage Numbers	**3811**	**3070**	**3068**	**3066**	**3058**	**3059**	**3069**	**3064**
Max Strain	-0.02	118.79	46.38	-0.02	67.58	45.19	12.60	0.03
Minimum Strain	-20.11	-0.69	0.04	-16.06	-1.09	-0.50	-0.35	-23.76
Gage Numbers	**3063**	**3810**	**3074**	**3061**	**3062**	**3071**	**3057**	**3056**
Max Strain	0.02	24.82	10.30	0.05	92.35	36.44	17.30	0.03
Minimum Strain	-38.14	-1.64	-1.86	-1.84	-0.32	-1.76	-1.81	-1.44
Gage Numbers	**3072**	**3073**	**3075**	**3076**	**3055**	**3067**	**3060**	**3065**
Max Strain	13.53	5.48	28.89	1.87	31.91	53.95	23.78	-0.01
Minimum Strain	-3.88	-4.57	-1.51	-23.46	-3.78	-1.43	-0.48	-6.37

Evans Brook Bridge (No. 5506)

Table 9 - Peak Strains for Test 1 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	0.02	2.56	1.12	1.62	35.22	44.86	17.35	1.27
Minimum Strain	-16.17	-16.84	-8.37	-4.23	-0.11	0.04	0.05	-5.96
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	9.20	30.86	10.23	0.30	75.93	0.44	54.09	107.11
Minimum Strain	-0.09	-0.11	-0.12	-12.18	0.05	-11.17	-0.16	0.03
Gage Numbers	**3065**	**3059**	**3810**	**3058**	**3064**	**3075**	**3074**	**3073**
Max Strain	49.99	68.91	9.13	1.75	1.77	14.58	5.90	0.12
Minimum Strain	-0.03	-0.09	-4.30	-43.70	-13.32	-0.08	0.05	-2.56

Table 10 - Peak Strains for Test 2 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	1.12	3.70	1.31	1.56	27.49	59.54	22.31	8.52
Minimum Strain	-9.53	-11.74	-5.03	-3.41	0.06	-0.08	0.00	-12.80
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	7.70	43.25	13.57	2.05	59.74	1.01	27.03	52.74
Minimum Strain	0.06	-0.03	-0.02	-22.28	-0.02	-3.81	-0.06	-0.05
Gage Numbers	**3065**	**3059**	**3810**	**3058**	**3064**	**3075**	**3074**	**3073**
Max Strain	38.73	30.73	2.73	0.61	2.53	44.62	18.55	2.10
Minimum Strain	0.01	-0.74	-5.85	-22.97	-11.25	-0.47	0.00	-7.21

Table 11 - Peak Strains for Test 3 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	0.07	2.09	0.90	1.17	19.35	71.96	26.00	3.33
Minimum Strain	-5.32	-8.64	-3.09	-1.92	-0.46	-0.17	-0.16	-18.55
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	5.31	50.29	14.96	0.42	42.91	0.21	7.41	16.10
Minimum Strain	-0.79	-0.33	-0.15	-26.28	-0.12	-3.71	-0.58	-0.57
Gage Numbers	**3065**	**3059**	**3810**	**3058**	**3064**	**3075**	**3074**	**3073**
Max Strain	24.56	7.63	1.81	0.17	0.20	83.05	30.26	1.12
Minimum Strain	-0.05	-0.20	-0.68	-4.49	-6.70	-0.22	0.04	-27.80

Table 12 - Peak Strains for Test 5 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	0.53	3.68	1.13	1.74	36.46	50.32	18.84	1.24
Minimum Strain	-19.34	-18.42	-10.74	-4.70	-0.03	-0.11	-0.11	-9.48
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	9.56	34.18	10.83	0.48	81.17	1.08	54.87	111.05
Minimum Strain	0.00	-0.12	0.01	-15.67	-0.11	-12.72	-0.20	-0.08
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	9.56	34.18	10.83	0.48	81.17	1.08	54.87	111.05
Minimum Strain	0.00	-0.12	0.01	-15.67	-0.11	-12.72	-0.20	-0.08

Table 13 - Peak Strains for Test 6 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	1.75	3.55	1.28	2.21	29.15	72.46	25.17	5.27
Minimum Strain	-11.23	-14.38	-6.28	-3.08	0.01	-0.11	-0.77	-20.26
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	7.84	48.50	15.48	0.96	65.73	0.23	28.88	57.86
Minimum Strain	0.09	-0.17	-0.14	-24.25	-0.04	-5.28	-0.15	-0.24
Gage Numbers	**3065**	**3059**	**3810**	**3058**	**3064**	**3075**	**3074**	**3073**
Max Strain	40.71	29.32	2.50	0.34	0.25	60.71	23.51	0.12
Minimum Strain	0.03	-0.13	-6.01	-27.97	-13.43	-0.08	-0.17	-15.82

Table 14 - Peak Strains for Test 7 Bridge 5506 (microstrain)

Gage Numbers	**3067**	**3057**	**3056**	**3060**	**3070**	**3072**	**3076**	**3061**
Max Strain	0.01	2.06	0.67	1.16	24.37	81.90	29.39	4.75
Minimum Strain	-7.17	-10.24	-3.82	-2.31	-0.11	-0.07	-0.01	-20.58
Gage Numbers	**3069**	**3066**	**3055**	**3811**	**3062**	**3063**	**3068**	**3071**
Max Strain	6.21	56.48	16.82	-0.08	52.56	-0.08	12.81	26.08
Minimum Strain	0.03	-0.23	-0.12	-29.35	-0.07	-3.47	-0.23	-0.27
Gage Numbers	**3065**	**3059**	**3810**	**3058**	**3064**	**3075**	**3074**	**3073**
Max Strain	31.22	12.65	2.45	0.04	0.10	90.80	31.06	0.94
Minimum Strain	-0.10	-0.28	-0.62	-8.41	-8.23	-0.08	-0.15	-34.38

Appendix B – Rating Factor and Test Data Analysis

12/9/2013 1/

BATCHELDERS GRANT #5506

W. Davids

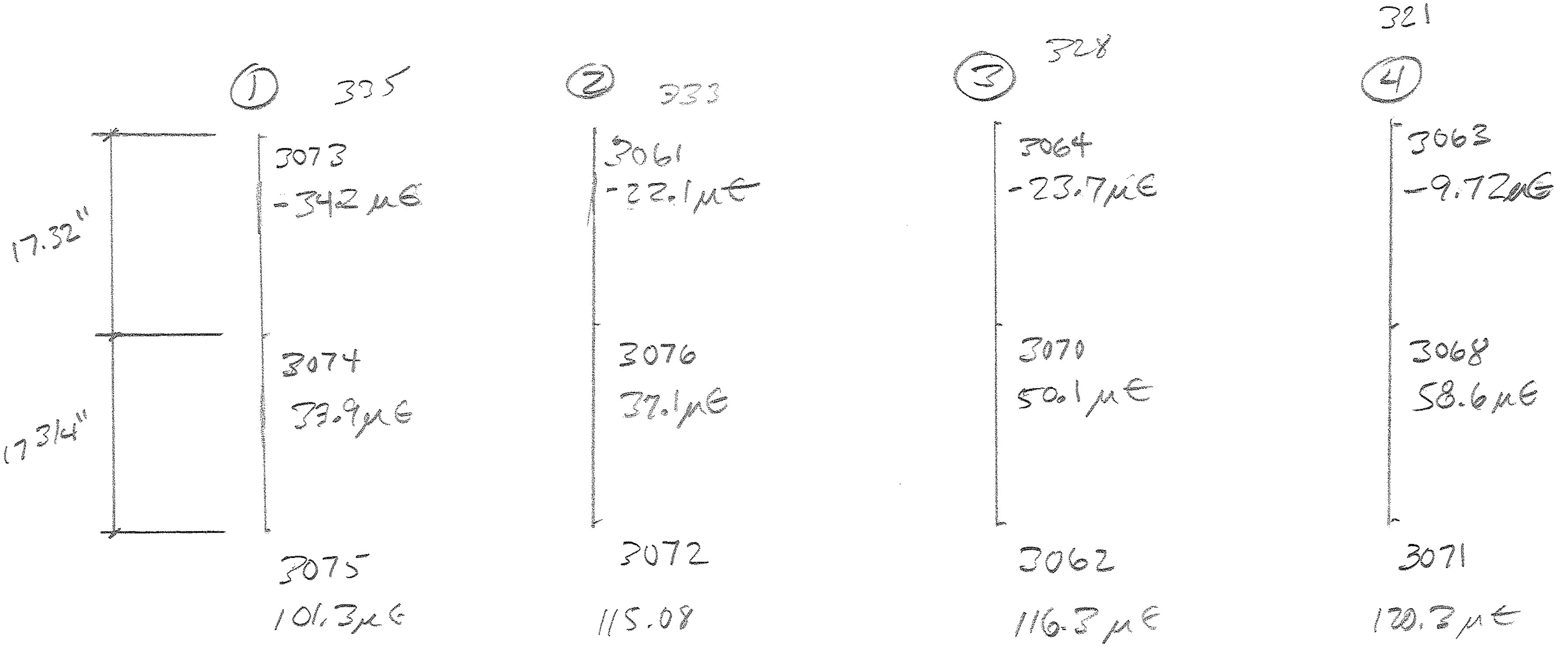
REF LBG Calcs

- Focus on interior girder, moment, 2 lanes loaded, Mg = 0.477, HL-93 operating rating = 0.56
- Peak measured strains are summarized on the next page
  - Indicate partial composite action
  - We can match extent of partial composite action assuming an effective slab width of 36.5" or effective thickness of 6.9" (vs. 10.2" actual avg).
- Will assume eff deck thickness = 6.9", since this conservatively produces the smallest $S = 762 in^3$
- Cannot rely on composite action @ higher loads, since flange is not embedded & gaps between slab & girder are present
- Will modify LBG-reported RF of 0.56, keep DF = 0.477

RATCHEDER'S GRANT - #5506

12/9/2013
W. DAVIDS 21

LOAD CASE Y4 - 2 TRUCKS



Interior $I = 26071 \text{ in}^4$, $\bar{y} = 31.1''$ (composite)

$\bar{y}$ estimated from field strains: (avg = 28.2" overall, 28.0" interior only)

① : $\left(\frac{101.3}{101.3+26.7}\right)35.07'' = 25.7''$; ② : $\left(\frac{115.08}{115.08+32.7}\right)35.07'' = 27.3''$

③ : $\left(\frac{116.3}{116.3+26.1}\right)35.07 = 28.6''$; ④ : $\left(\frac{120.3}{120.3+44.9}\right)35.07'' = 31.2''$

Girder Section Properties -- 5506, W36x170

exterior girder, slab = 9.17" at girder CL
interior girder, slab = 10.2" at girder CL

haunch = 0

component	width	thickness	modular ratio	transf. area	y	Area*y	I_bar	A*y^2
Slab	62	10.2	10	63.24	41.30	2612.1	548.3	6633.4
Top Bars	0	0	1	0.00	41.30	0.0	0.0	0.0
Top Flange	12.03	1.12	1	13.44	35.65	479.0	1.4	282.3
Web	33.97	0.68	1	23.10	18.10	418.1	2221.3	3880.0
Bot. Flange	12.03	1.12	1	13.44	0.56	7.5	1.4	12503.4
				113		3516.7	2772.4	23299.0

Note: flange thicknesses increases slightly to give correct I for non-composite section

y_bar = 31.1 in from bottom
Moment of Inertia = 26071 in^4

Girder Section Properties -- 5506, W36x170

exterior girder, slab = 9.17" at girder CL
interior girder, slab = 10.2" at girder CL

haunch = 0

component	width *	thickness	modular ratio	transf. area	y	Area*y	I_bar	A*y^2
Slab	36.5	10.2	10	37.23	41.30	1537.7	322.8	6582.1
Top Bars	0	0	1	0.00	41.30	0.0	0.0	0.0
Top Flange	12.03	1.12	1	13.44	35.65	479.0	1.4	783.9
Web	33.97	0.68	1	23.10	18.10	418.1	2221.3	2266.5
Bot. Flange	12.03	1.12	1	13.44	0.56	7.5	1.4	10124.5
				87		2442.4	2546.9	19757.0

Note: flange thicknesses increases slightly to give correct I for non-composite section

y_bar = 28.008 in from bottom
Moment of Inertia = 22304 in^4 } $S = 796 in^3$

* slab width reduced to give $\bar{y} = 28.0''$, as inferred from measured strains in interior girders

Girder Section Properties -- 5506, W36x170

exterior girder, slab = 9.17" at girder CL
interior girder, slab = 10.2" at girder CL

haunch =	0

component	width	* thickness	modular ratio	transf. area	y	Area*y	I_bar	A*y^2
Slab	62	6.9	10	42.78	39.65	1696.4	169.7	5768.2
Top Bars	0	0	1	0.00	39.65	0.0	0.0	0.0
Top Flange	12.03	1.12	1	13.44	35.65	479.0	1.4	776.8
Web	33.97	0.68	1	23.10	18.10	418.1	2221.3	2282.4
Bot. Flange	12.03	1.12	1	13.44	0.56	7.5	1.4	10150.0
				93		2601.0	2393.9	18977.5

Note: flange thicknesses increases slightly to give correct I for non-composite section

y_bar = 28.042 in from bottom
Moment of Inertia = 21371 in^4

$S = 762 \text{ in}^3$

* deck thickness reduced to produce $\bar{y} = 28.0''$

BATCHELDERS GRANT #5506

REVISE HL-93 OPERATING RF:

- Per AASHTO MBE & NCHRP Research Results Digest #234, RF adjustments should be made based on observed response, i.e. partial composite action in this case.

- This approach removes the effect of unreliable partial composite action, but allows other beneficial effects such as lateral load distribution & other load-carrying mechanisms to be accounted for

$K_b = 0.8$ since $0.4 < T/w < 0.7$

$K = 1 + K_b K_a$

per RISA analysis, $M_{truck} = 767.1$ ft-kips @ gage location (4' off ℄ span to avoid diaphragm)

$K_a = \frac{\epsilon_c}{\epsilon_T} - 1$; $\epsilon_c = \frac{0.477 \times 767.1 \times 12}{762 in^3 \times 29000 ksi} \times 10^6 = 199 \mu\epsilon$

min w/ partial comp. action

ϵ_T = measured strain = $116.3 \mu\epsilon$

$K_a = \frac{199}{116.3} - 1 = 0.71$

$K = 1 + 0.8 \times 0.71 = 1.568$

$RF = 1.568 \times 0.56 = 0.88 < 1$

12/06/2013 4

5507 - DEAD LOADS, INTERIOR GIRDER W. Davids

REF. LBG Rating

$W_{DC} = 616 + 299/4 + 154/4 + 170 = 899 \#/lf$

$W_{DW} = 208$

Deck: $\frac{8.76''}{12} \times 150 \times \frac{68''}{12} = 616$ plf

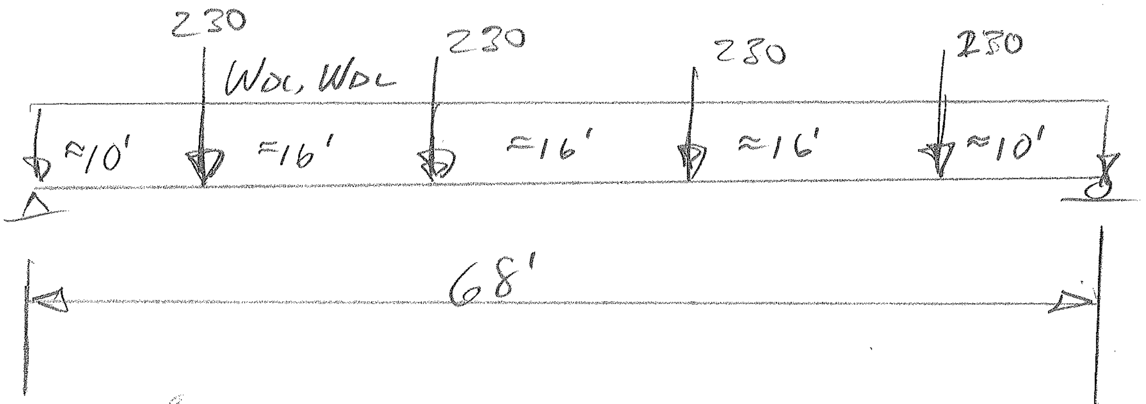
WS: $\frac{3.15''}{12} \times 140 \times \frac{68}{12} = 208$ plf

Brush curbs: $= 299$ plf (ea, 1 each side)

railings: $= 154$ plf (ea, 1 each side)

diaphragms: $= 230 \#$ ea, interior

ignore bump-outs, distribute railing evenly to all girders which is conservative for interior girders.



$$M_{DC} \simeq 0.899 \times 68^2/8 + \frac{2}{1000} \times \left(\frac{230 \times 26 \times 42}{68} \times \frac{34}{42} + \frac{230 \times 10 \times 58}{68} \times \frac{34}{58} \right)$$

$$= 520 + \frac{2}{1000} \times (2990 + 1150) = 528 \text{ ft-k}$$

$$M_{DW} = 0.208 \times 68^2/8 = 120 \text{ ft-k.}$$

5507 - SECTION PROPS

12/06/2013 21
W. Davids

Girder Section Properties -- 5507, W36x170

exterior girder, slab = 8.01" at girder CL
interior girder, slab = 8.76" at girder CL

haunch =	0							
component	width	thickness	modular ratio	transf. area	y	Area*y	I_bar	A*y^2
Slab	68.00	8.76	10	59.57	40.58	2417.5	380.9	6266.4
Top Bars	0	0	1	0.00	40.58	0.0	0.0	0.0
Top Flange	12.03	1.12	1	13.44	35.65	479.0	1.4	380.0
Web	33.97	0.68	1	23.10	18.10	418.1	2221.3	3452.5
Bot. Flange	12.03	1.12	1	13.44	0.56	7.5	1.4	11908.2
				110		3322.1	2605.1	22007.1

Note: flange thicknesses increased slightly to give correct I for non-composite section

y_bar =	30.3	in from bottom
Moment of Inertia =	24612	in^4

$S_{cmp} = 24612/30.3 = 812 \text{ in}^3$

$M_y = 33^{ksi} \times 812 = 26805 \text{ in·k} = 2234 \text{ ft·k}$ (composite)

$M_y = 33 \times 580 = 19140 \text{ in·k} = 1595 \text{ ft·k}$ (non-composite)

$M_p = 33 \times 668 = 22044 \text{ in·k} = 1837 \text{ ft·k}$ (non-composite)

1400/2010: p1

SJU7 – FLEXURAL RATING, INTERIOR GIRDER W. Davids

Non-composite

M_p, non-composite

$\phi M_r \simeq 1.0 \times M_p = 1.0 \times 22044$ in·k $= 1837$ ft·k

$M_{LL} = 1364 + 0.8 \times (1711 - 1364) = 1642$ (AASHTO BDM p. 6-69, includes IM)

$$R = \frac{1837 - 1.25 \times (528 + 120)}{1.35 \times 0.506 \times 1642} = \frac{1027}{1122} = 0.92 > 0.70$$

per LBG

$\llcorner$ LBG DF, 2 lanes,

However, if $\phi M_r = 1.0 M_y = 19140$ in·k $= 1595$ ft·k, the limiting capacity for a discretely braced, non-composite section is:

$$R = \frac{1595 - 1.25 \times (528 + 120)}{1122} = 0.70 = \text{LBG rating}$$

In reality, flange is laterally braced by the deck due to partial embedment; $R = 0.92$ based on my calcs is OK.

BATCHELDER'S GRANT - #5507

12/06/2013 4/
W Davids

ANALYZE MID-SPAN STRAINS

Max moment per truck = 800.3 ft-k (2 trucks, LC Y4)

Per 4BG, interior girder controls, $mg = 0.506$

Per AASHTO MBE, should base strain calcs on field-observed composite response.

$S = 24612 \, in^3 / 30.3^{in} = 812 \, in^3$

expected strains based on applied moments & composite section:

$$\frac{M}{SE} = \frac{0.506 \times 800.3 \overset{in}{\times} 12}{812 in^3 \times 29000} = 206 \mu\epsilon$$

Largest measured strains in interior girder (see next p.) $= 125 \mu\epsilon$

$K = 1 + K_a K_b$; $K_b = 0.8$ $(0.4 < T/W < 0.7)$

$$K_a = \frac{\epsilon_c}{\epsilon_T} - 1 = \frac{206}{125} - 1 = 0.648$$

$$K = 1 + \overset{K_b}{0.8} \times 0.648 = 1.52$$

2 ISSUES:

1) gauges @ 3' off spt indicate some $M^{(-)}$ @ spt affecting results.

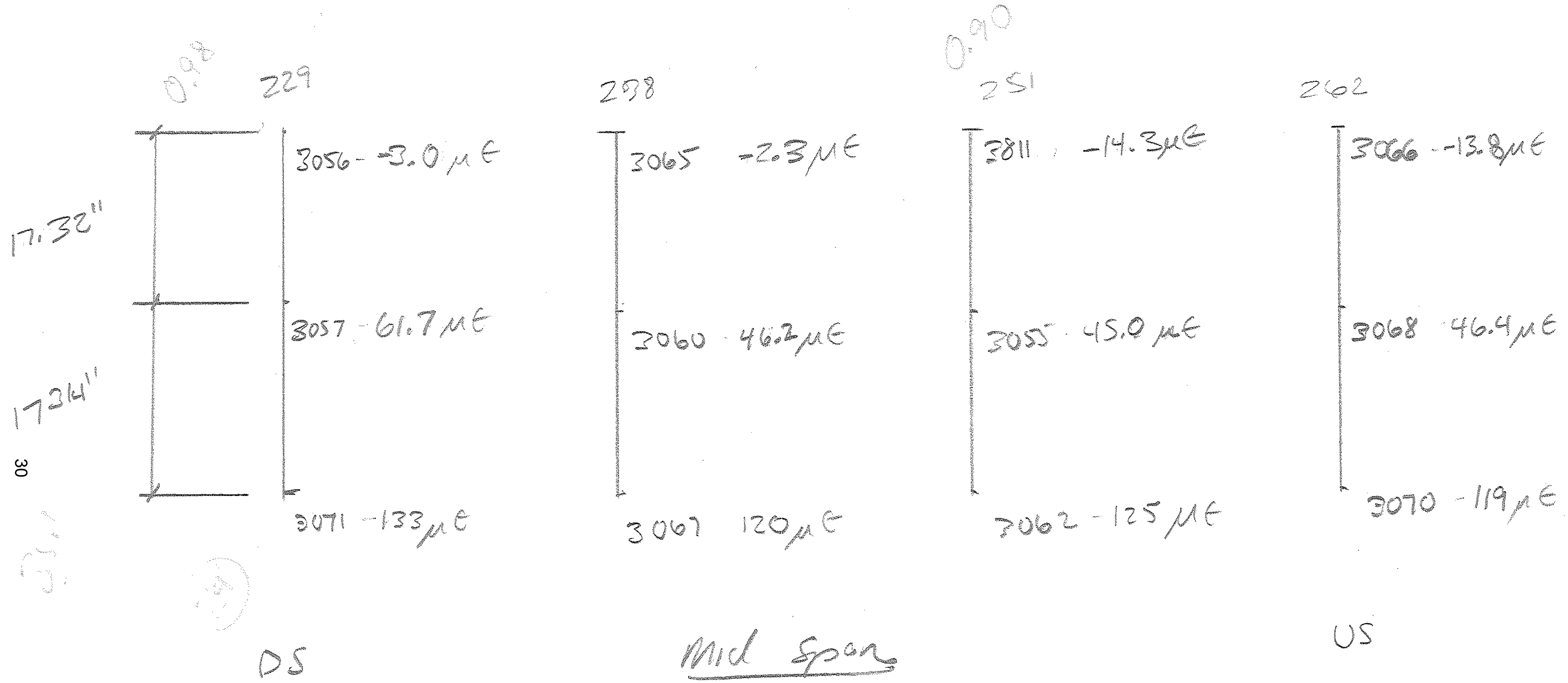
2) Must be able to extrapolate composite action to higher loads to scale up RF by K

BATCHELDER'S GRANT - #5501

12/06/2013
W. Davids

5/

LOAD CASE Y4 - 2 trucks



Mid Span

Interior girder composite $I = 25211\ in^4$, $\bar{y} = 30.7''$
Exterior girder composite $I = 23524\ in^4$, $\bar{y} = 29.6''$
} assumes $n = 10$ ($f'_c = 2.5$ ksi), does not include any top slab reinforcing.

Section is clearly behaving as composite

Non-composite $S_{xx} = 580 in^3$

BATCHELDER'S GRANT #5507

12/06/2013 8/
W. Davids

ANALYZE MID-SPAN STRAINS

CORRECT FOR $M^{(-)}$ @ SPRT

With truck positioned to produce max $M^{(+)}$ @ midspan, $-37.9\mu\epsilon$ & $37.5\mu\epsilon$ were observed @ 3076 (bottom) & 3075 (top) in DS interior girder @ 3' from Abutment. These strains indicate $M^{(-)}$ w/ a non-composite section. Non-composite response is expected here, since the slab will not have been mobilized @ the sprt.

We can compute then $M^{(-)} = \sigma S = \underbrace{\left(29000 \times \frac{37.9}{1000000}\right)}_{\sigma} \times \frac{580 in^3}{12} = 53.1 ft^k$

so, actual moment applied to girder @ mid-span is 53.1 ft-k too large, assuming same degree of fixity @ both abutments.

To account for this, we must increase measured strain,

increase ϵ_T: $\frac{M}{S_{comp} E} = \frac{53.1 \times 12}{821 \times 29000} \times (1 \times 10^6) = 26.8 \mu\epsilon$

$Ka = \frac{\epsilon_c}{\epsilon_T} - 1 = \frac{206}{125 + 26.8} - 1 = 0.36$

12/06/2013 7/
W. Davids

BATCHELDER'S GRANT # 5507

ANALYZE MID-SPAN STRAINS:

CORRECT FOR $M^{(-)}$ @ SPRT

We cannot rely on partial fixity @ the sprt under higher loads. Because of this, strains & moments will likely approach simple-span values w/ larger vehicles.

EXTRAPOLATE COMPOSITE ACTION TO HIGHER LOADS:

Per NCHRP report No. 224, an interfacial shear stress due to steel-concrete bond of up to 100 psi can be conservatively relied on given that the flange is embedded.

The maximum shear DF = 0.646 (ratings report, 2 lanes, interior girder).

The max shear = $\underline{51.0^k}$ per RISA (1-truck, simple span)

$V_{test} = 0.646 \times 51 = 32.9^k$

$\tau_{test} = \dfrac{VQ}{Ib}$ $Q = 630\ in^3$ (sprdsht - 1st moment of deck about NA)

$\tau_{test} = \dfrac{32.9 \times 630}{25211 \times 12.03''} = 0.068\ ksi = \underline{68.3\ psi}$

can extrapolate to truck wt W = $\dfrac{100}{68.3} \times \underset{\approx \text{test truck wt}}{56^K} = 82^K$ truck

Need to look @ shear: 19.2

$V_{HL\text{-}93} = 32^k + \dfrac{54}{68} \times 32 + \dfrac{40}{68} \times 8 + 0.64 \times 34' = 83.9^k \leftarrow$ No IM.

$\tau_{HL93} = \dfrac{83.9}{51} \times 68.3\ psi = 112.4\ psi > 100\ psi$ NG

BATCHELDER'S GRANT #5507

12/06/2013 8/
W. Davids

RF RECOMMENDATION

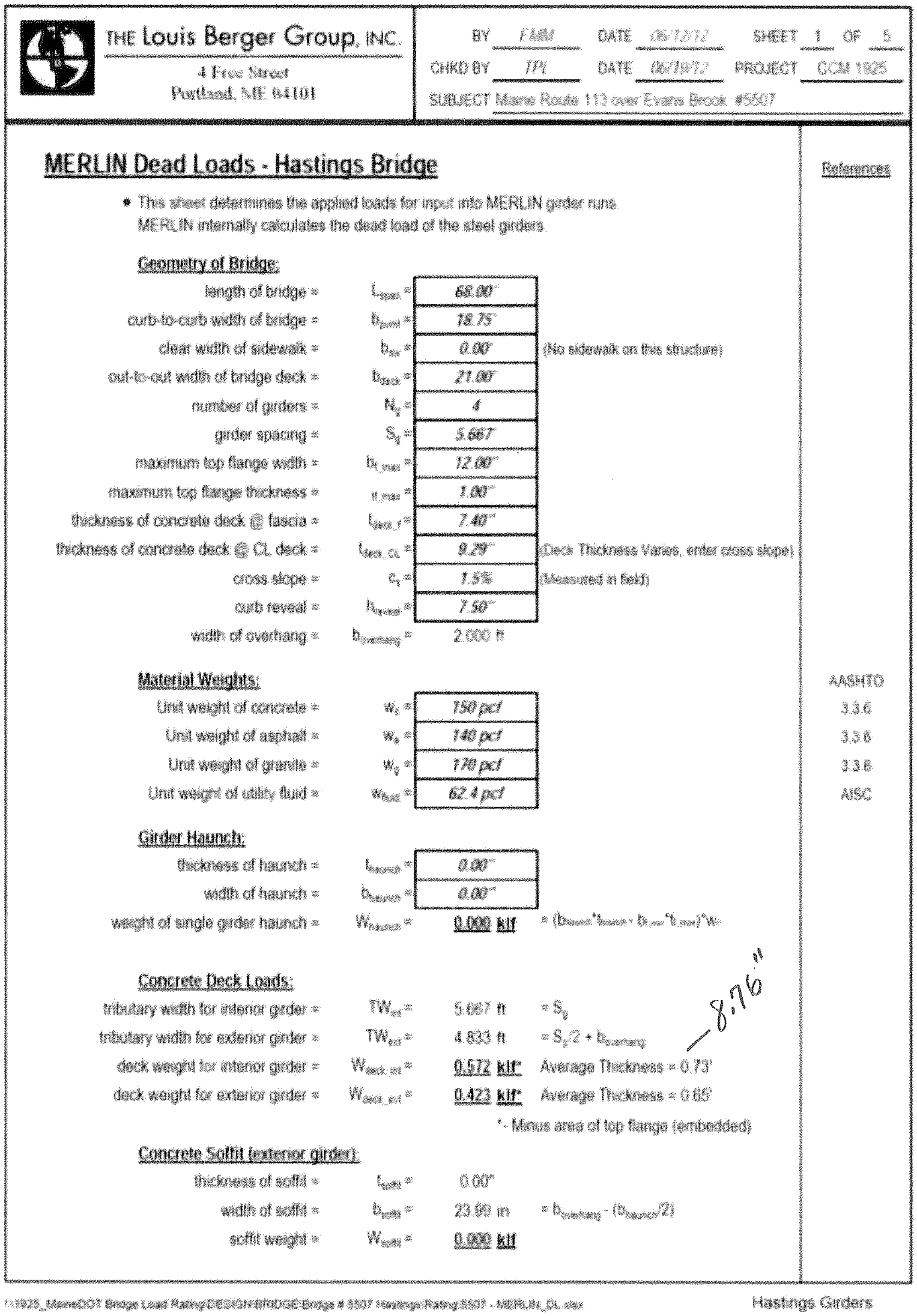
Based on shear analysis, limit Ka to $\frac{100}{112.4} \times 0.36 = 0.32$ if, we are relying on composite action.

However, RF = 0.92 based on developing Mp for non-composite section, which only requires lateral bracing from partial embedment. No need to knock down Ka for less than HL-93.

Kb

$$RF = 0.92 \times (1 + 0.8 \times 0.36) = 1.18 > 1.0$$

HL-93 operating OK



THE Louis Berger Group, INC.
4 Free Street
Portland, ME 04101

BY EMM DATE 06/12/12 SHEET 1 OF 5
CHKD BY TPI DATE 06/19/12 PROJECT CCM 1925
SUBJECT Maine Route 113 over Evans Brook #5507

MERLIN Dead Loads - Hastings Bridge

References

- This sheet determines the applied loads for input into MERLIN girder runs.
  MERLIN internally calculates the dead load of the steel girders.

Geometry of Bridge:

length of bridge =	L_{span} =	*68.00'*		
curb-to-curb width of bridge =	b_{pvmt} =	*18.75'*		
clear width of sidewalk =	b_{sw} =	*0.00'*	(No sidewalk on this structure)	
out-to-out width of bridge deck =	b_{deck} =	*21.00'*		
number of girders =	N_g =	*4*		
girder spacing =	S_g =	*5.667'*		
maximum top flange width =	$b_{t,max}$ =	*12.00"*		
maximum top flange thickness =	$t_{f,max}$ =	*1.00"*		
thickness of concrete deck @ fascia =	$t_{deck,f}$ =	*7.40"*		
thickness of concrete deck @ CL deck =	$t_{deck,CL}$ =	*9.29"*	(Deck Thickness Varies, enter cross slope)	
cross slope =	c_s =	*1.5%*	(Measured in field)	
curb reveal =	h_{reveal} =	*7.50"*		
width of overhang =	$b_{overhang}$ =	2.000 ft		

Material Weights:

			AASHTO
Unit weight of concrete =	w_c =	*150 pcf*	3.3.6
Unit weight of asphalt =	w_a =	*140 pcf*	3.3.6
Unit weight of granite =	w_g =	*170 pcf*	3.3.6
Unit weight of utility fluid =	w_{fluid} =	*62.4 pcf*	AISC

Girder Haunch:

thickness of haunch =	t_{haunch} =	*0.00"*	
width of haunch =	b_{haunch} =	*0.00"*	
weight of single girder haunch =	W_{haunch} =	**0.000 klf**	$= (b_{haunch}*t_{haunch} - b_{t,max}*t_{f,max})*w_c$

Concrete Deck Loads:

—8.76"

tributary width for interior girder =	TW_{int} =	5.667 ft	$= S_g$
tributary width for exterior girder =	TW_{ext} =	4.833 ft	$= S_g/2 + b_{overhang}$
deck weight for interior girder =	W_{deck_int} =	**0.572 klf***	Average Thickness = 0.73'
deck weight for exterior girder =	W_{deck_ext} =	**0.423 klf***	Average Thickness = 0.65'

*- Minus area of top flange (embedded)

Concrete Soffit (exterior girder):

thickness of soffit =	t_{soffit} =	0.00"	
width of soffit =	b_{soffit} =	23.99 in	$= b_{overhang} - (b_{haunch}/2)$
soffit weight =	W_{soffit} =	**0.000 klf**	

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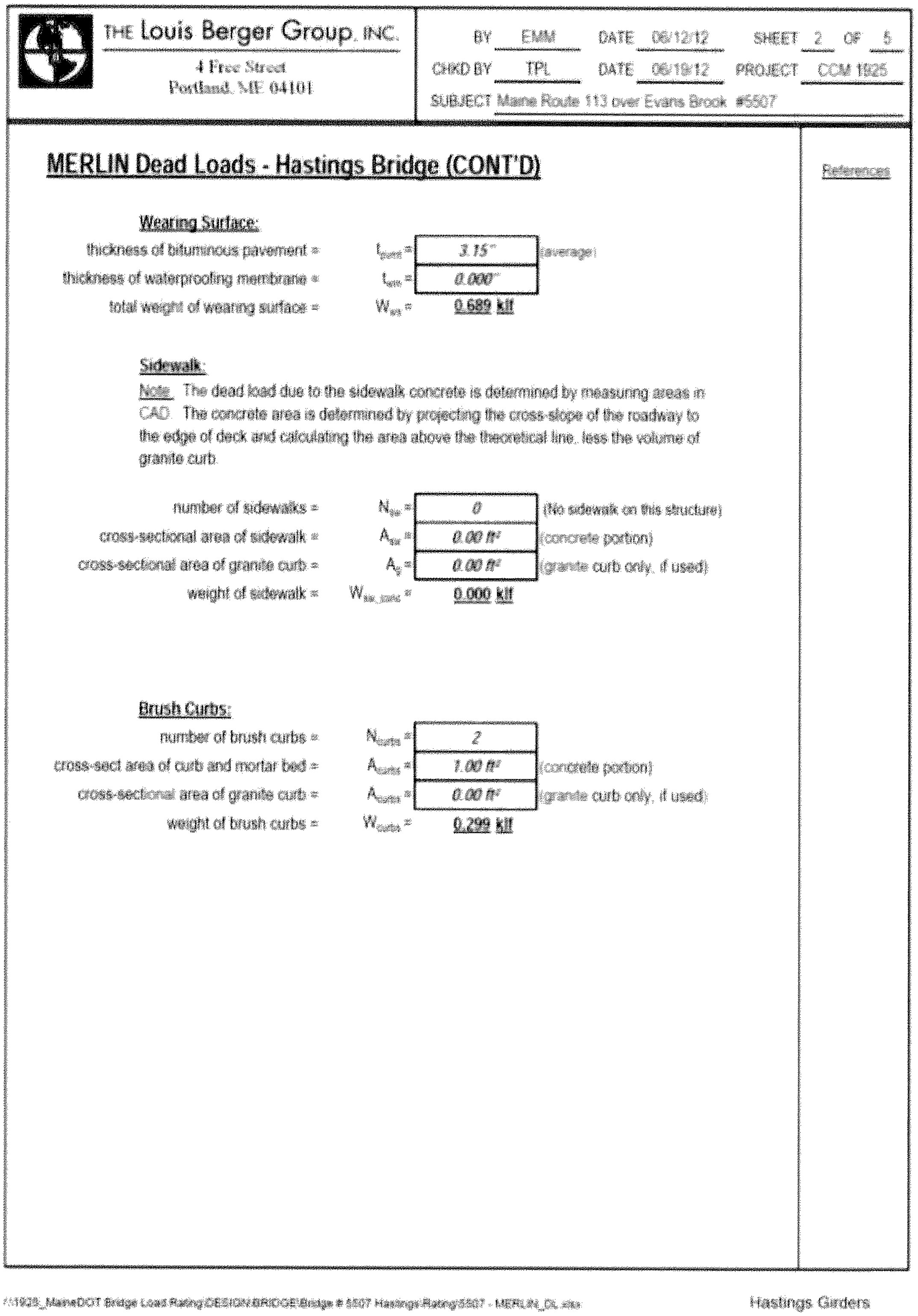
Hastings Girders



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Portland, ME 04101

BY	EMM	DATE	06/12/12	SHEET	2 OF 5
CHKD BY	TPL	DATE	06/19/12	PROJECT	CCM 1925

SUBJECT Maine Route 113 over Evans Brook #5507

References

MERLIN Dead Loads - Hastings Bridge (CONT'D)

Wearing Surface:

thickness of bituminous pavement = t_{pave} = 3.15" (average)
thickness of waterproofing membrane = t_{wm} = 0.000"
total weight of wearing surface = W_{ws} = 0.689 klf

Sidewalk:

Note: The dead load due to the sidewalk concrete is determined by measuring areas in CAD. The concrete area is determined by projecting the cross-slope of the roadway to the edge of deck and calculating the area above the theoretical line, less the volume of granite curb.

number of sidewalks = N_{sw} = 0 (No sidewalk on this structure)
cross-sectional area of sidewalk = A_{sw} = 0.00 ft² (concrete portion)
cross-sectional area of granite curb = A_g = 0.00 ft² (granite curb only, if used)
weight of sidewalk = W_{sw_conc} = 0.000 klf

Brush Curbs:

number of brush curbs = N_{curbs} = 2
cross-sect area of curb and mortar bed = A_{curbs} = 1.00 ft² (concrete portion)
cross-sectional area of granite curb = A_{curbs} = 0.00 ft² (granite curb only, if used)
weight of brush curbs = W_{curbs} = 0.299 klf

\1925_MaineDOT Bridge Load Rating\DESIGN\BRIDGE\Bridge # 5507 Hastings\Rating\5507 - MERLIN_DL.xlsx Hastings Girders

THE Louis Berger Group, INC.
4 Free Street
Portland, ME 04101

BY EMM DATE 06/12/12 SHEET 3 OF 5
CHKD BY TPL DATE 06/19/12 PROJECT CCM 1925
SUBJECT Maine Route 113 over Evans Brook #5507

MERLIN Dead Loads - Hastings Bridge (CONT'D)

References

Bump-outs and Lamposts (Dead Loads):

Note: The dead load due to bump-outs and and light poles are determined by measuring the area in CAD. The load will be applied as a localized distributed load to the exterior girder only. Deck portions are non-composite, remaining portions are composite.

Additional deck conc area = $A_{pilaster\_deck}$ = *0.58 ft²*
Deck Concrete Thickness = $t_{soffit+deck}$ = *7.50"*
Additional Curb conc area = $A_{pilaster\_SW}$ = *0.58 ft²*
Curb Concrete Thickness = t_{sw} = *10.65"*
Pilaster Core Area = A_{p_core} = *0.00 ft²*
Granite Veneer Area = A_{p_veneer} = *0.00 ft²*
Pilaster height = $h_{pilaster}$ = *0.00"*
Concrete coping area = A_{coping} = *0.00 ft²*
Concrete coping thickness = t_{coping} = *0.00"*
Applied Bump-out Length = $L_{pilaster}$ = *1.00 ft* (enter 1.0 for point loads)
Light Pole DL = DL_{post} = *0 lb* (assumed, based on other projects)
non-composite weight/foot of bump-out = W_{pl_nc} = 0.055 kips
composite weight/foot of bump-out = W_{pl_comp} = 0.078 kips

Railings:

$Rail_{Left}$ = Other 0.077 klf
$Rail_{Right}$ = Other 0.077 klf
Barrier Mounted Rail Left = NONE 0.000 klf
Barrier Mounted Rail Right = NONE 0.000 klf
total weight of railing = W_{rails} = 0.154 klf

Snow / Pedestrian Fences:

left fence height = *NONE* Weight = 0 plf
right fence height = *NONE* Weight = 0 plf
total fence weight = W_{fences} = 0.000 klf

Diaphragms:

Note: C15x33.9 based on field measurements
Interior Girder Diaphragm Weight = *0.23 kips*
Exterior Girder Diaphragm Weight = *0.12 kips*
total diaphragm weight = $W_{diaphragm}$ = 0.692 kips

\1925_MaineDOT Bridge Load Rating\DESIGN\BRIDGE\Bridge # 5507 Hastings\Rating\5507 - MERLIN_DL.xlsx Hastings Girders

MERLIN Dead Loads - Hastings Bridge (CONT'D)

References

Bridge Mounted Utilities:

Note: No utilities are present on this bridge

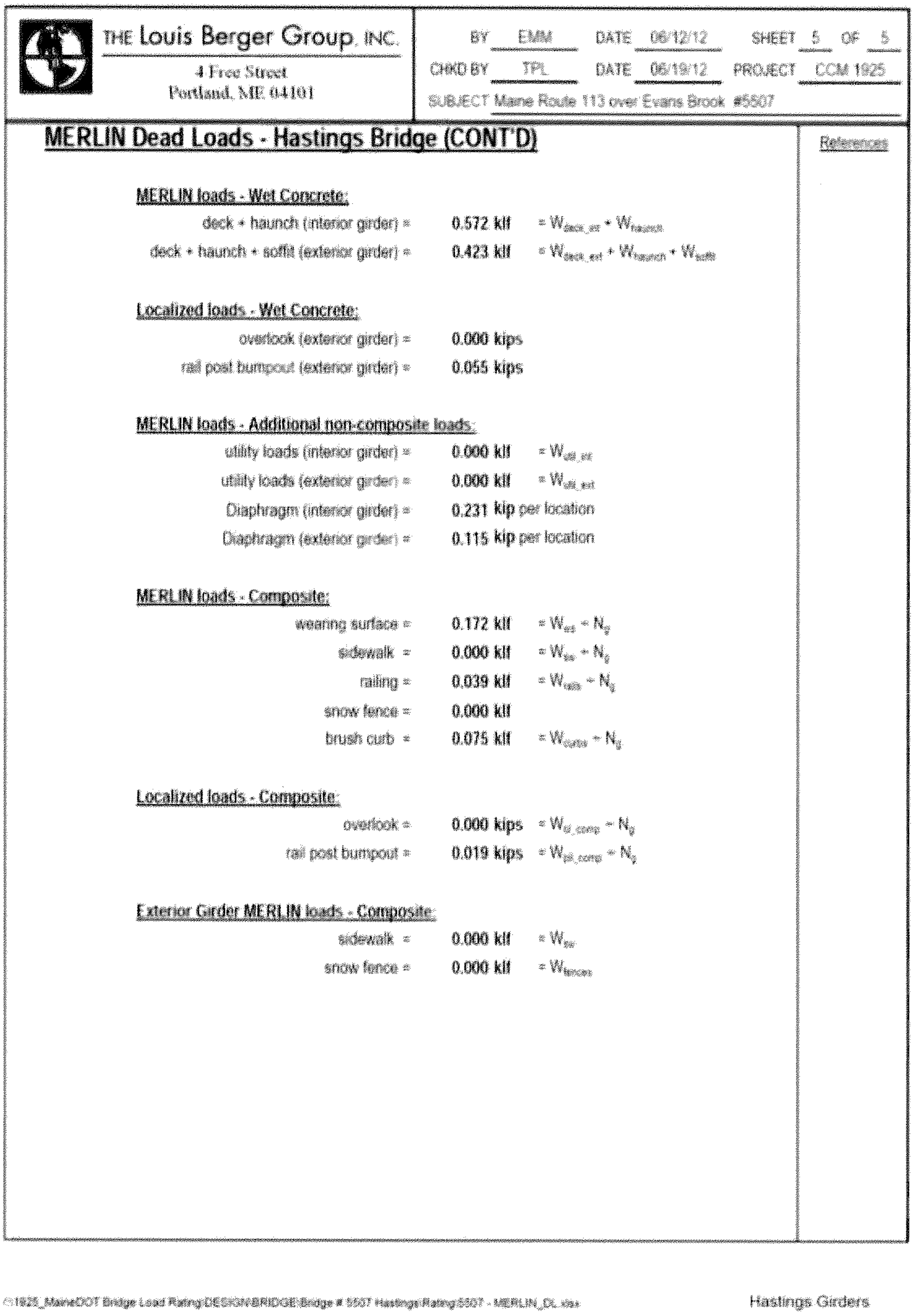
Weight of sewer main line = 0.0 lbs/ft
Inside diameter of sewer main line = 0.00"
Insulation & Hardware Allowance = 0.0 lbs/ft
liquid weight = W_{liquid} = 0.0 lbs/ft
Total line weight = W_{sewer} = 0.000 klf

Weight of water main line = 0.0 lbs/ft
Inside diameter of water main line = 0.00"
Insulation & Hardware Allowance = 0.0 lbs/ft
liquid weight = W_{liquid} = 0.0 lbs/ft
Total line weight = W_{water} = 0.000 klf

Weight of gas line = 0.0 lbs/ft (Assumed)
Inside diameter of gas line = 0.00" (Assumed)
Insulation & Hardware Allowance = 0.0 lbs/ft (Assumed)
liquid weight = W_{liquid} = 0.0 lbs/ft
Total line weight = W_{gas} = 0.000 klf

Weight of electrical ductbank = 0.0 lbs/ft (Assumed)
Hardware Allowance = 0.0 lbs/ft (Assumed)
Total ductbank weight = $W_{electric}$ = 0.000 klf

Girder 1 N/C utility load = 0.000 klf
Girder 2 N/C utility load = 0.000 klf
Girder 3 N/C utility load = 0.000 klf
Girder 4 N/C utility load = 0.000 klf
Girder 5 N/C utility load = 0.000 klf



THE Louis Berger Group, INC.
4 Free Street
Portland, ME 04101

BY EMM DATE 06/12/12 SHEET 5 OF 5
CHKD BY TPL DATE 06/19/12 PROJECT CCM 1925
SUBJECT Maine Route 113 over Evans Brook #5507

References

MERLIN Dead Loads - Hastings Bridge (CONT'D)

MERLIN loads - Wet Concrete:

deck + haunch (interior girder) = **0.572 klf** $= W_{deck\_int} + W_{haunch}$

deck + haunch + soffit (exterior girder) = **0.423 klf** $= W_{deck\_ext} + W_{haunch} + W_{soffit}$

Localized loads - Wet Concrete:

overlook (exterior girder) = **0.000 kips**

rail post bumpout (exterior girder) = **0.055 kips**

MERLIN loads - Additional non-composite loads:

utility loads (interior girder) = **0.000 klf** $= W_{util\_int}$

utility loads (exterior girder) = **0.000 klf** $= W_{util\_ext}$

Diaphragm (interior girder) = **0.231 kip** per location

Diaphragm (exterior girder) = **0.115 kip** per location

MERLIN loads - Composite:

wearing surface = **0.172 klf** $= W_{ws} \div N_g$

sidewalk = **0.000 klf** $= W_{sw} \div N_g$

railing = **0.039 klf** $= W_{rails} \div N_g$

snow fence = **0.000 klf**

brush curb = **0.075 klf** $= W_{curbs} \div N_g$

Localized loads - Composite:

overlook = **0.000 kips** $= W_{ol\_comp} \div N_g$

rail post bumpout = **0.019 kips** $= W_{rpb\_comp} \div N_g$

Exterior Girder MERLIN loads - Composite:

sidewalk = **0.000 klf** $= W_{sw}$

snow fence = **0.000 klf** $= W_{fences}$

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Hastings Girders

The Louis Berger Group, Inc. - 4 Free Street Portland, ME 04101

BY EMM DATE 6/12/2012 PROJECT Maine Load Rating-Steel Bridges
CHK BY TPL DATE 6/18/2012 SUBJECT Hastings Bridge - Br# 5507 SHT NO. 1 OF 3

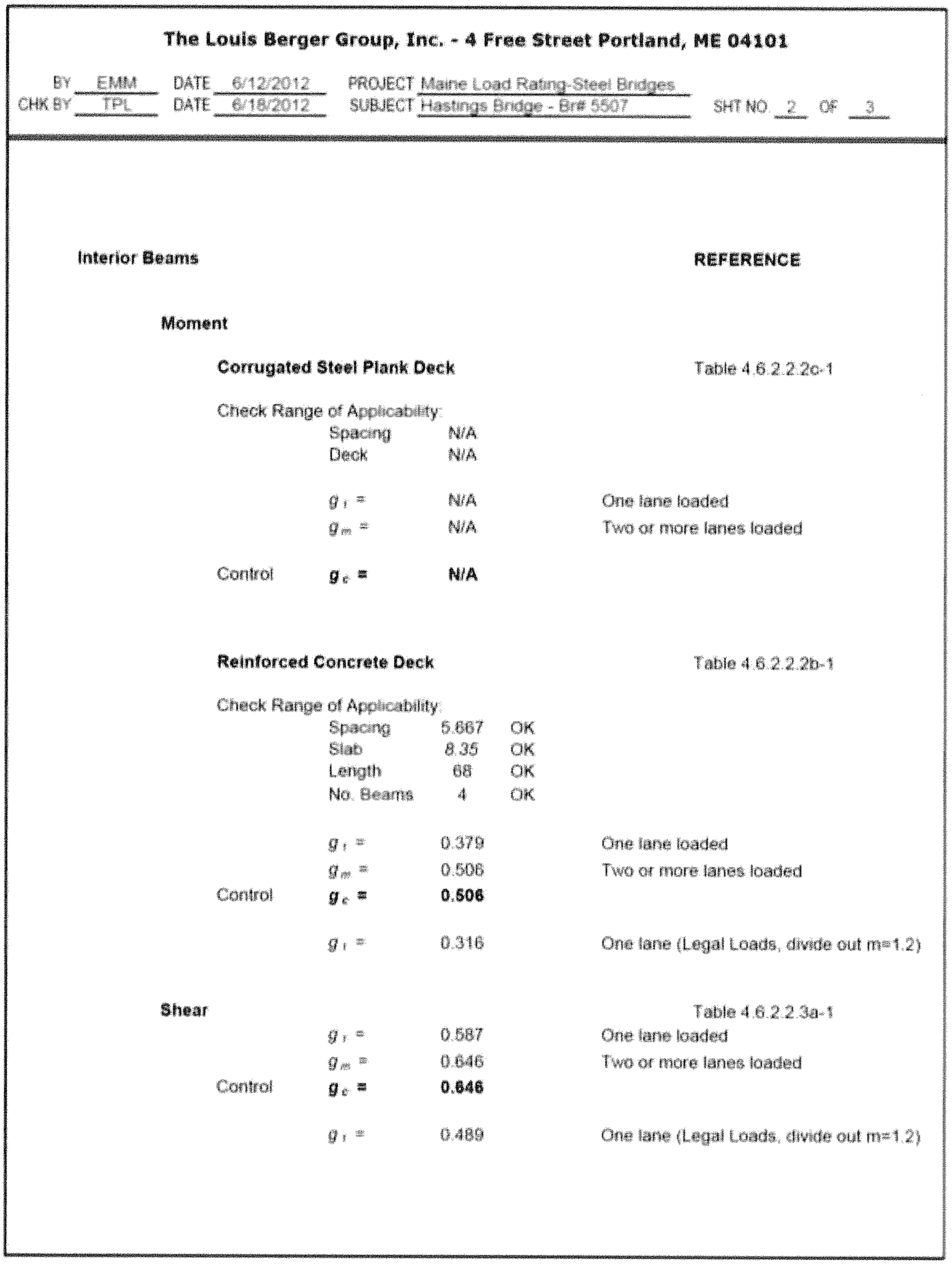
LIVE LOAD DISTRIBUTION FACTORS FOR STEEL BEAM-SLAB BRIDGES **AASHTO 4.6.2.2**

Application

- constant deck width
- number of beams is not less than 4
- parallel beam with same approximate stiffness
- roadway overhang (d_e)is less than or equal to 3 ft.
- beam horizontal curvature is less than 12 degrees
- cross section is consistent with table 4.6.2.2-1 (a)

Variables

Bridge Skew	θ	30 degrees	
Beam/Stringer Spacing	S	5.667 Ft.	
Beam/Stringer Span Length	L	68 Ft.	
Deck Overhang	d_{ovhg}	2 Ft.	
Curb/Sidewalk/Rail	$d_{curb/sw/rail}$	1.125 Ft.	
Roadway Overhang	d_e	0.875 Ft.	
Corrugated Steel Plank ? (Yes/No)		No	
Depth of steel grid or corrugated steel plank	t_g	0 in.	
Depth of concrete slab	t_s	8.35 in.	
Deck Concrete Strength	f'_c	2.5 ksi	
Unit Weight of Concrete	γ_{conc}	0.15 kcf	**Table 3.5.1-1**
No. of Beams	N_b	4	
Depth of beam	D	36.2 in.	**AISC Tbl 1-1**
Beam Steel Strength	f_y	33.0 ksi	
Area of Beam	A	50.1 in^2	**AISC Tbl 1-1**
Moment of Inertia for Beam	I	10500 in^4	**AISC Tbl 1-1**
Modulus of Elasticity for Beam	E_B	29000 ksi	
Modulus of Elasticity for Deck	E_D	3031 ksi	**AASHTO 5.4.2.4-1**
Modular Ratio	n	9.6	**AASHTO 4.6.2.2.1-2**
CG dist. between deck and beam	e_g	22.28 in.	
Longitudinal Stiffness Parameter	K_g	338274.9	**AASHTO 4.6.2.2.1-1**
Longitudinal Stiffness Constant for Moment		0.97	**Table 4.6.2.2.1-2**
Longitudinal Stiffness Constant (Skew Correction for Moment)		0.92	**Table 4.6.2.2.1-2**
Longitudinal Stiffness Constant (Skew Correction for Shear)		1.11	**Table 4.6.2.2.1-2**
Skew Correction for Shear		1.13	**Table 4.6.2.2.3c-1**
Skew Correction for Moment		0.97	**Table 4.6.2.2.2e-1**



The Louis Berger Group, Inc. - 4 Free Street Portland, ME 04101

BY EMM DATE 6/12/2012 PROJECT Maine Load Rating-Steel Bridges
CHK BY TPL DATE 6/18/2012 SUBJECT Hastings Bridge - Br# 5507 SHT NO. 2 OF 3

Interior Beams **REFERENCE**

Moment

Corrugated Steel Plank Deck Table 4.6.2.2.2c-1

Check Range of Applicability:

	Spacing	N/A		
	Deck	N/A		
	g_1 =	N/A		One lane loaded
	g_m =	N/A		Two or more lanes loaded
Control	$\mathbf{g_c =}$	**N/A**		

Reinforced Concrete Deck Table 4.6.2.2.2b-1

Check Range of Applicability:

	Spacing	5.667	OK	
	Slab	8.35	OK	
	Length	68	OK	
	No. Beams	4	OK	
	g_1 =	0.379		One lane loaded
	g_m =	0.506		Two or more lanes loaded
Control	$\mathbf{g_c =}$	**0.506**		
	g_1 =	0.316		One lane (Legal Loads, divide out m=1.2)

Shear Table 4.6.2.2.3a-1

	g_1 =	0.587	One lane loaded
	g_m =	0.646	Two or more lanes loaded
Control	$\mathbf{g_c =}$	**0.646**	
	g_1 =	0.489	One lane (Legal Loads, divide out m=1.2)

The Louis Berger Group, Inc. - 4 Free Street Portland, ME 04101

BY EMM DATE 6/12/2012 PROJECT Maine Load Rating-Steel Bridges
CHK BY TPL DATE 6/18/2012 SUBJECT Hastings Bridge - Br# 5507 SHT NO 3 OF 3

Exterior Beams **REFERENCE**

Moment Table 4.6.2.2.2d-1

Reinforced Concrete Deck

Check Range of Applicability:

Roadway Overhang 0.88 OK

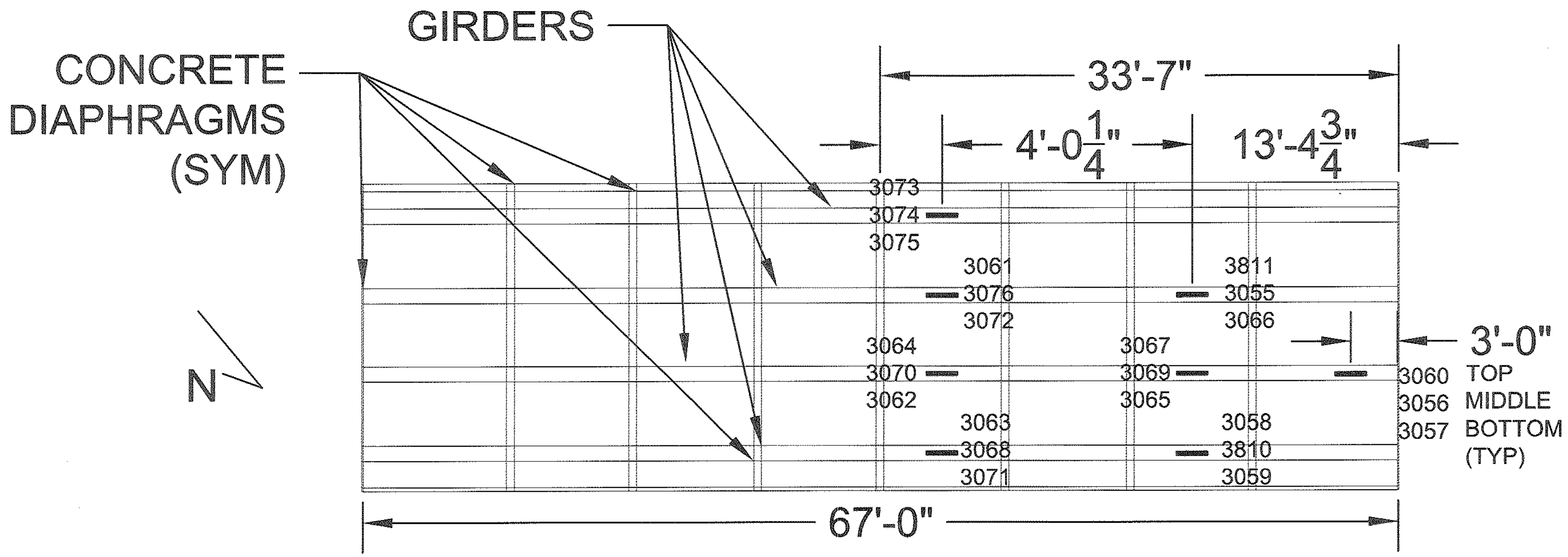
	g_1 =	0.481	Lever Rule	One lane loaded (m=1.2)
	g_m =	0.439		Two or more lanes loaded
	g_{RB} =	N/A		Rigid Body Rotation
With Skew Correction	$\mathbf{g_{c1}}$ **=**	**0.467**		One Lane Loaded
With Skew Correction	$\mathbf{g_{c2}}$ **=**	**0.426**		Two or more lanes loaded
	g_1 =	0.401	Lever Rule	One lane (Legal Loads, divide out m=1.2)

Shear Table 4.6.2.2.3b-1

	g_1 =	0.481	Lever Rule	One lane loaded (m=1.2)
	g_m =	0.444		Two or more lanes loaded
	g_{RB} =	N/A		Rigid Body Rotation
With Skew Correction	$\mathbf{g_{c1}}$ **=**	**0.542**		One Lane Loaded
With Skew Correction	$\mathbf{g_{c2}}$ **=**	**0.501**		Two or more lanes loaded
	g_1 =	0.401	Lever Rule	One lane (Legal Loads, divide out m=1.2)

5506

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



5506

Basic Load Cases

BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area (Me...	Surface (...

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N2	67	0	0	0	

Envelope Joint Reactions

	Joint		X [k]	lc	Y [k]	lc	Z [k]	lc	MX [k-ft]	lc	MY [k-ft]	lc	MZ [k-ft]	lc
1	N1	max	0	1	50.906	1	0	1	0	1	0	1	0	1
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N2	max	0	1	50.906	1	0	1	0	1	0	1	0	1
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	Totals:	max	0	1	57.55	1	0	1						
6		min	0	1	15.95	1	0	1						

Load Combination Design

	Description	ASIF	CD	ABIF	Service	Hot Rolled	Cold Formed	Wood	Concrete	Footings
1						Yes	Yes	Yes	Yes	Yes

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...
1		Yes			M1	1														

Moving Loads

	Tag	Pattern	Increment...	Both Ways	1st Joi...	2nd Jo...	3rd Joint	4th ...	5th ...	6th ...	7th ...	8th ...	9th ...	10th...
1	M1	TESTDUMPTRUCK	1	Yes	N1	N2								

Moving Load Patterns

Pattern Label	Load (k)	Direction	Distance (ft)
TESTDUMPTRUCK	-15.95	Y	0
	-20.65	Y	16
	-20.95	Y	4.667

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2			HR1A	Beam	Wide Flange	A36 Gr.36	Typical

Envelope Member Section Forces

	Member	Sec		Axial[k]	lc	y Shear[k]	lc	z Shear[k]	lc	Torque[k...	lc	y-y Mom...	lc	z-z Mom...	lc
1	M1	1	max	0	1	50.906	1	0	1	0	1	0	1	0	1
2			min	0	1	0	1	0	1	0	1	0	1	0	1
3		2	max	0	1	47.47	1	0	1	0	1	0	1	0	1
4			min	0	1	-.834	1	0	1	0	1	0	1	-167.393	1
5		3	max	0	1	44.893	1	0	1	0	1	0	1	0	1

5506

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[k]	lc	y Shear[k]	lc	z Shear[k]	lc	Torque[k...	lc	y-y Mom...	lc	z-z Mom...	lc
6			min	0	1	-2.701	1	0	1	0	1	0	1	-316.613	1
7		4	max	0	1	41.457	1	0	1	0	1	0	1	0	1
8			min	0	1	-4.564	1	0	1	0	1	0	1	-442.506	1
9		5	max	0	1	38.88	1	0	1	0	1	0	1	0	1
10			min	0	1	-7.047	1	0	1	0	1	0	1	-548.416	1
11		6	max	0	1	35.444	1	0	1	0	1	0	1	0	1
12			min	0	1	-8.91	1	0	1	0	1	0	1	-633.83	1
13		7	max	0	1	32.868	1	0	1	0	1	0	1	0	1
14			min	0	1	-11.394	1	0	1	0	1	0	1	-696.3	1
15		8	max	0	1	29.432	1	0	1	0	1	0	1	0	1
16			min	0	1	-14.829	1	0	1	0	1	0	1	-741.353	1
17		9	max	0	1	26.855	1	0	1	0	1	0	1	0	1
18			min	0	1	-17.406	1	0	1	0	1	0	1	-773.281	1
19		10	max	0	1	23.419	1	0	1	0	1	0	1	0	1
20			min	0	1	-20.842	1	0	1	0	1	0	1	-787.288	1
21		11	max	0	1	20.842	1	0	1	0	1	0	1	0	1
22			min	0	1	-23.419	1	0	1	0	1	0	1	-787.288	1
23		12	max	0	1	17.406	1	0	1	0	1	0	1	0	1
24			min	0	1	-26.855	1	0	1	0	1	0	1	-773.281	1
25		13	max	0	1	14.829	1	0	1	0	1	0	1	0	1
26			min	0	1	-29.432	1	0	1	0	1	0	1	-741.353	1
27		14	max	0	1	11.394	1	0	1	0	1	0	1	0	1
28			min	0	1	-32.868	1	0	1	0	1	0	1	-696.3	1
29		15	max	0	1	8.91	1	0	1	0	1	0	1	0	1
30			min	0	1	-35.444	1	0	1	0	1	0	1	-633.83	1
31		16	max	0	1	7.047	1	0	1	0	1	0	1	0	1
32			min	0	1	-38.88	1	0	1	0	1	0	1	-548.416	1
33		17	max	0	1	4.564	1	0	1	0	1	0	1	0	1
34			min	0	1	-41.457	1	0	1	0	1	0	1	-442.506	1
35		18	max	0	1	2.701	1	0	1	0	1	0	1	0	1
36			min	0	1	-44.893	1	0	1	0	1	0	1	-316.613	1
37		19	max	0	1	.834	1	0	1	0	1	0	1	0	1
38			min	0	1	-47.47	1	0	1	0	1	0	1	-167.393	1
39		20	max	0	1	0	1	0	1	0	1	0	1	0	1
40			min	0	1	-50.906	1	0	1	0	1	0	1	0	1

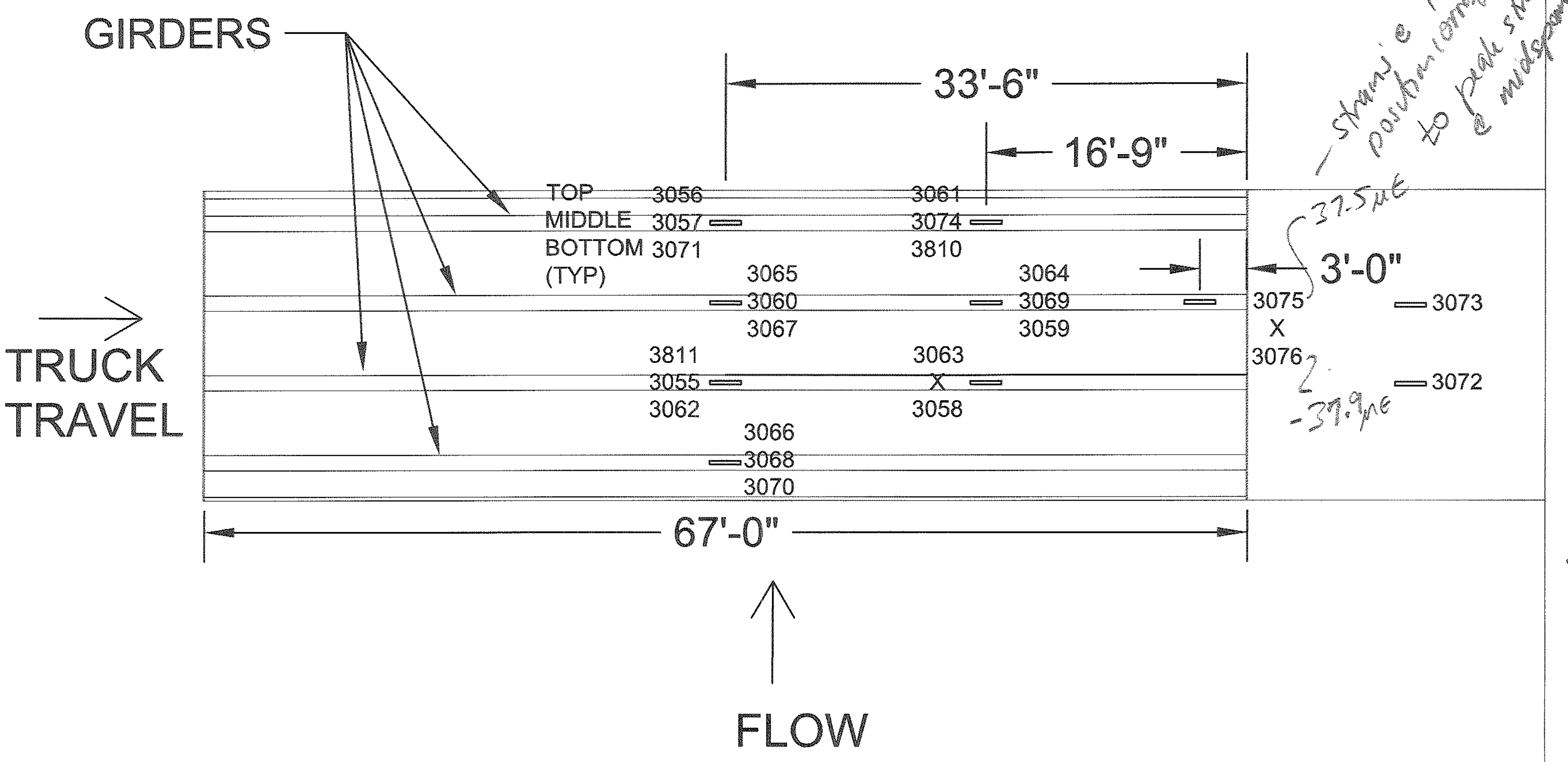
20 elements ⇒ el length = $\frac{67'}{20}$ = 3.35'

Moment @ gauges near midspan = 741.4 + (6.7 − 4') × $\left(\frac{773.3 - 741.4}{3.35'}\right)$
(4' from midspan)

= 767.1 ft-k ← per truck,

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

@238



— strains @ truck position corresponding to peak strains @ midspan

37.5 με

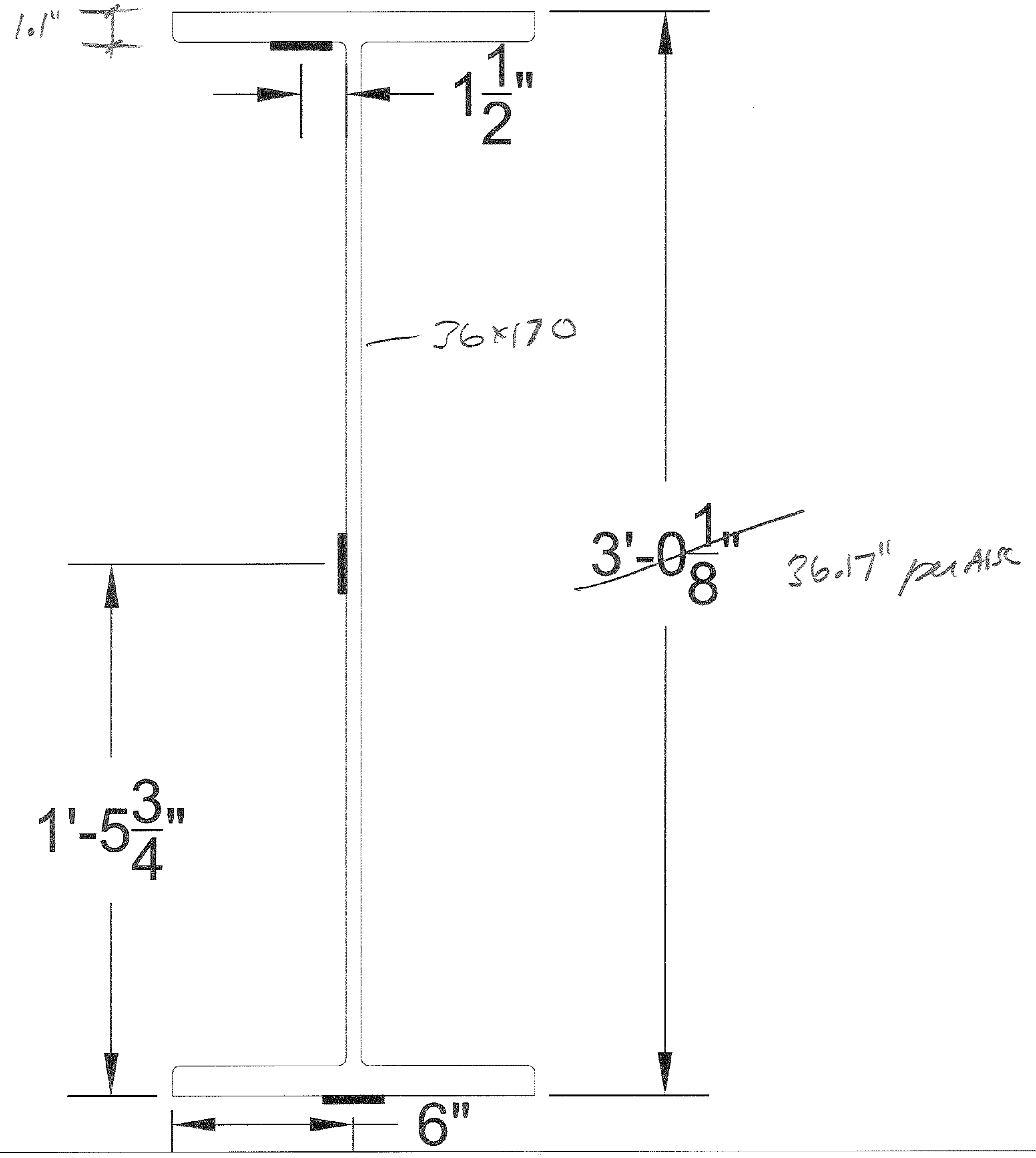
-37.9 με

11/14/2015
W. Davids

11/14/2013

W. Davids

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



5507

Basic Load Cases

BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area (Me...	Surface (...

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N2	68	0	0	0	

Envelope Joint Reactions

	Joint		X [k]	lc	Y [k]	lc	Z [k]	lc	MX [k-ft]	lc	MY [k-ft]	lc	MZ [k-ft]	lc
1	N1	max	0	1	51.003	1	0	1	0	1	0	1	0	1
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N2	max	0	1	51.003	1	0	1	0	1	0	1	0	1
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	Totals:	max	0	1	57.55	1	0	1						
6		min	0	1	15.95	1	0	1						

Load Combination Design

	Description	ASIF	CD	ABIF	Service	Hot Rolled	Cold Formed	Wood	Concrete	Footings
1						Yes	Yes	Yes	Yes	Yes

Load Combinations

	Description	Solve	PDelta	SRSS	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...
1		Yes			M1	1														

Moving Loads

	Tag	Pattern	Increment...	Both Ways	1st Joi...	2nd Jo...	3rd Joint	4th ...	5th ...	6th ...	7th ...	8th ...	9th ...	10th...
1	M1	TESTDUMPTRUCK	1	Yes	N1	N2								

Moving Load Patterns

Pattern Label	Load (k)	Direction	Distance (ft)
TESTDUMPTRUCK	-15.95	Y	0
	-20.65	Y	16
	-20.95	Y	4.667

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2			HR1A	Beam	Wide Flange	A36 Gr.36	Typical

Envelope Member Section Forces

	Member	Sec		Axial[k]	lc	y Shear[k]	lc	z Shear[k]	lc	Torque[k...	lc	y-y Mom...	lc	z-z Mom...		lc
1	M1	1	max	0	1	51.003	1	0	1	0	1	0	1	0		1
2			min	0	1	0	1	0	1	0	1	0	1	0		1
3		2	max	0	1	47.618	1	0	1	0	1	0	1	0		1
4			min	0	1	-.822	1	0	1	0	1	0	1	-170.422		1
5		3	max	0	1	45.079	1	0	1	0	1	0	1	0		1

5507

Envelope Member Section Forces (Continued)

	Member	Sec		Axial[k]	lc	y Shear[k]	lc	z Shear[k]	lc	Torque[k...	lc	y-y Mom...	lc	z-z Mom...	lc
6			min	0	1	-2.661	1	0	1	0	1	0	1	-322.671	1
7		4	max	0	1	41.694	1	0	1	0	1	0	1	0	1
8			min	0	1	-5.108	1	0	1	0	1	0	1	-448.285	1
9		5	max	0	1	39.155	1	0	1	0	1	0	1	0	1
10			min	0	1	-6.944	1	0	1	0	1	0	1	-560.531	1
11		6	max	0	1	35.769	1	0	1	0	1	0	1	0	1
12			min	0	1	-9.391	1	0	1	0	1	0	1	-643.462	1
13		7	max	0	1	32.384	1	0	1	0	1	0	1	0	1
14			min	0	1	-11.226	1	0	1	0	1	0	1	-710.634	1
15		8	max	0	1	29.845	1	0	1	0	1	0	1	0	1
16			min	0	1	-14.611	1	0	1	0	1	0	1	-754.585	1
17		9	max	0	1	26.46	1	0	1	0	1	0	1	0	1
18			min	0	1	-17.15	1	0	1	0	1	0	1	-788.692	1
19		10	max	0	1	23.921	1	0	1	0	1	0	1	0	1
20			min	0	1	-20.536	1	0	1	0	1	0	1	-800.277	1
21		11	max	0	1	20.536	1	0	1	0	1	0	1	0	1
22			min	0	1	-23.921	1	0	1	0	1	0	1	-800.277	1
23		12	max	0	1	17.15	1	0	1	0	1	0	1	0	1
24			min	0	1	-26.46	1	0	1	0	1	0	1	-788.692	1
25		13	max	0	1	14.611	1	0	1	0	1	0	1	0	1
26			min	0	1	-29.845	1	0	1	0	1	0	1	-754.585	1
27		14	max	0	1	11.226	1	0	1	0	1	0	1	0	1
28			min	0	1	-32.384	1	0	1	0	1	0	1	-710.634	1
29		15	max	0	1	9.391	1	0	1	0	1	0	1	0	1
30			min	0	1	-35.769	1	0	1	0	1	0	1	-643.462	1
31		16	max	0	1	6.944	1	0	1	0	1	0	1	0	1
32			min	0	1	-39.155	1	0	1	0	1	0	1	-560.531	1
33		17	max	0	1	5.108	1	0	1	0	1	0	1	0	1
34			min	0	1	-41.694	1	0	1	0	1	0	1	-448.285	1
35		18	max	0	1	2.661	1	0	1	0	1	0	1	0	1
36			min	0	1	-45.079	1	0	1	0	1	0	1	-322.671	1
37		19	max	0	1	.822	1	0	1	0	1	0	1	0	1
38			min	0	1	-47.618	1	0	1	0	1	0	1	-170.422	1
39		20	max	0	1	0	1	0	1	0	1	0	1	0	1
40			min	0	1	-51.003	1	0	1	0	1	0	1	0	1

Appendix C – Strain Plots

Evans Brook Bridge (No. 5506)

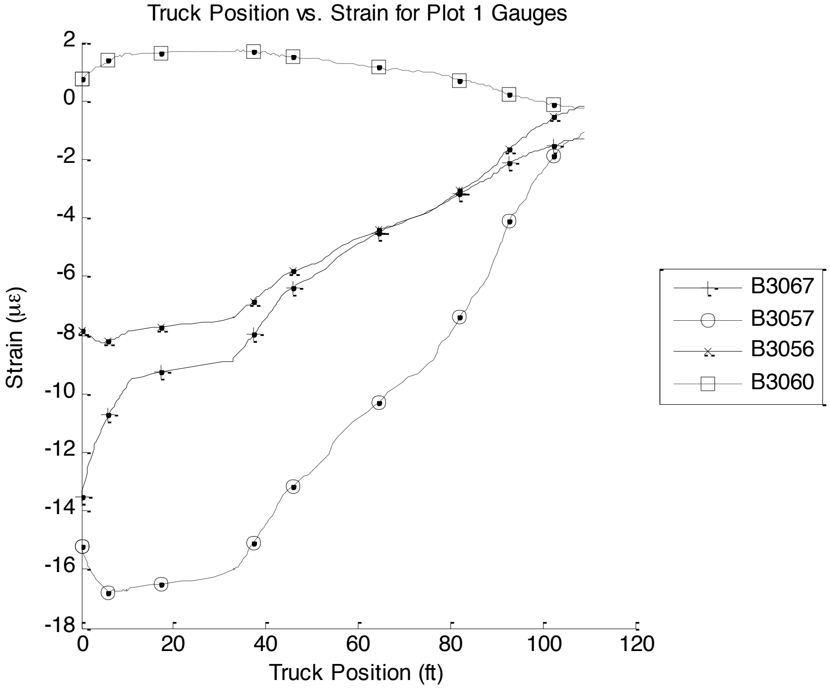


Figure 15 - Strain plots for Node 1 gages during Test 1 (beginning of test cut off).

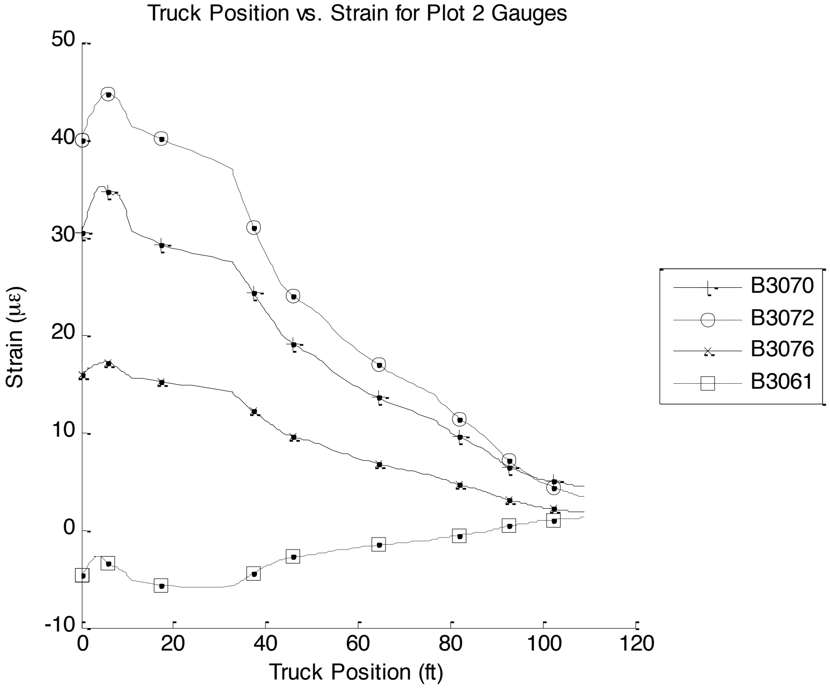


Figure 16 - Strain plots for Node 2 gages during Test 1 (beginning of test cut off).

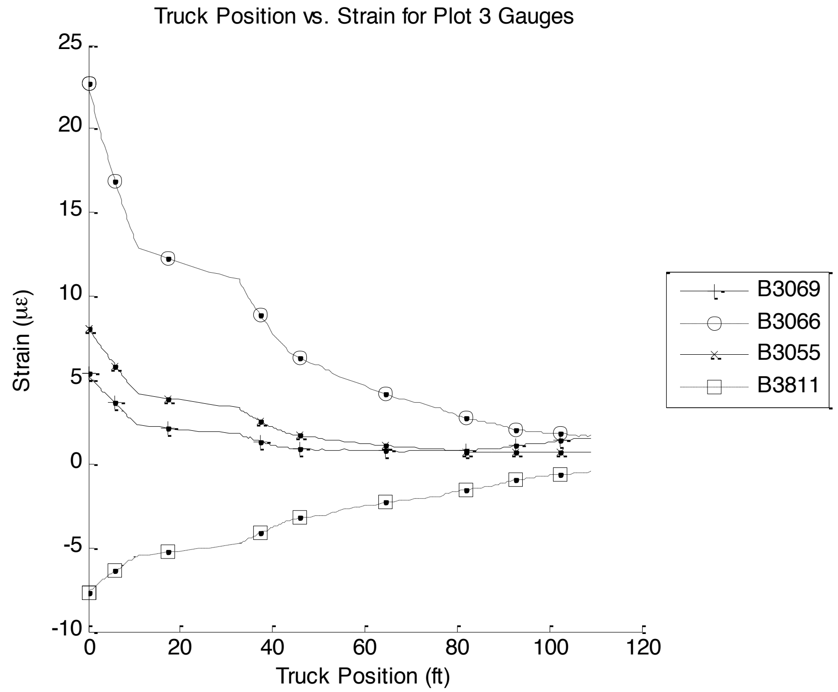


Figure 17 - Strain plots for Node 3 gages during Test 1 (beginning of test cut off).

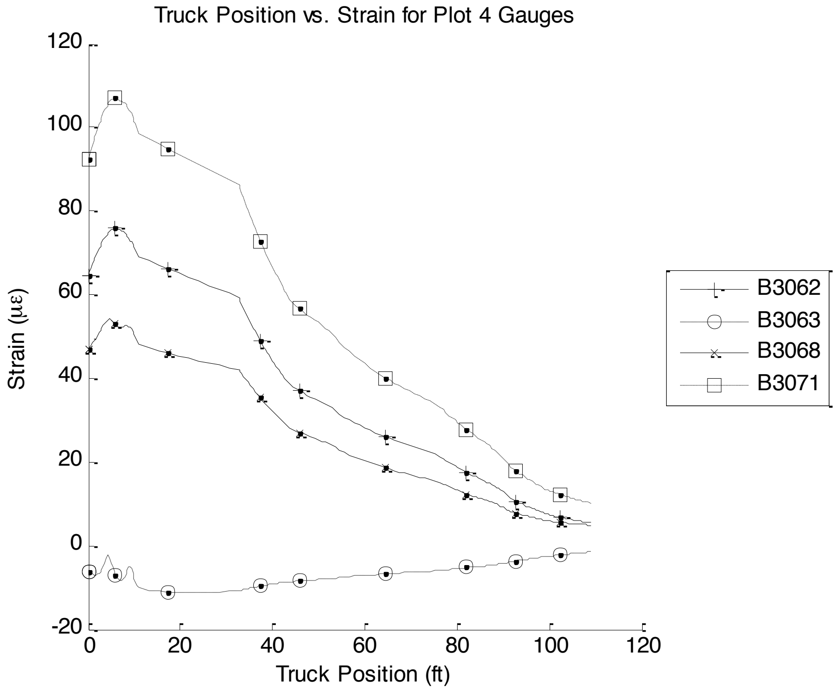


Figure 18 - Strain plots for Node 4 gages during Test 1 (beginning of test cut off).

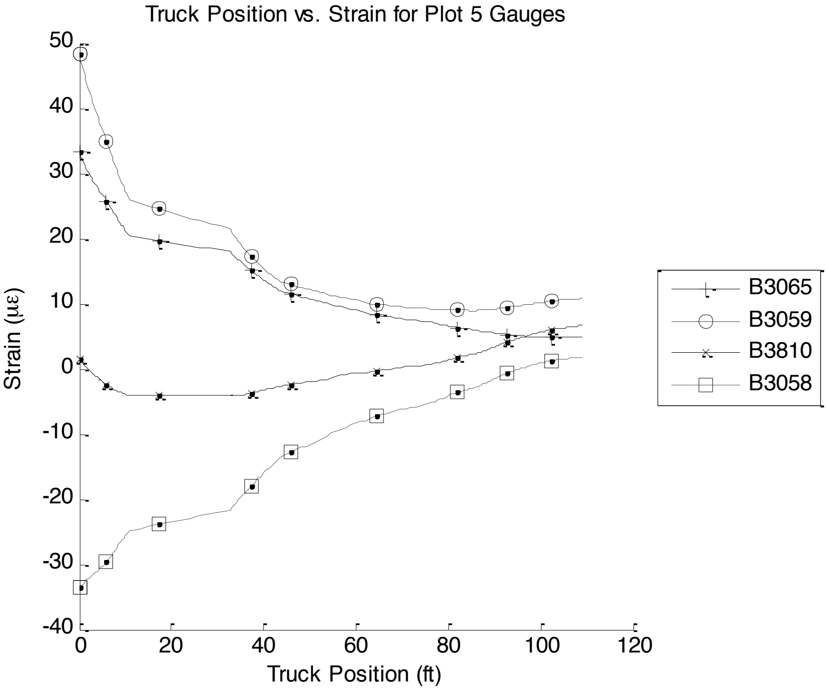


Figure 19 - Strain plots for Node 5 gages during Test 1 (beginning of test cut off).

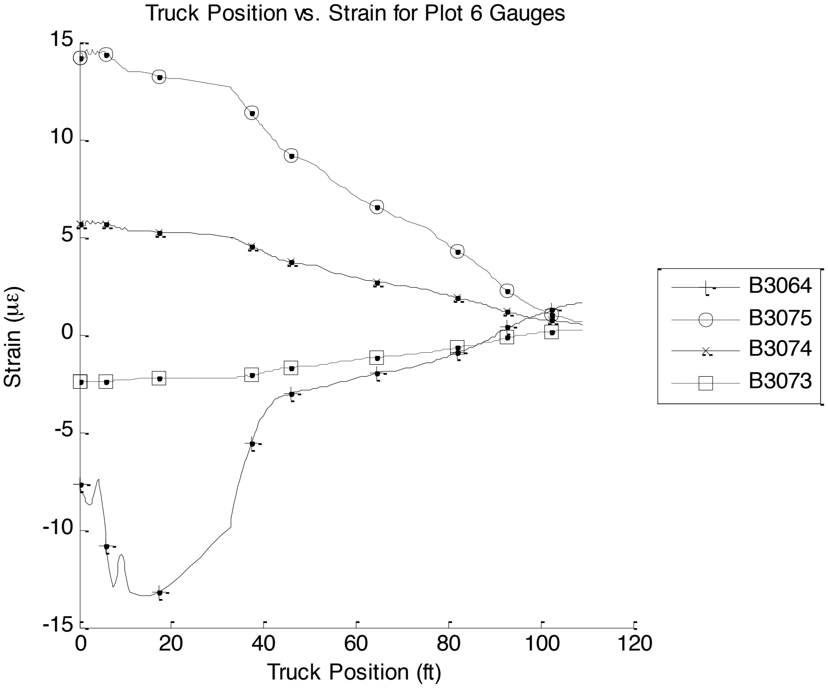


Figure 20 - Strain plots for Node 6 gages during Test 1 (beginning of test cut off).

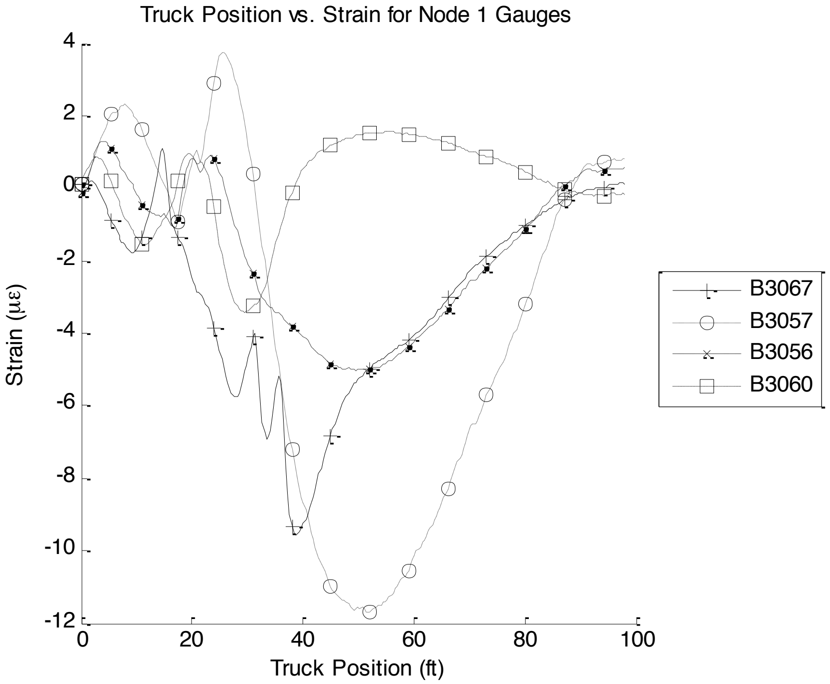


Figure 21 - Strain plots for Node 1 gages during Test 2

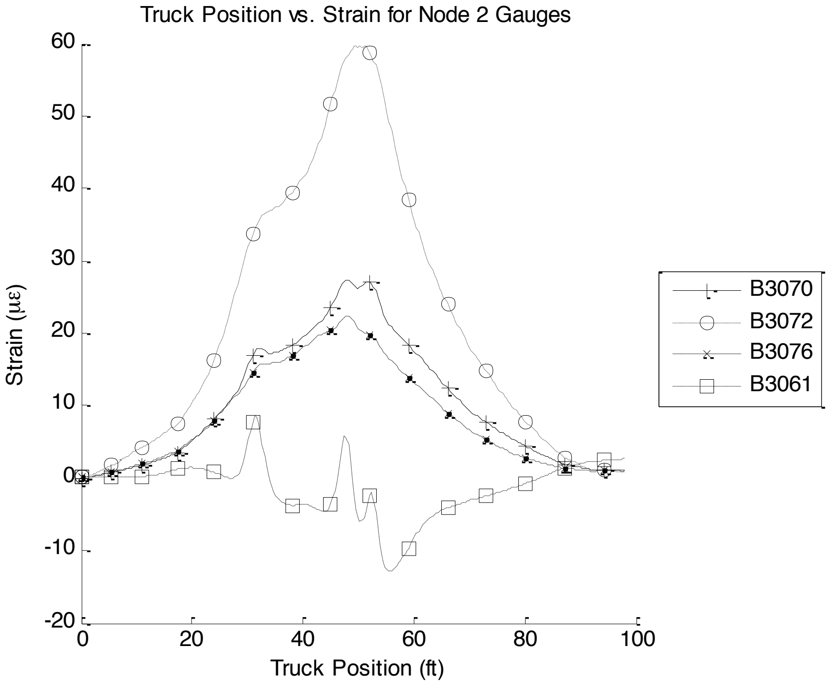


Figure 22 - Strain plots for Node 2 gages during Test 2.

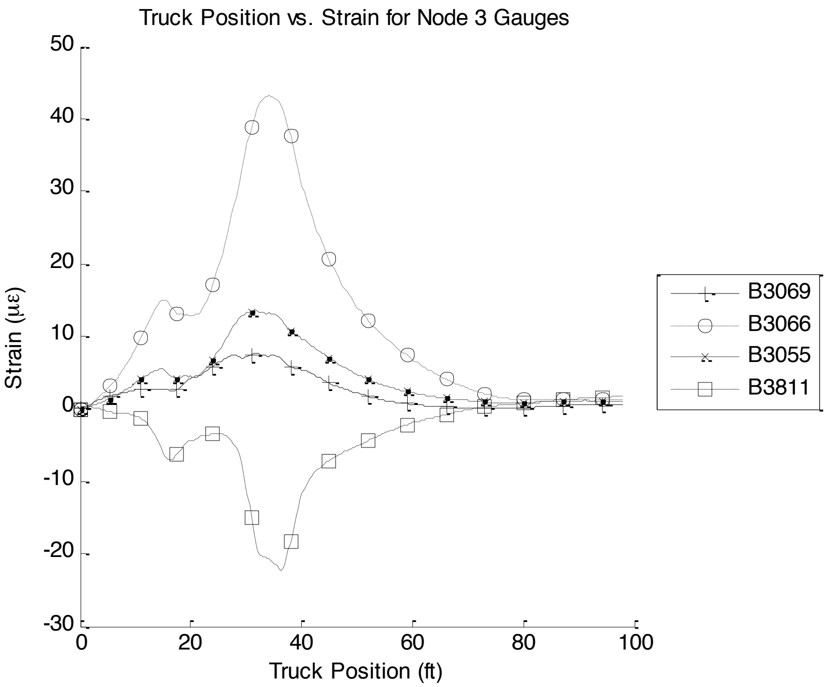


Figure 23 - Strain plots for Node 3 gages during Test 2.

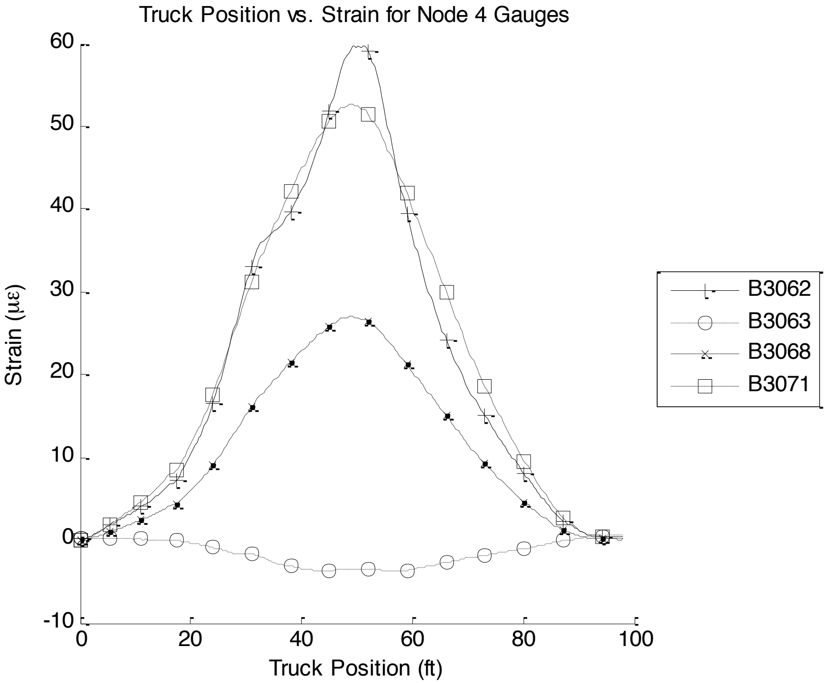


Figure 24 - Strain plots for Node 4 gages during Test 2.

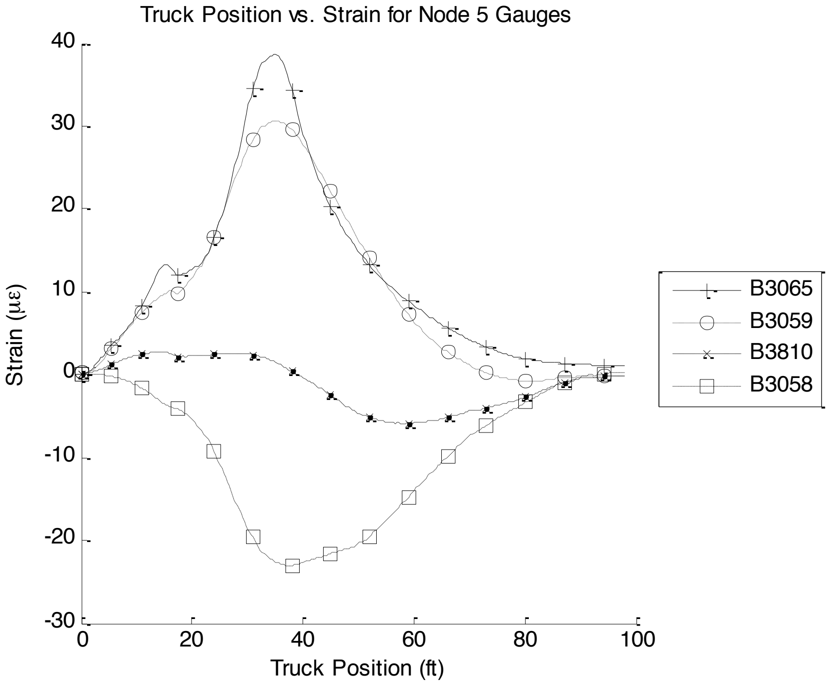


Figure 25 - Strain plots for Node 5 gages during Test 2.

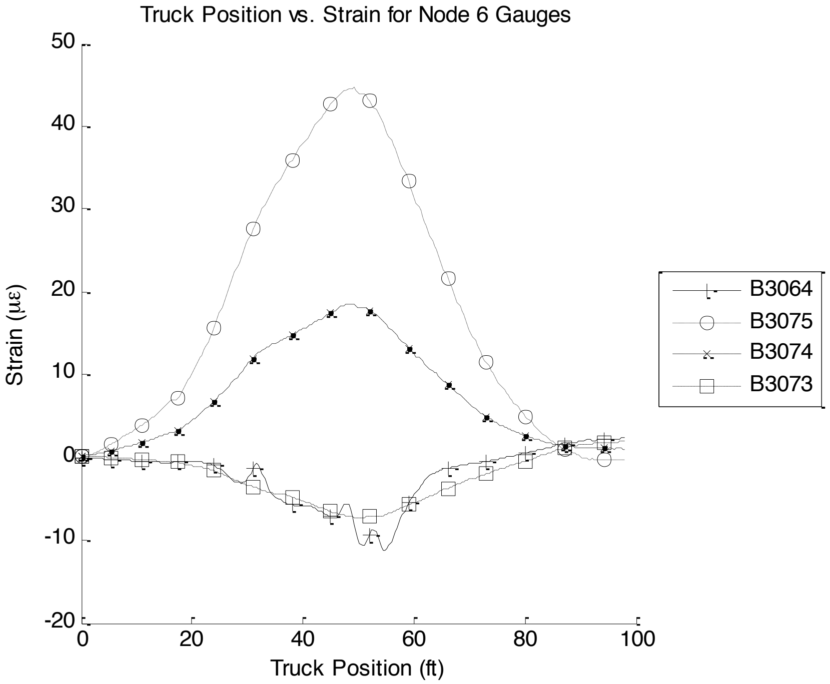


Figure 26 - Strain plots for Node 6 gages during Test 2.

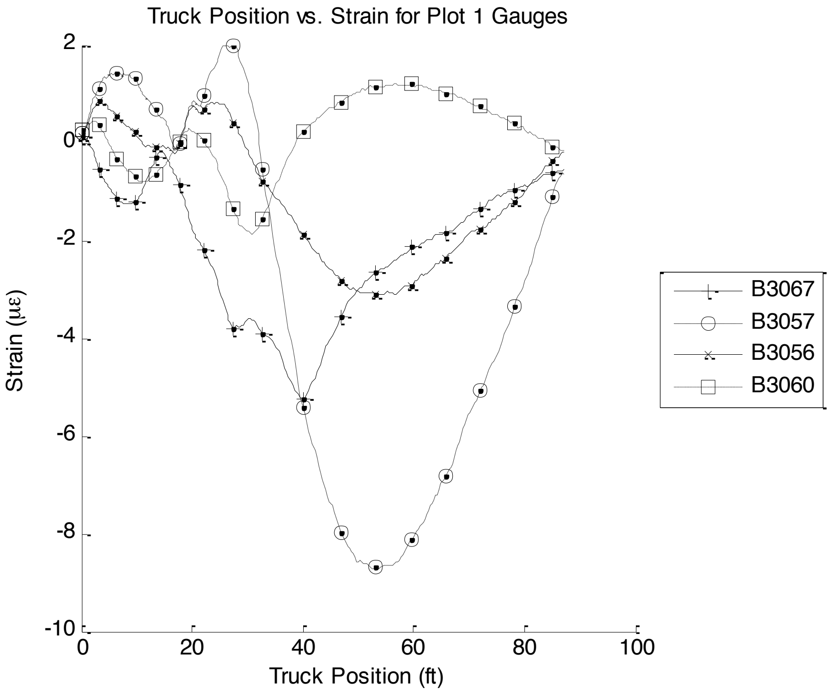


Figure 27 - Strain plots for Node 1 gages during Test 3.

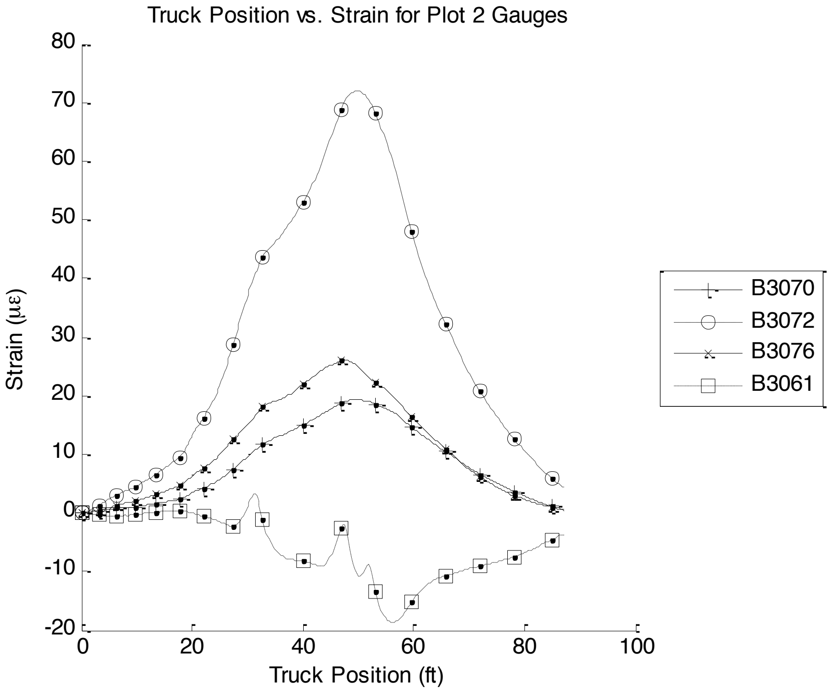


Figure 28 - Strain plots for Node 2 gages during Test 3.

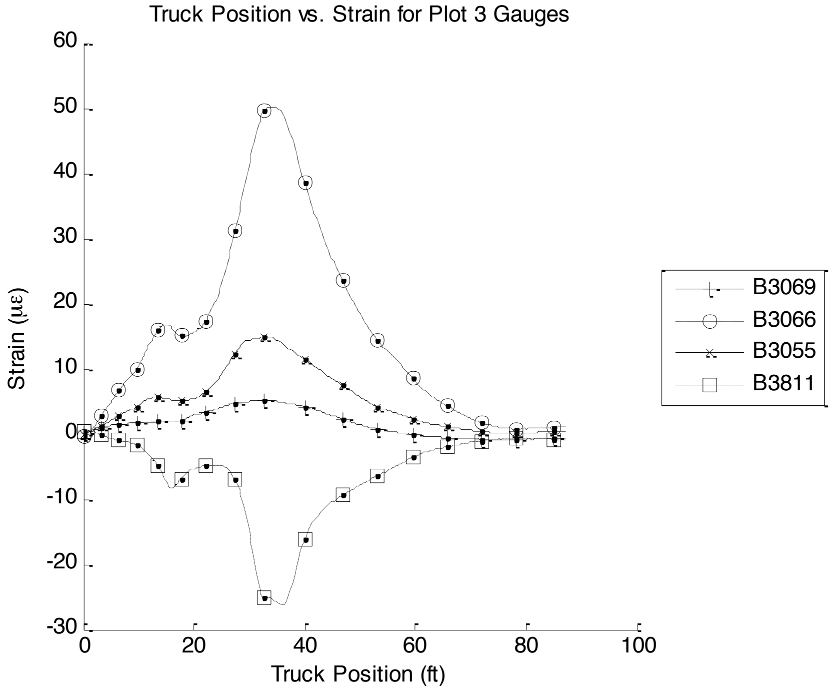


Figure 29 - Strain plots for Node 3 gages during Test 3.

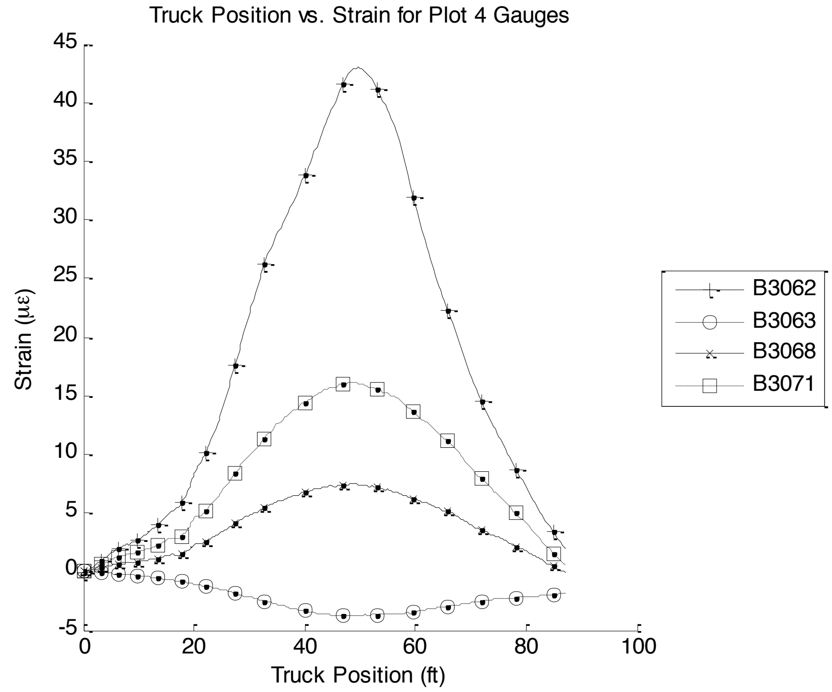


Figure 30 - Strain plots for Node 4 gages during Test 3.

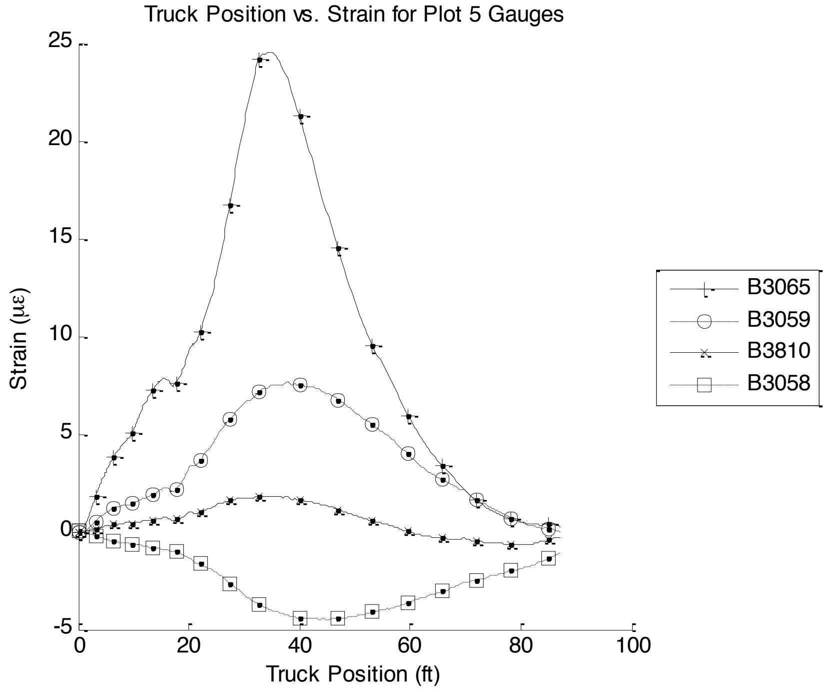


Figure 31 - Strain plots for Node 5 gages during Test 3.

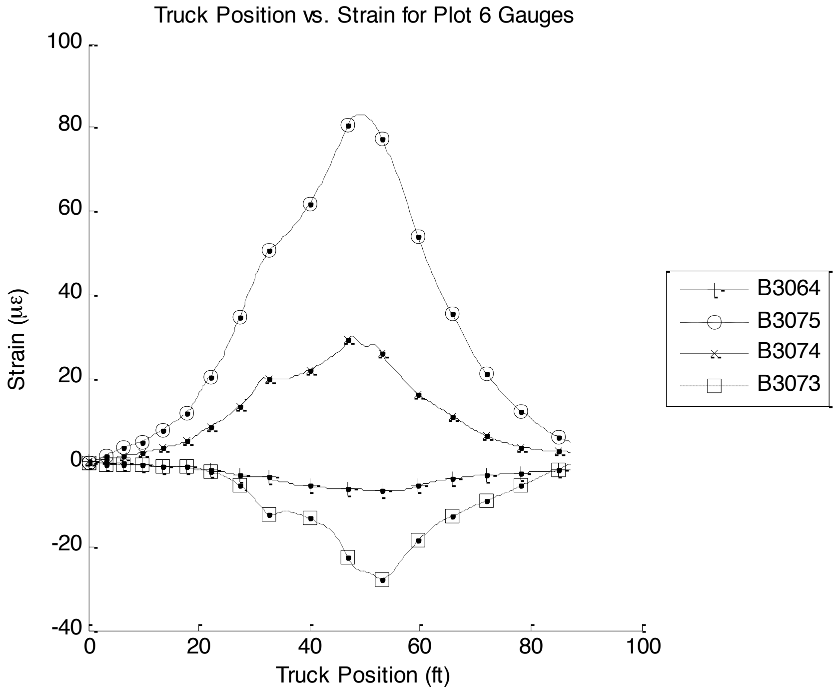


Figure 32 - Strain plots for Node 6 gages during Test 3.

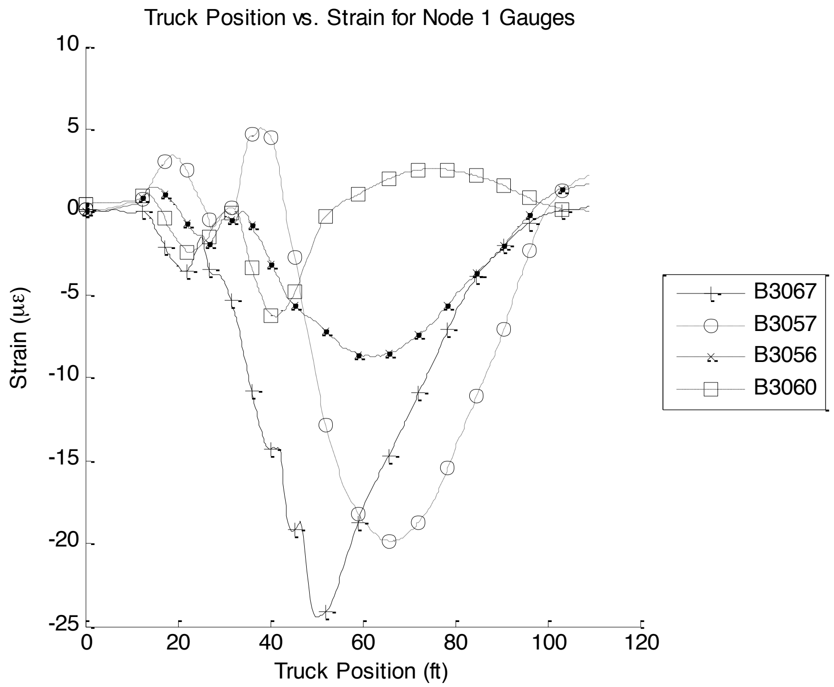


Figure 33 - Strain plots for Node 1 gages during Test 4.

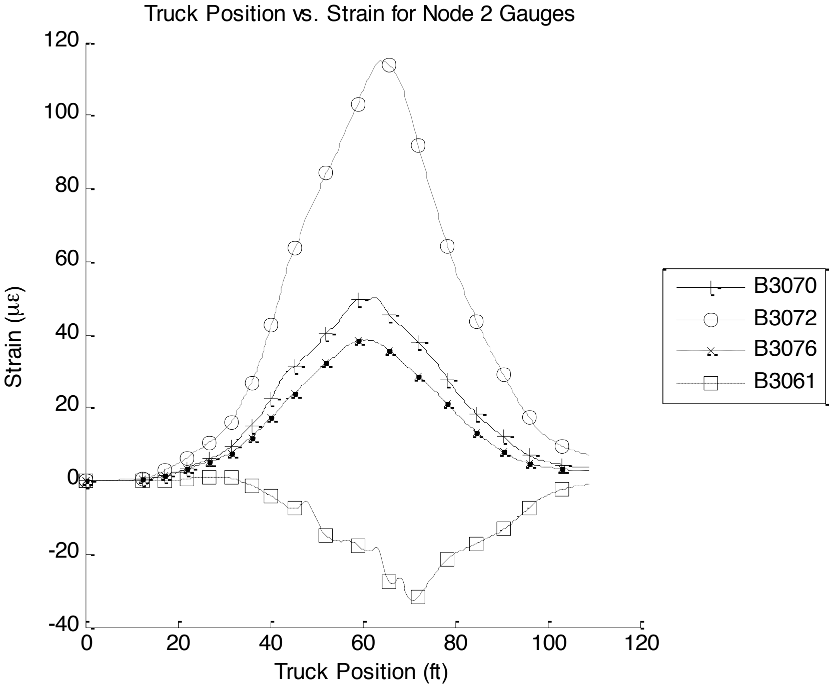


Figure 34 - Strain plots for Node 2 gages during Test 4.

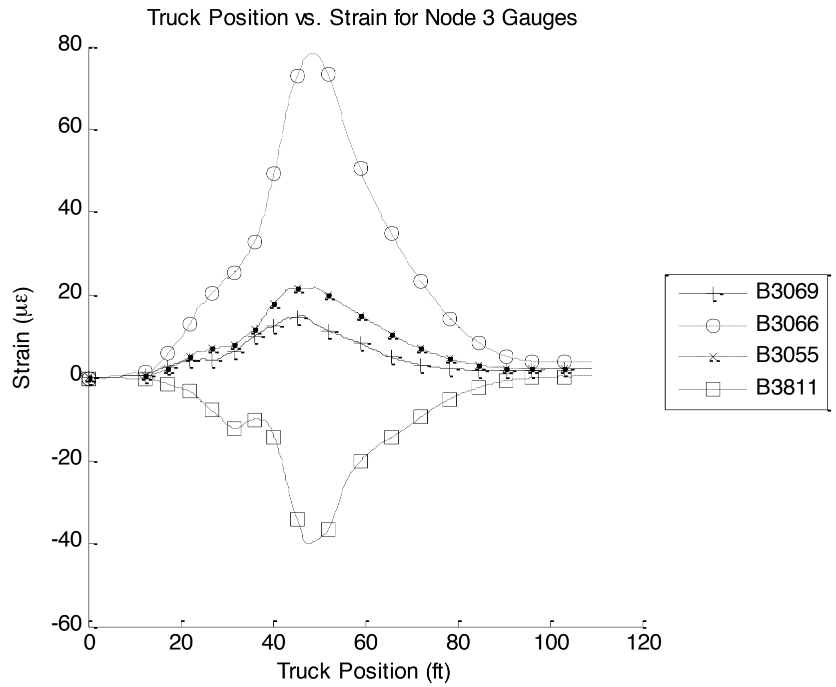


Figure 35 - Strain plots for Node 3 gages during Test 4.

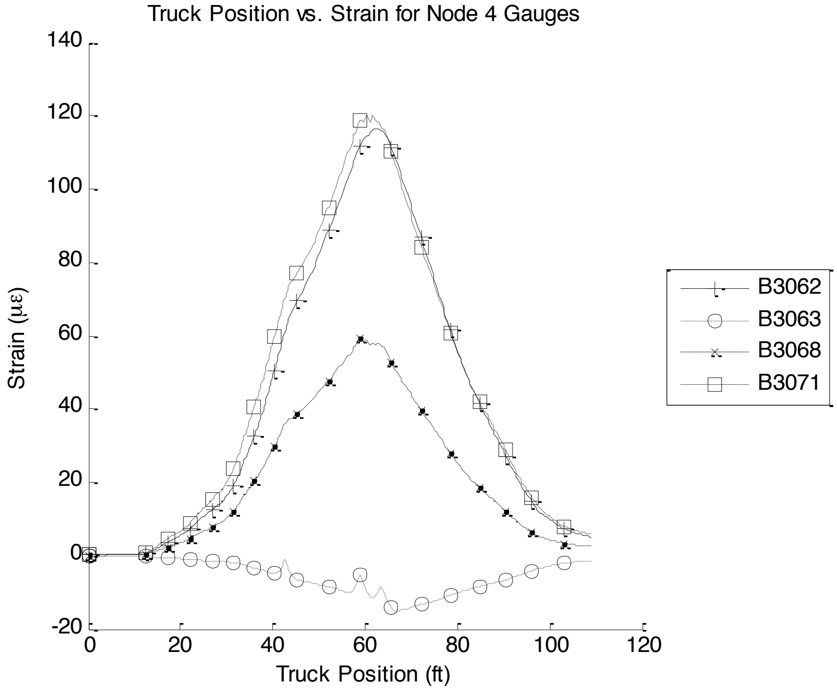


Figure 36 - Strain plots for Node 4 gages during Test 4.

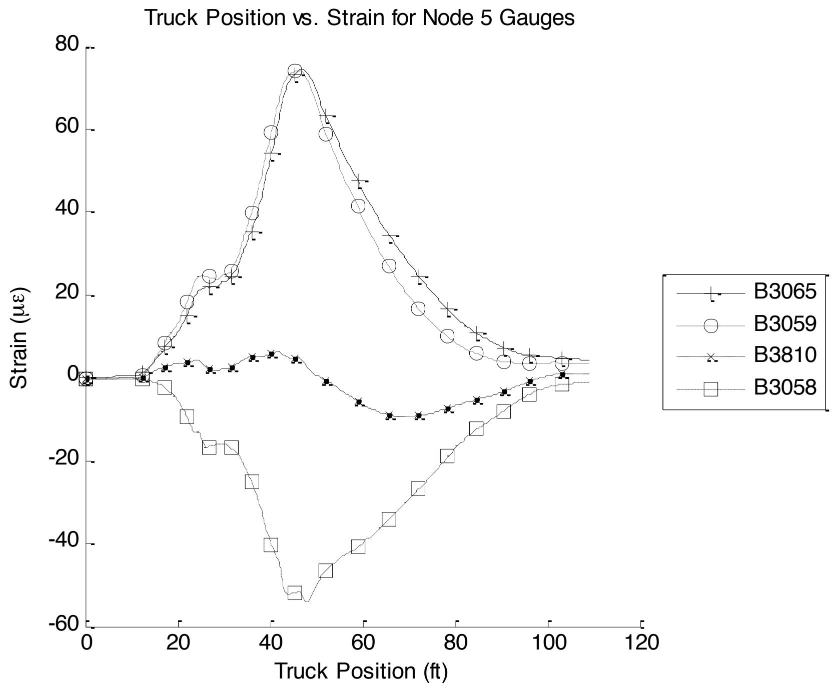


Figure 37 - Strain plots for Node 5 gages during Test 4.

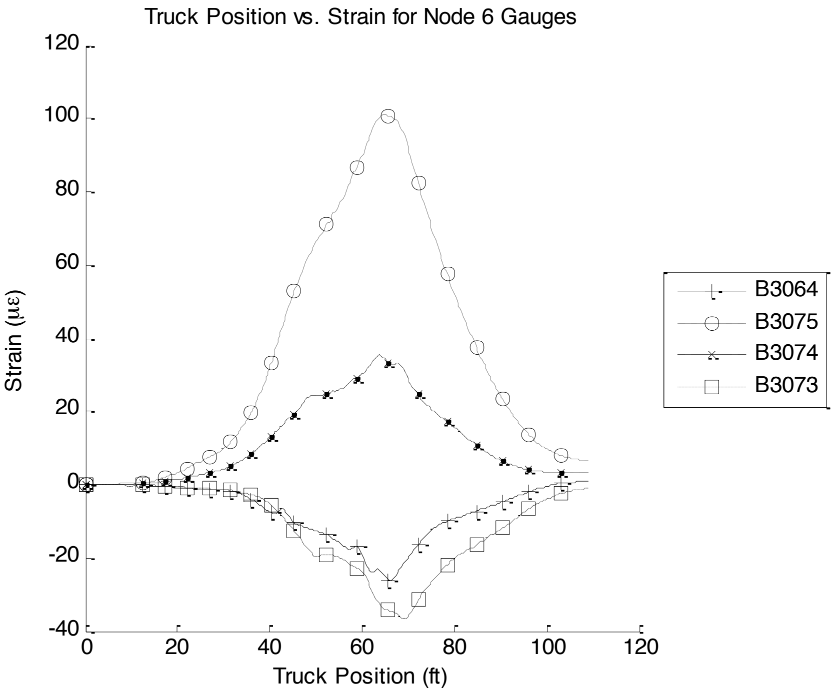


Figure 38 - Strain plots for Node 6 gages during Test 4.

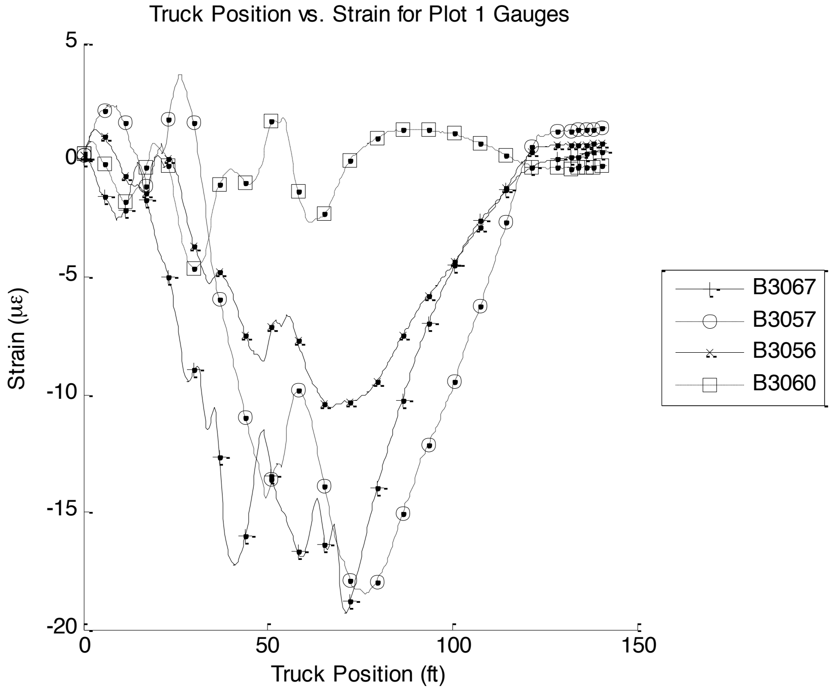


Figure 39 - Strain plots for Node 1 gages during Test 5.

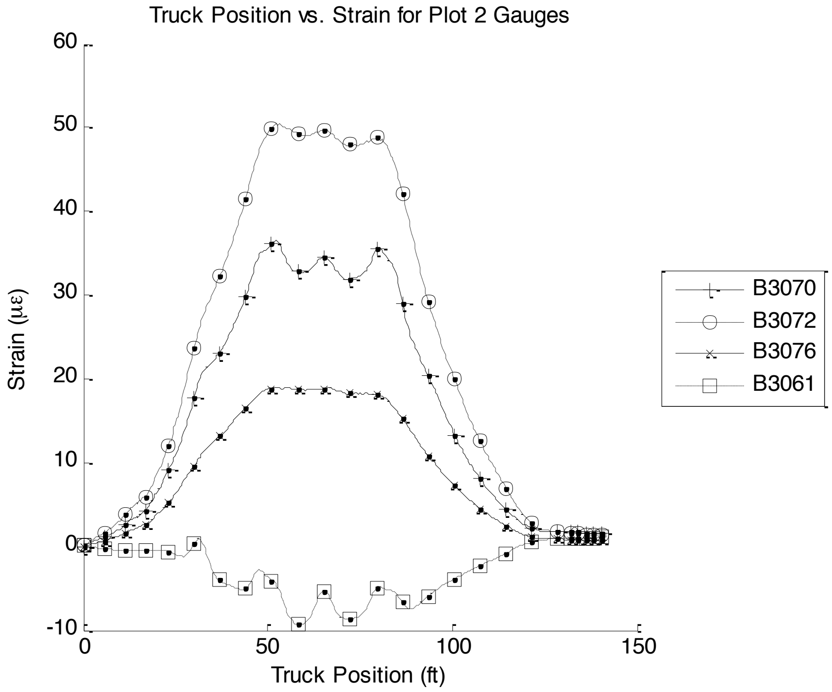


Figure 40 - Strain plots for Node 2 gages during Test 5.

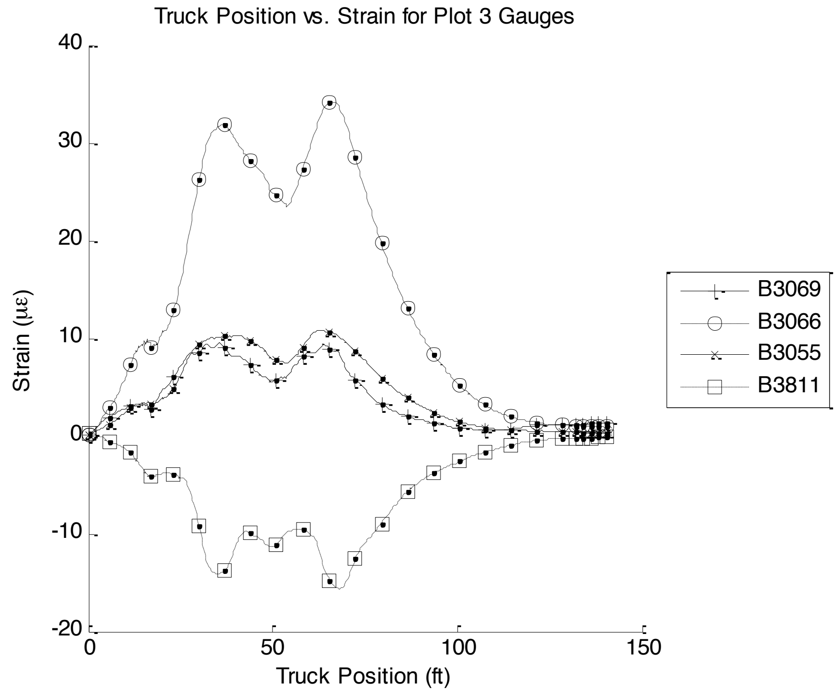


Figure 41 - Strain plots for Node 3 gages during Test 5.

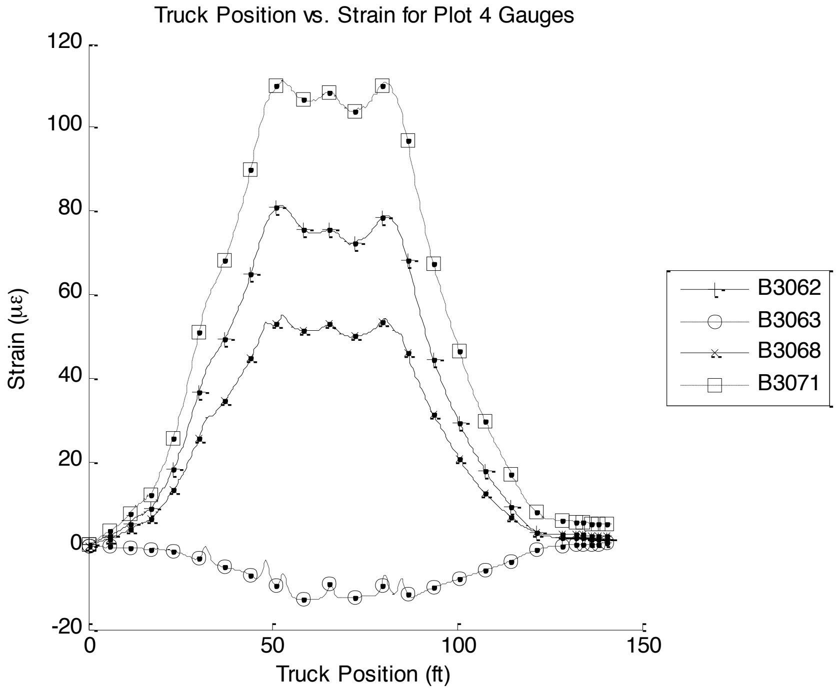


Figure 42 - Strain plots for Node 4 gages during Test 5.

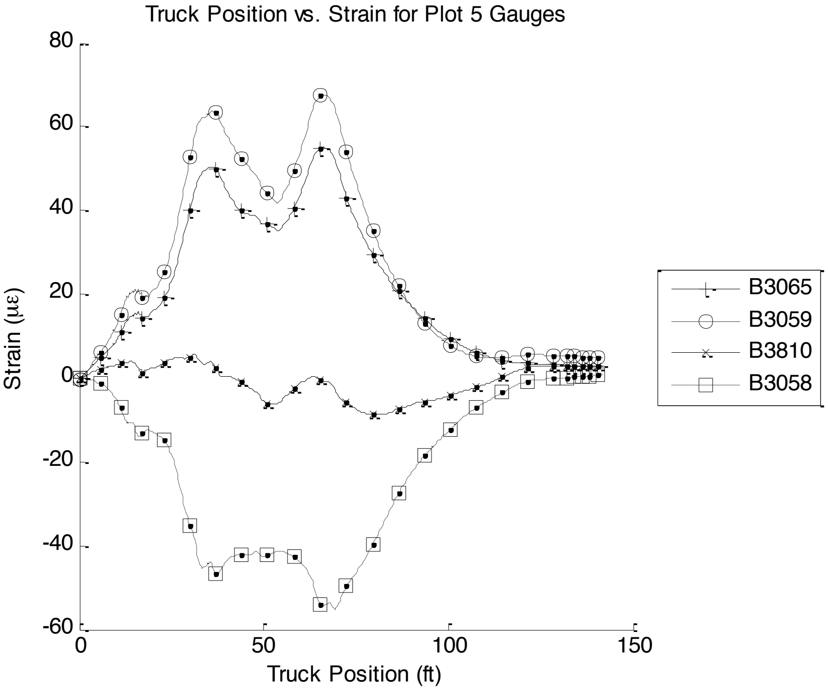


Figure 43 - Strain plots for Node 5 gages during Test 5.

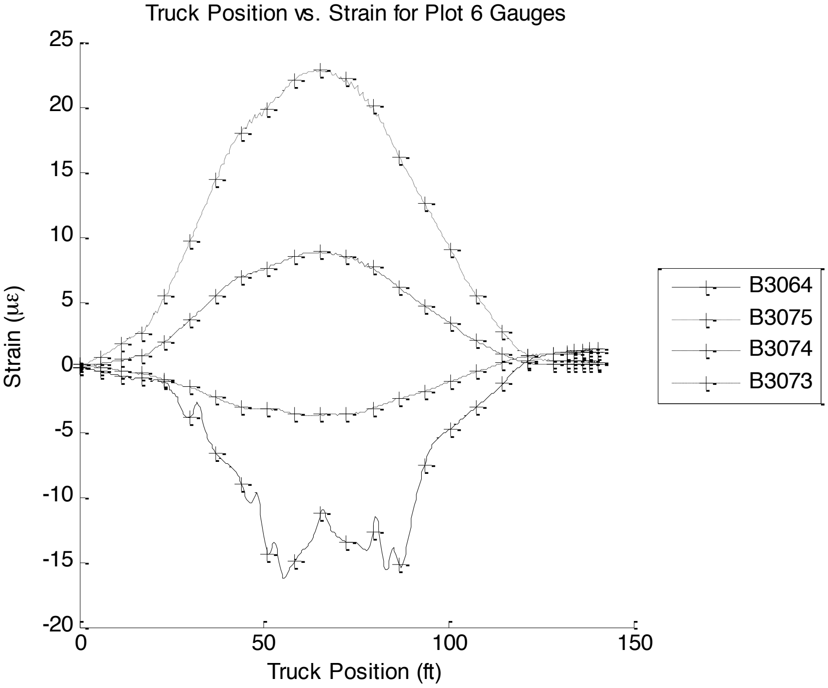


Figure 44 - Strain plots for Node 6 gages during Test 5.

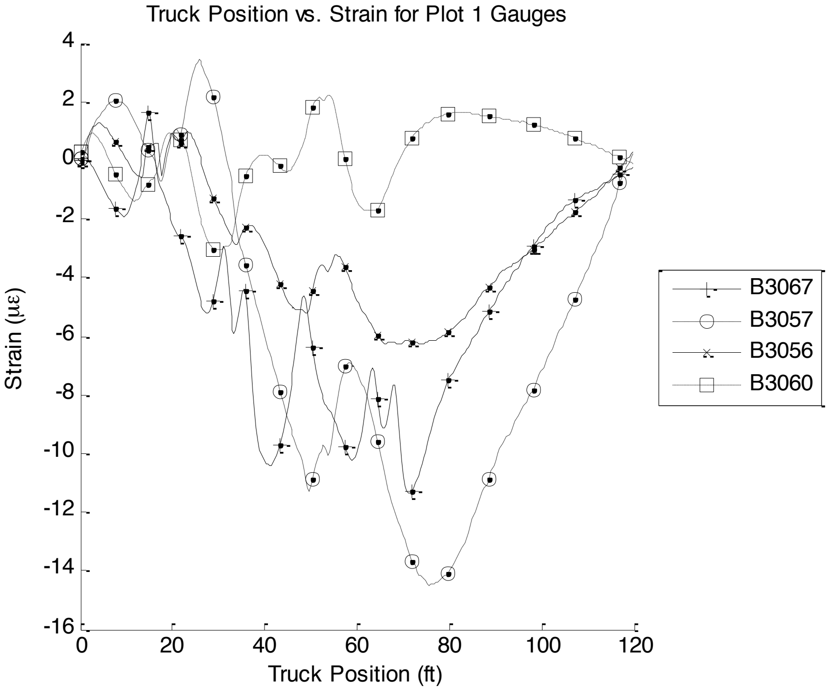


Figure 45 - Strain plots for Node 1 gages during Test 6.

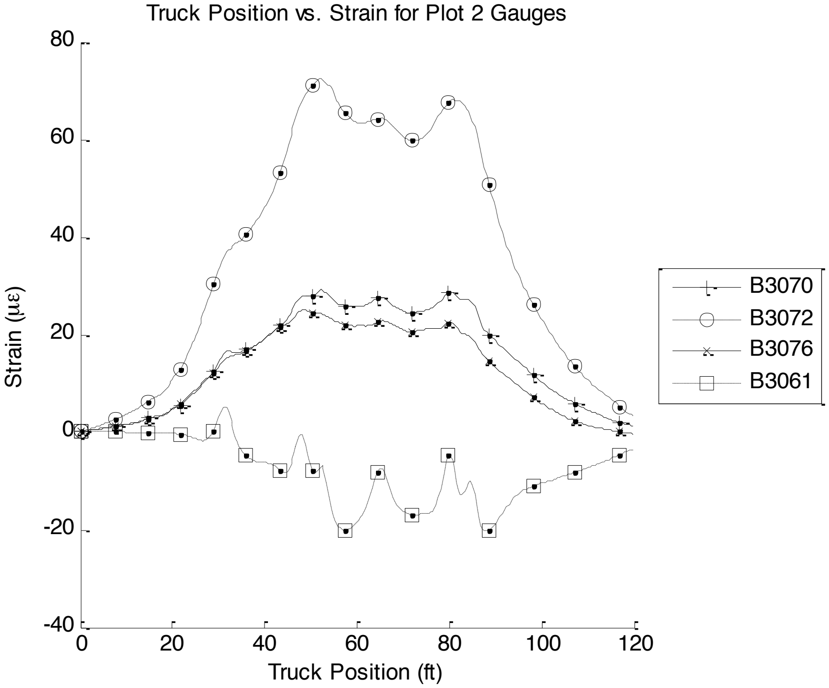


Figure 46 - Strain plots for Node 2 gages during Test 6.

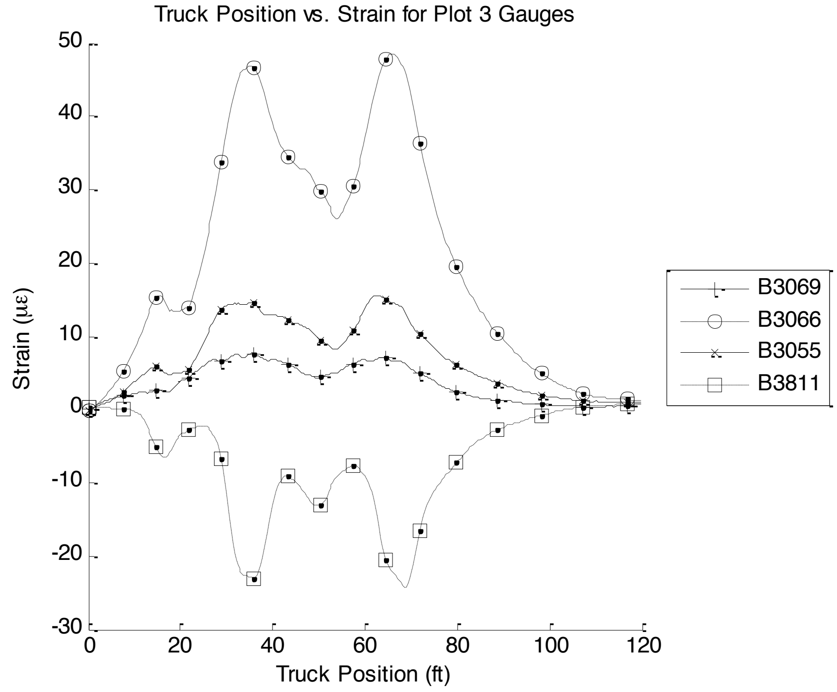


Figure 47 - Strain plots for Node 3 gages during Test 6.

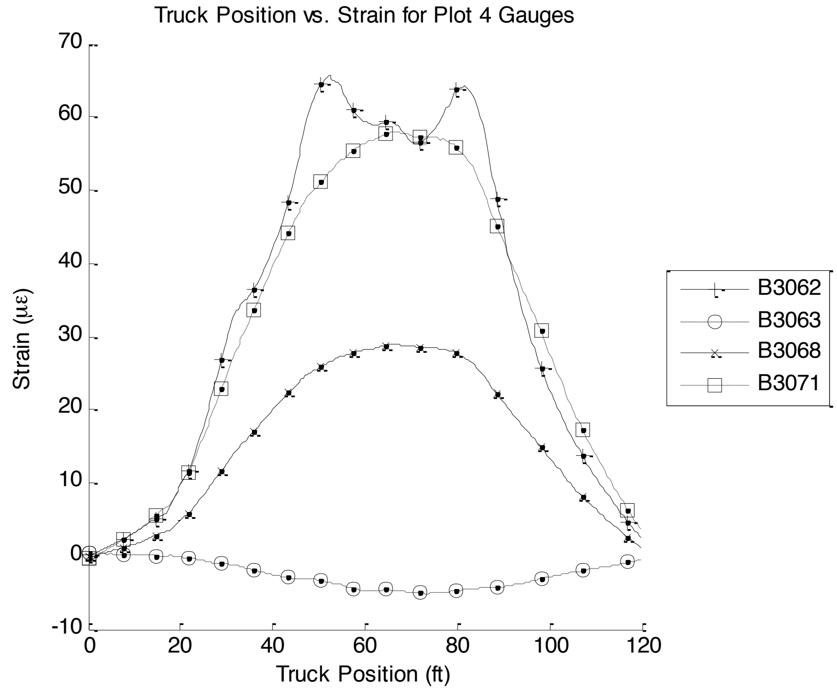


Figure 48 - Strain plots for Node 4 gages during Test 6.

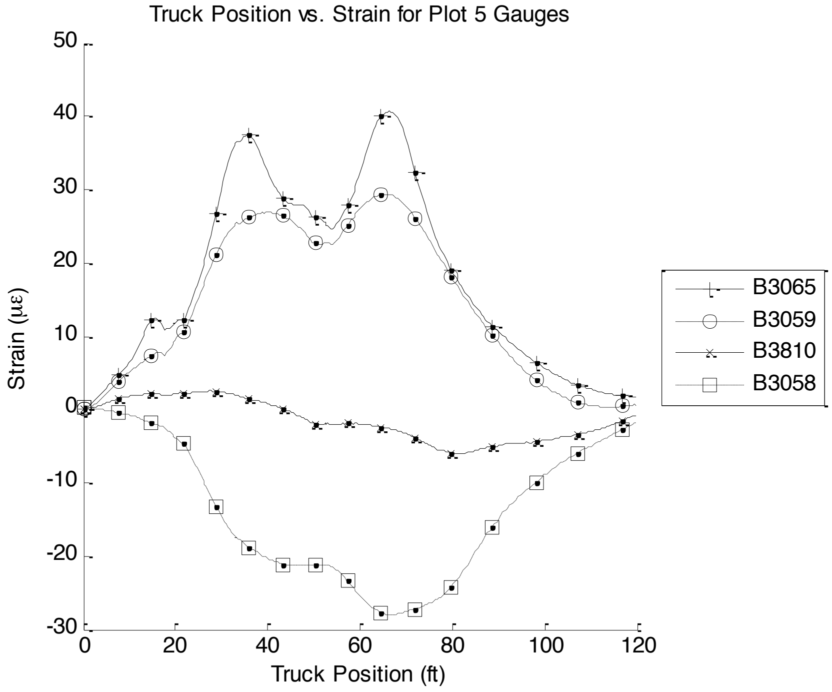


Figure 49 - Strain plots for Node 5 gages during Test 6.

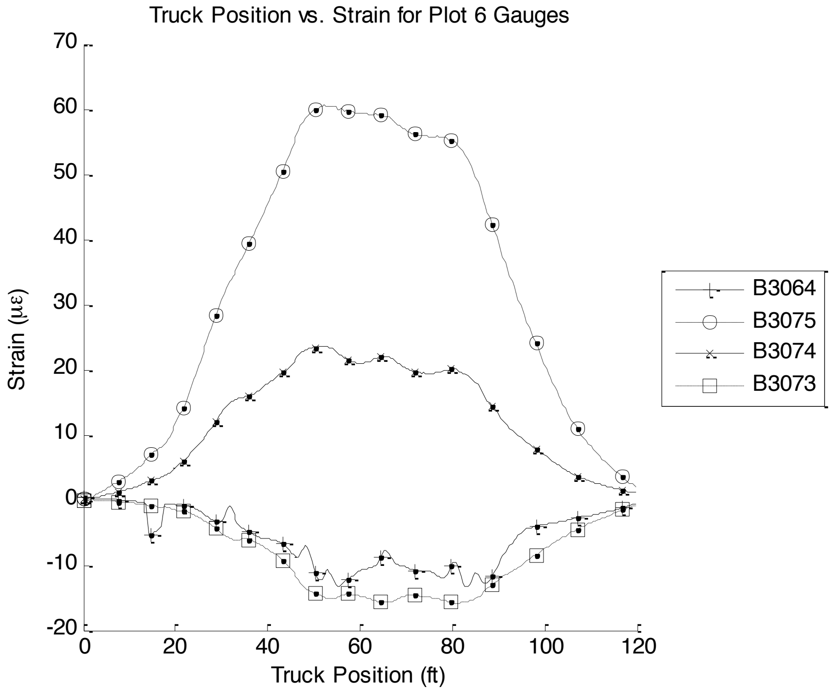


Figure 50 - Strain plots for Node 6 gages during Test 6.

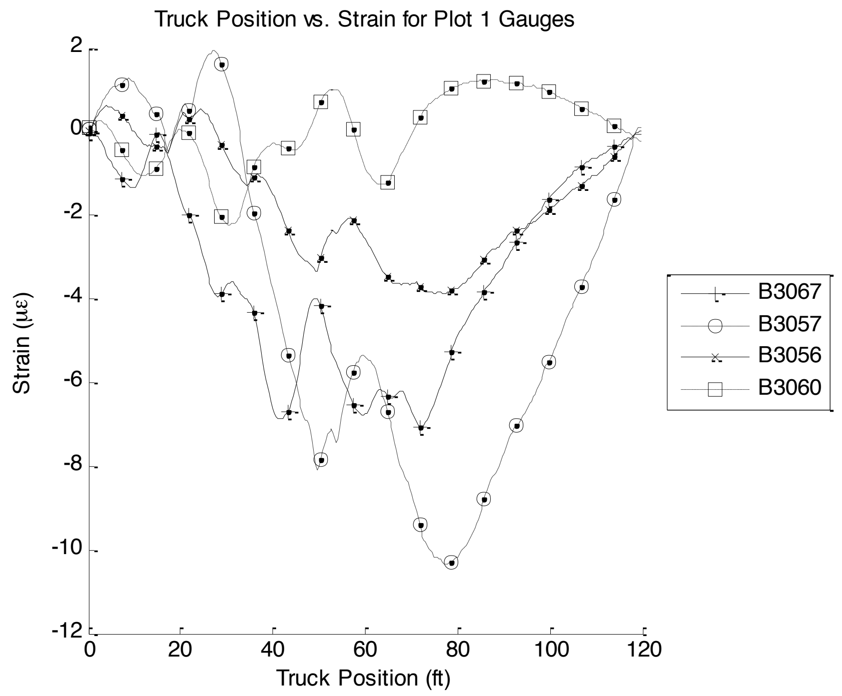


Figure 51 - Strain plots for Node 1 gages during Test 7.

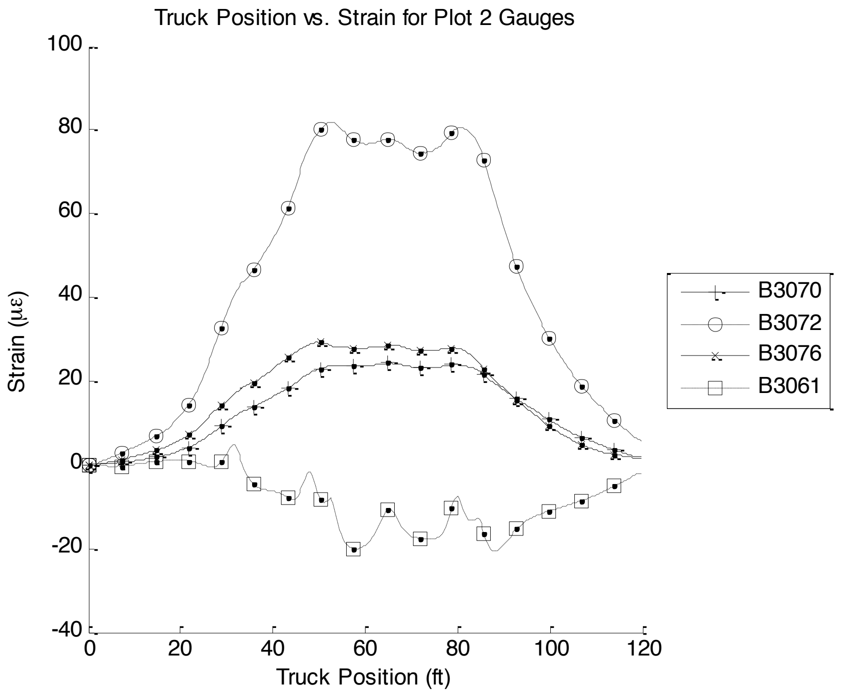


Figure 52 - Strain plots for Node 2 gages during Test 7.

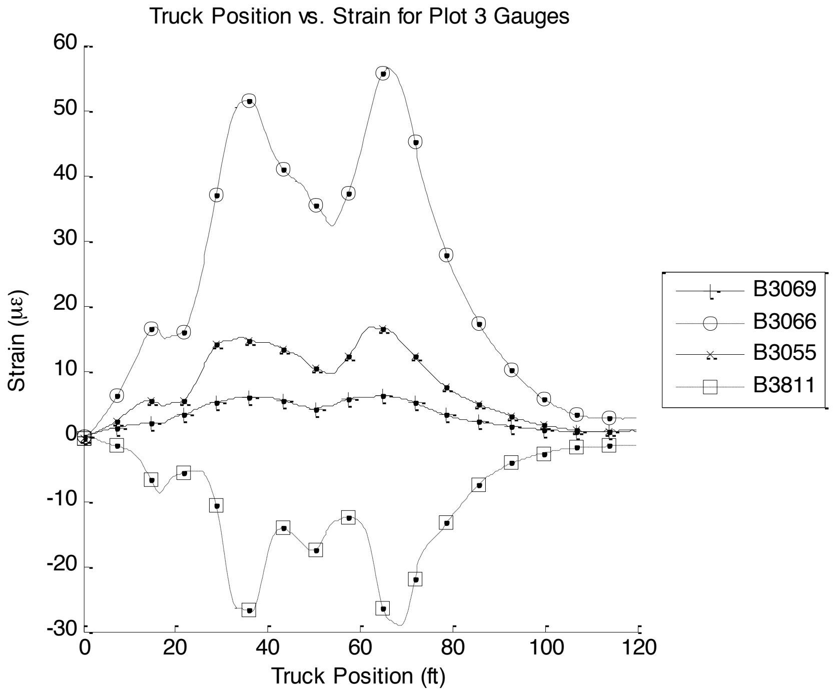


Figure 53 - Strain plots for Node 3 gages during Test 7.

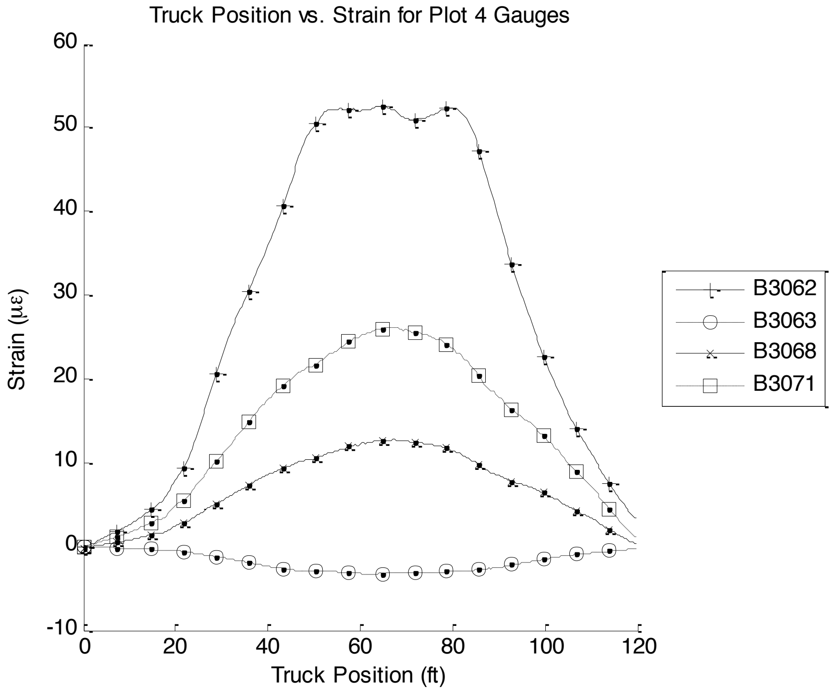


Figure 54 - Strain plots for Node 4 gages during Test 7.

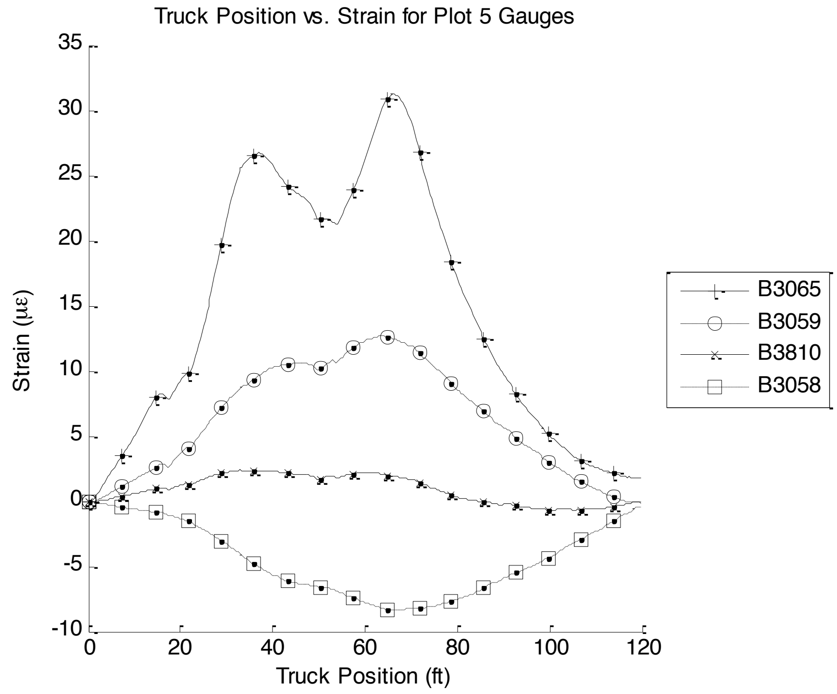


Figure 55 - Strain plots for Node 5 gages during Test 7.

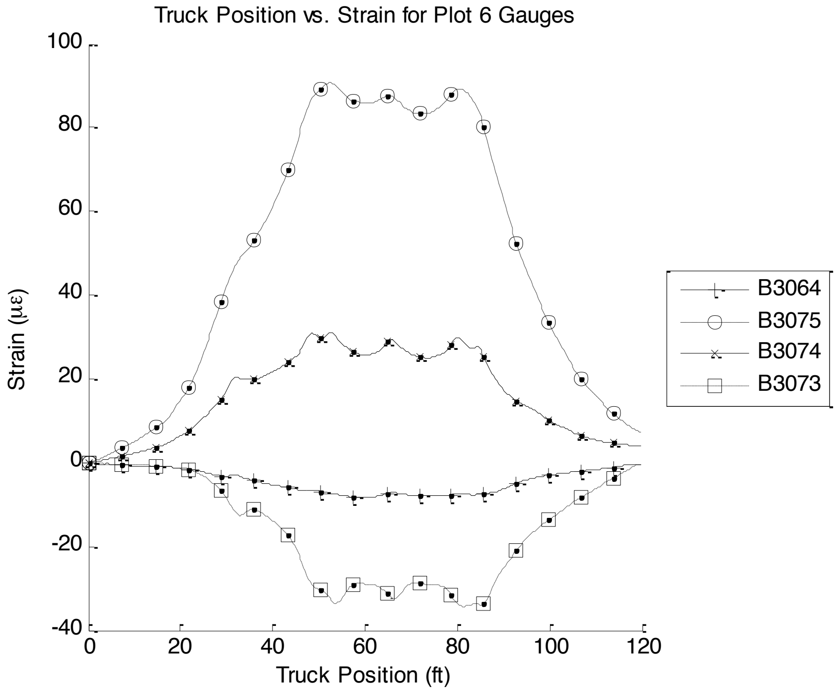


Figure 56 - Strain plots for Node 6 gages during Test 7.

Hastings Bridge (No. 5507)

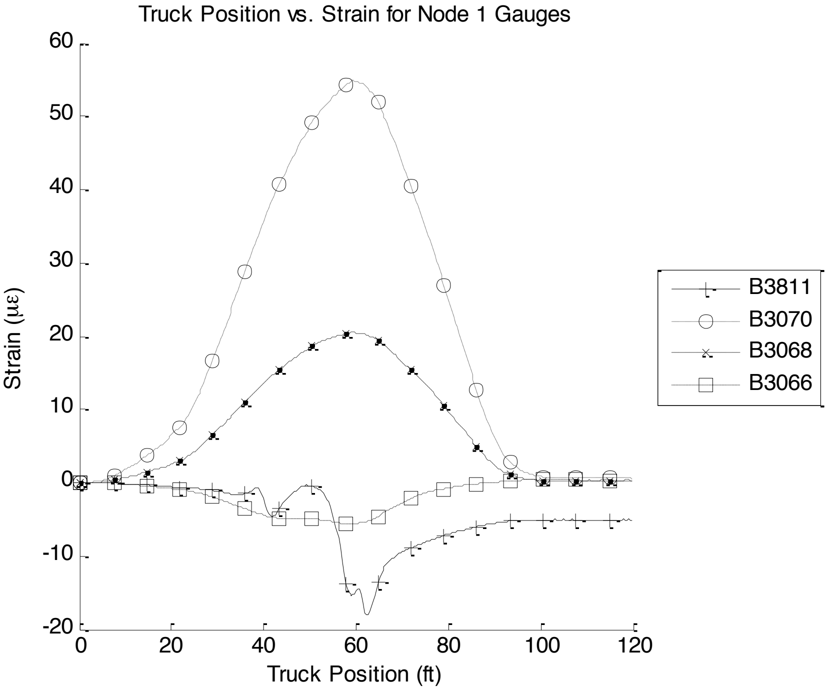


Figure 57 - Strain plots for Node 1 gages during Test 2.

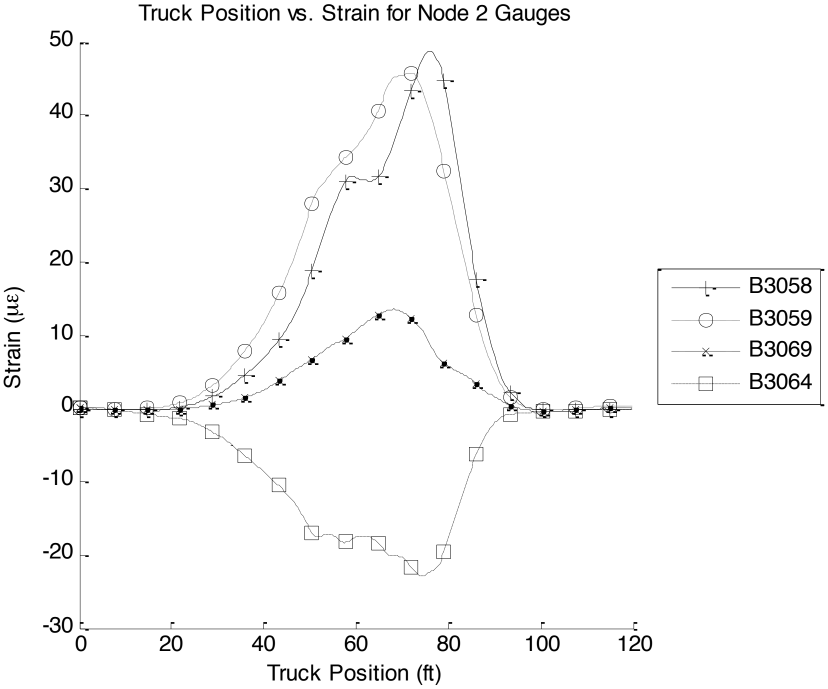


Figure 58 - Strain plots for Node 2 gages during Test 2.

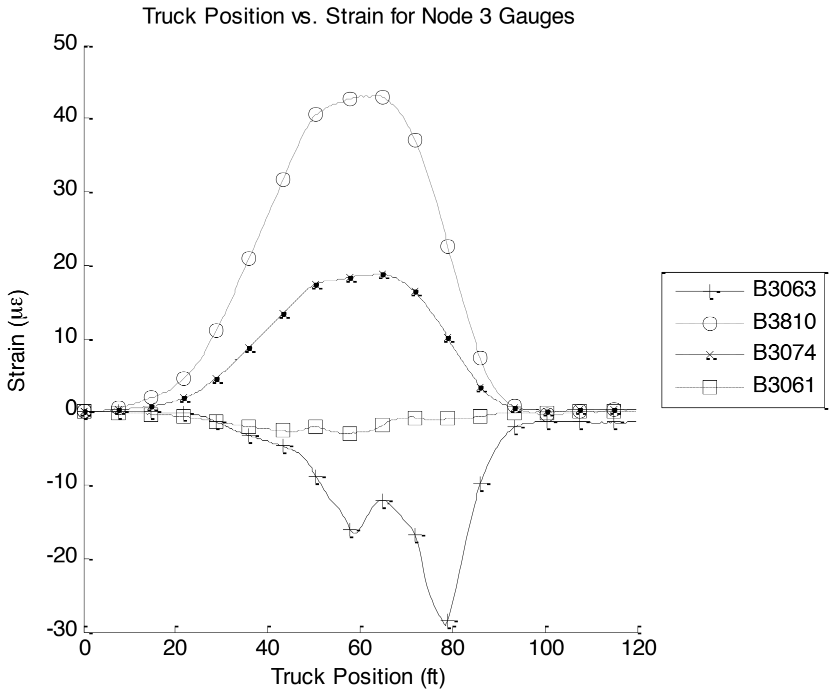


Figure 59 - Strain plots for Node 3 gages during Test 2.

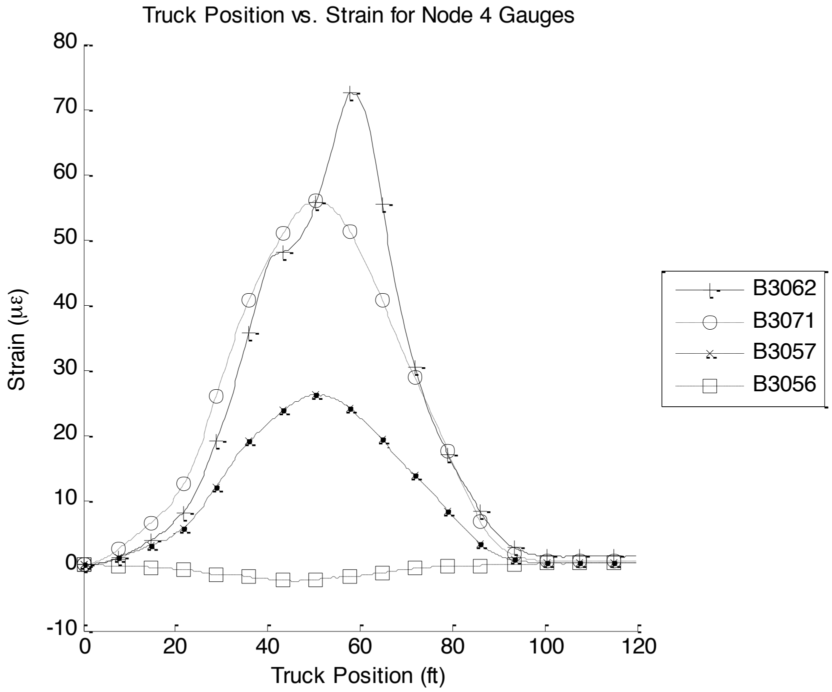


Figure 60 - Strain plots for Node 4 gages during Test 2.

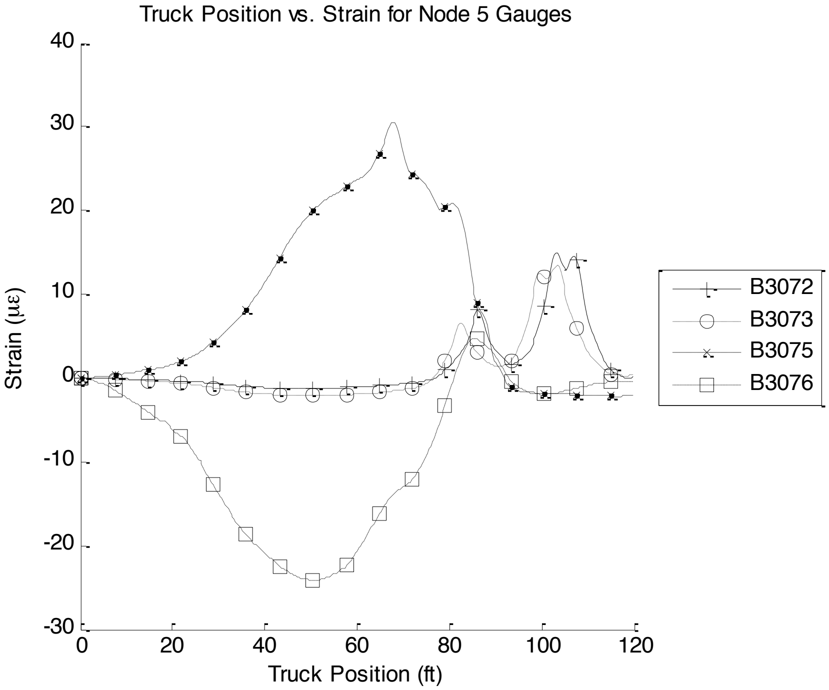


Figure 61 - Strain plots for Node 5 gages during Test 2.

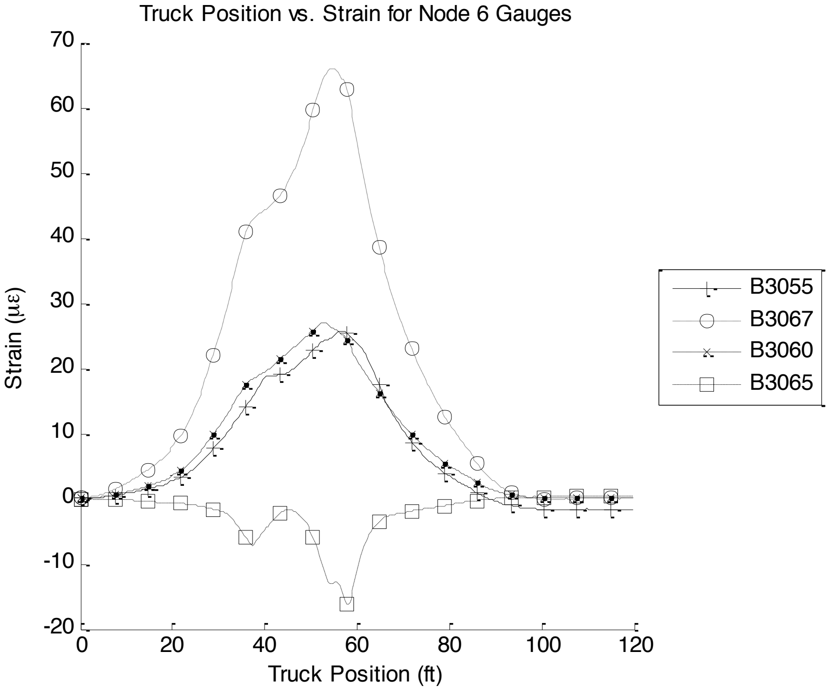


Figure 62 - Strain plots for Node 6 gages during Test 2.

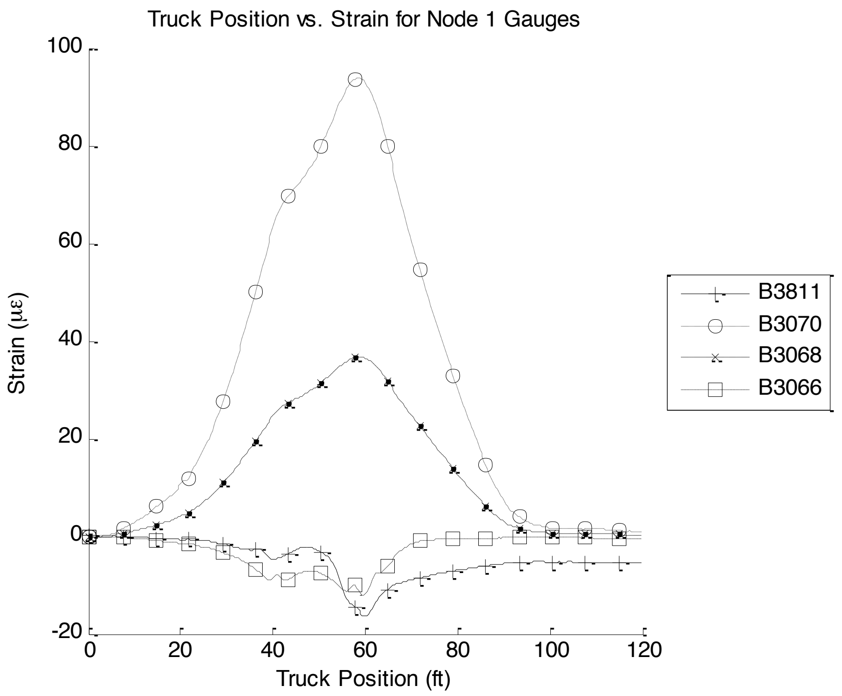


Figure 63 - Strain plots for Node 1 gages during Test 3

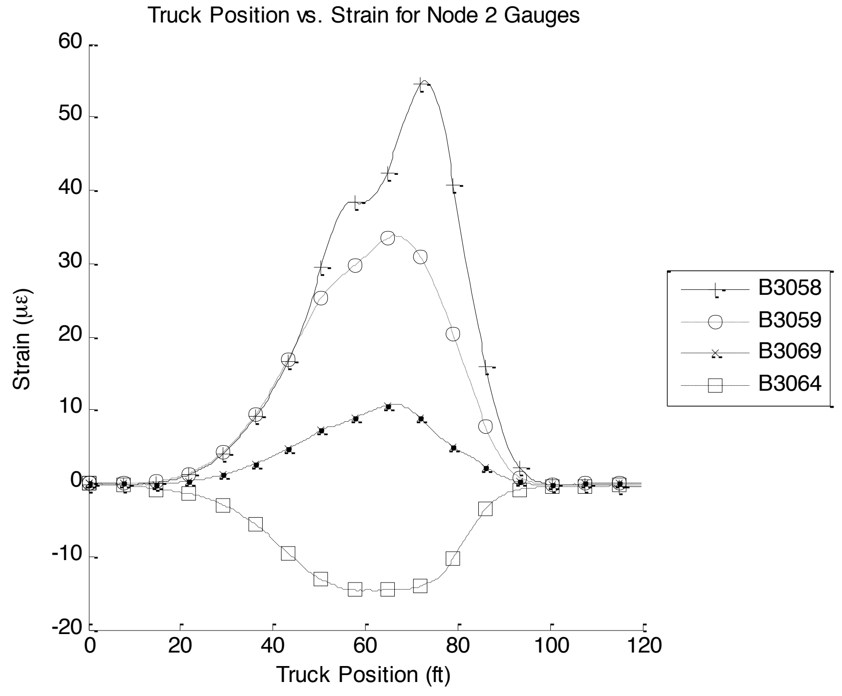


Figure 64 - Strain plots for Node 2 gages during Test 3.

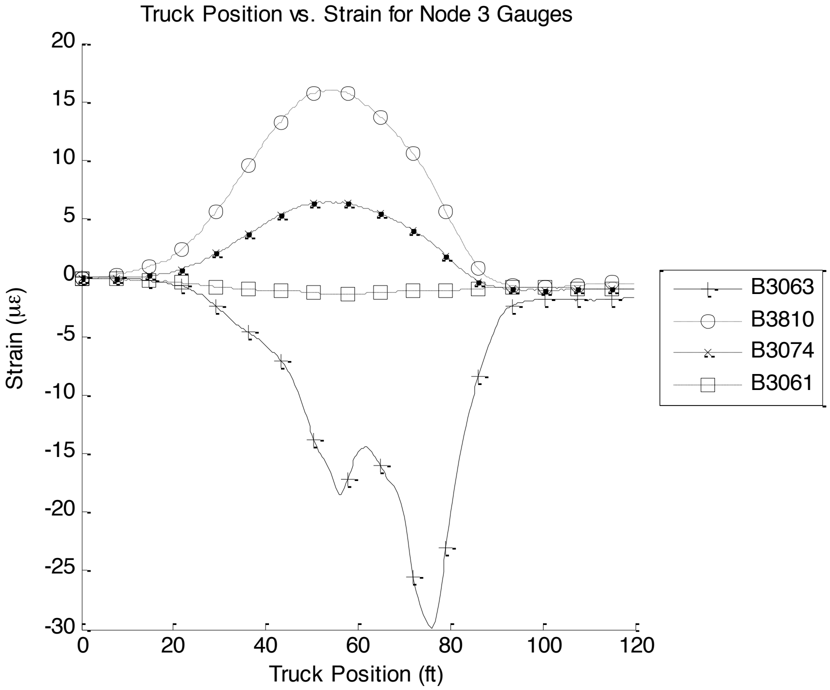


Figure 65 - Strain plots for Node 3 gages during Test 3.

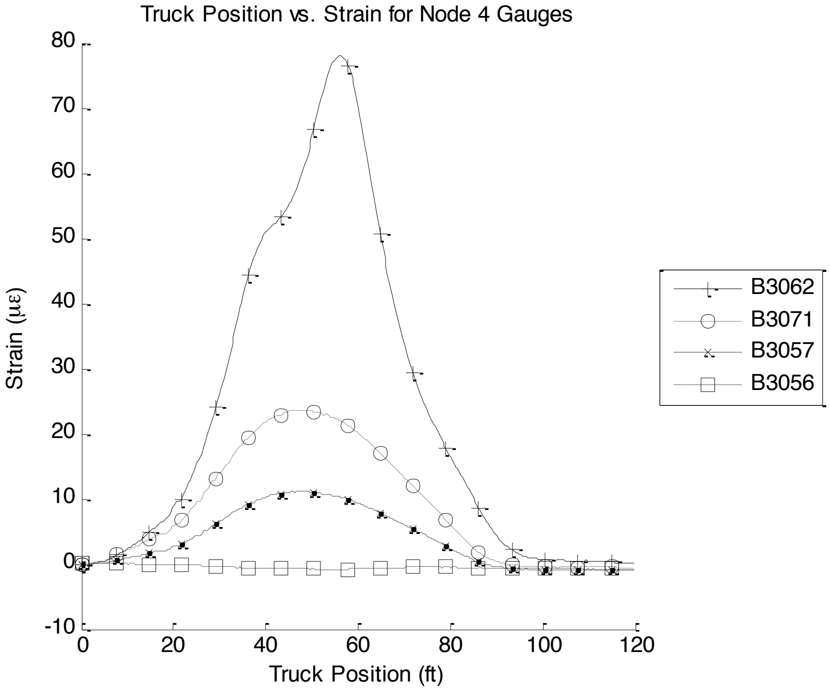


Figure 66 - Strain plots for Node 4 gages during Test 3.

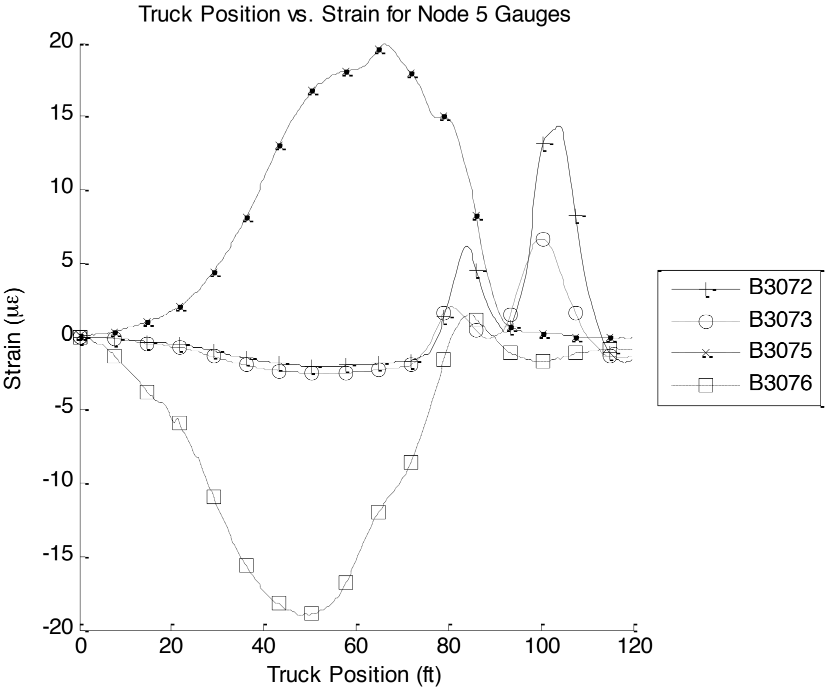


Figure 67 - Strain plots for Node 5 gages during Test 3.

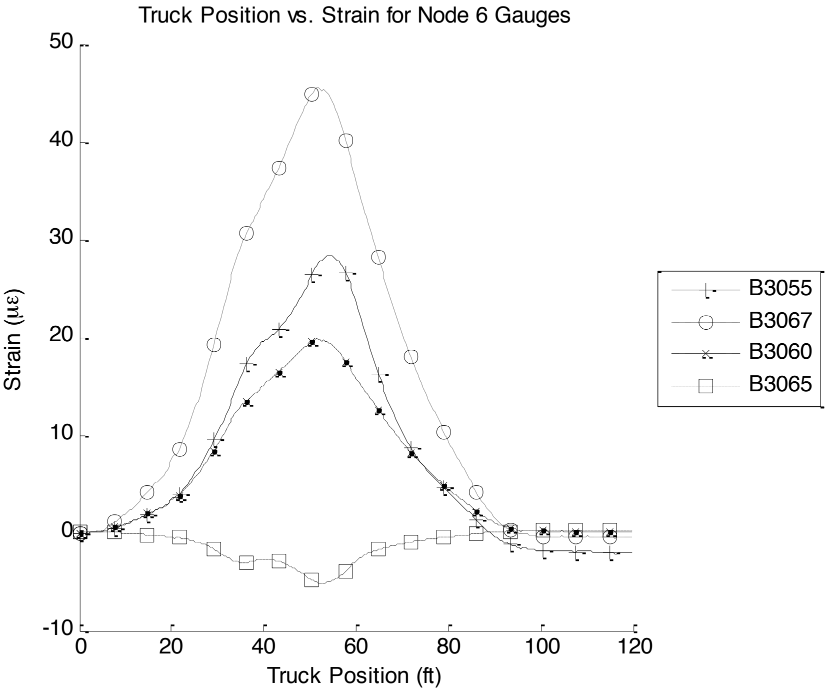


Figure 68 - Strain plots for Node 6 gages during Test 3.

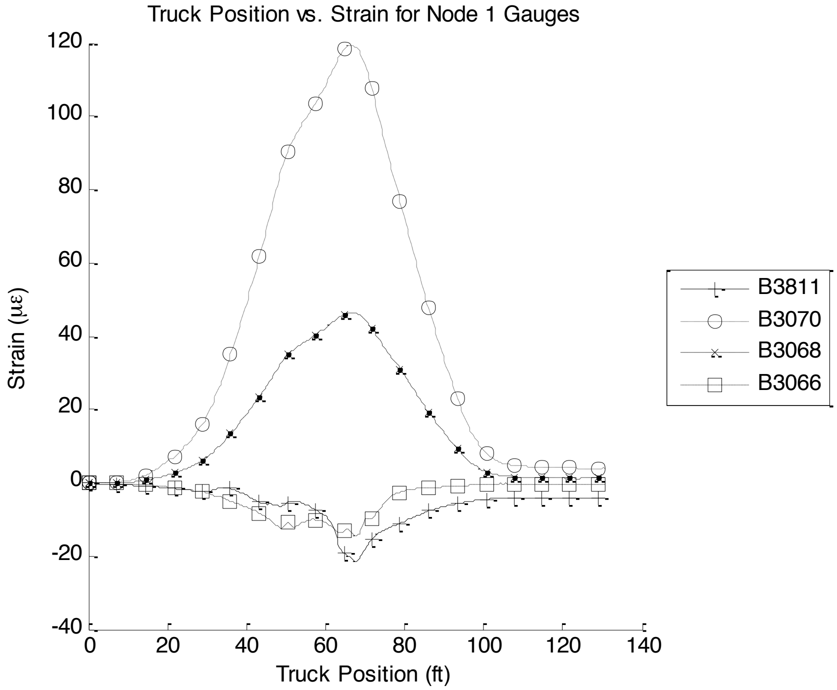


Figure 69 - Strain plots for Node 1 gages during Test 4.

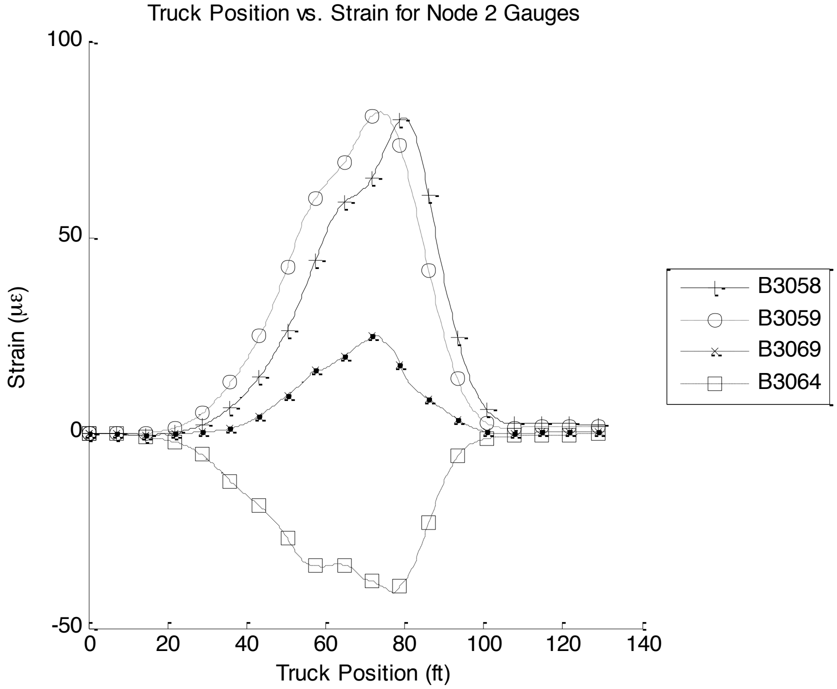


Figure 70 - Strain plots for Node 2 gages during Test 4.

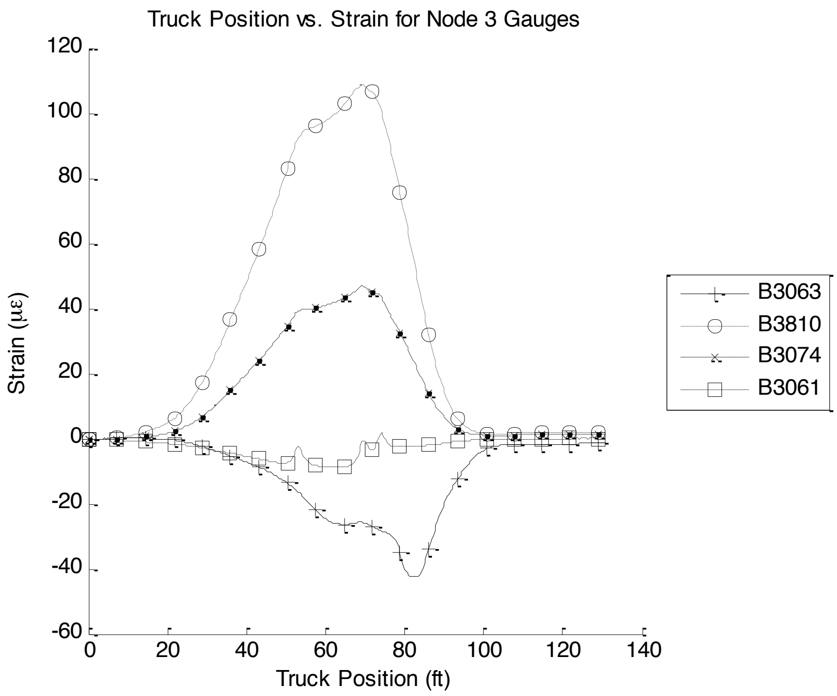


Figure 71 - Strain plots for Node 3 gages during Test 4.

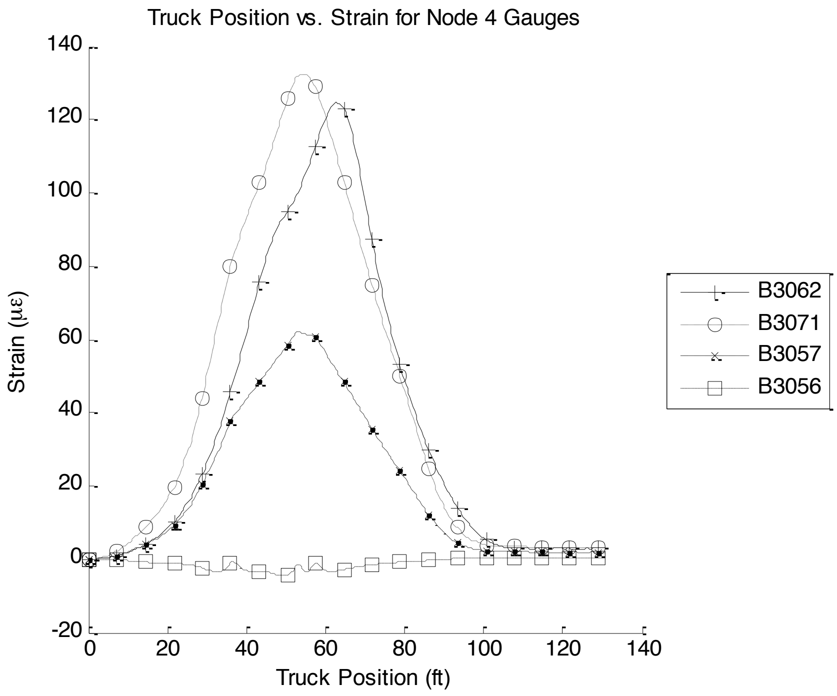


Figure 72 - Strain plots for Node 4 gages during Test 4.

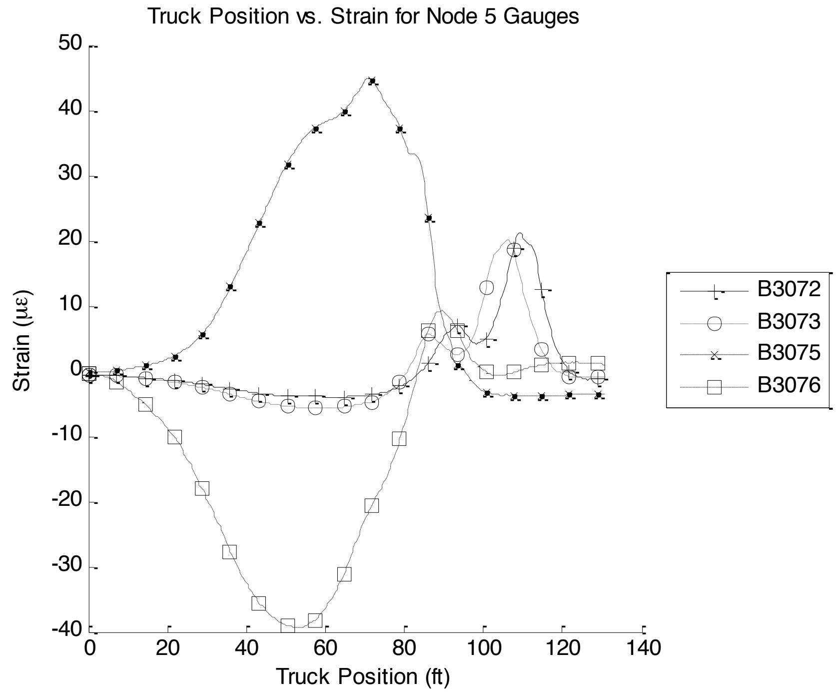


Figure 73 - Strain plots for Node 5 gages during Test 4.

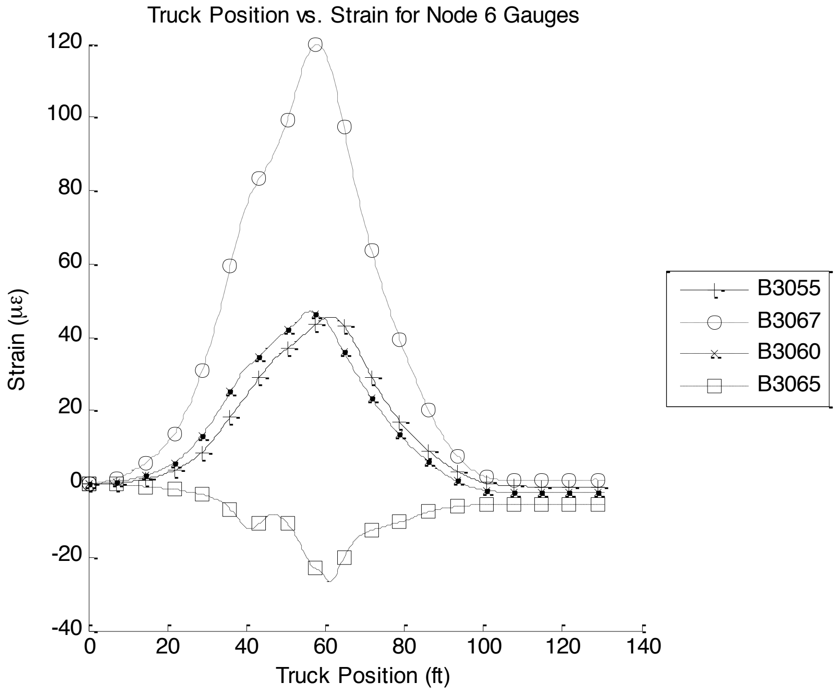


Figure 74 - Strain plots for Node 6 gages during Test 4.

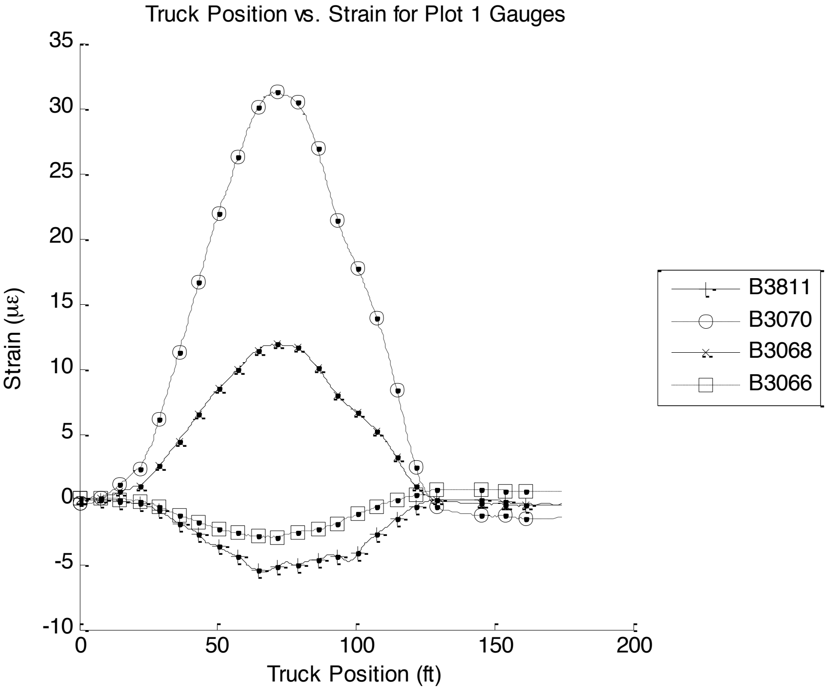


Figure 75 - Strain plots for Node 1 gages during Test 5.

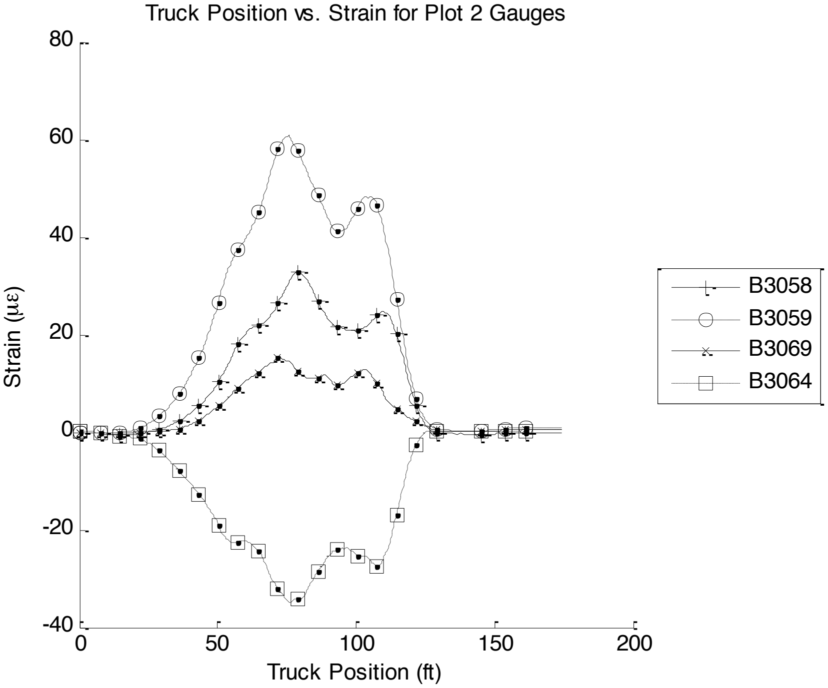


Figure 76 - Strain plots for Node 2 gages during Test 5.

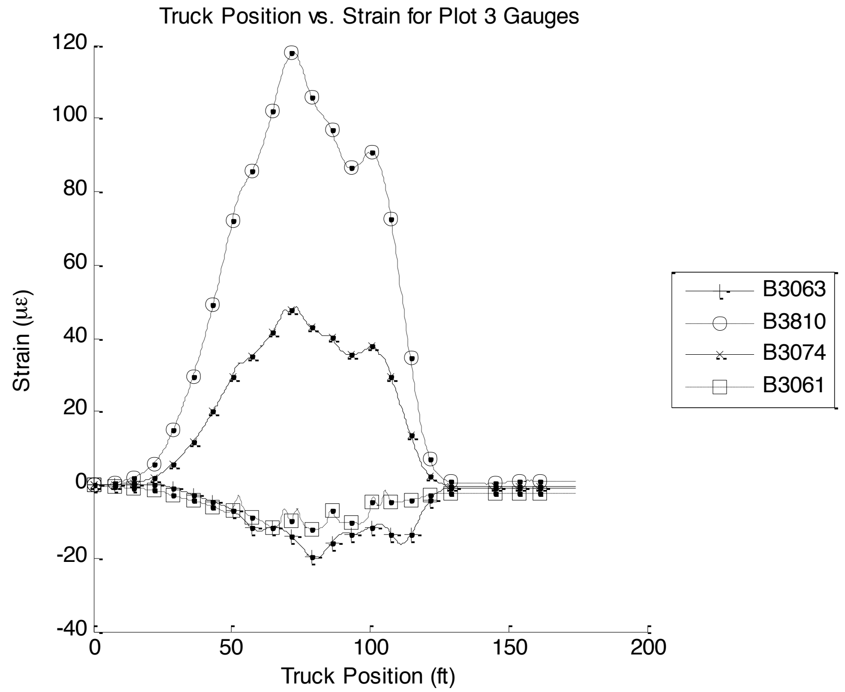


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Figure 77 - Strain plots for Node 3 gages during Test 5.

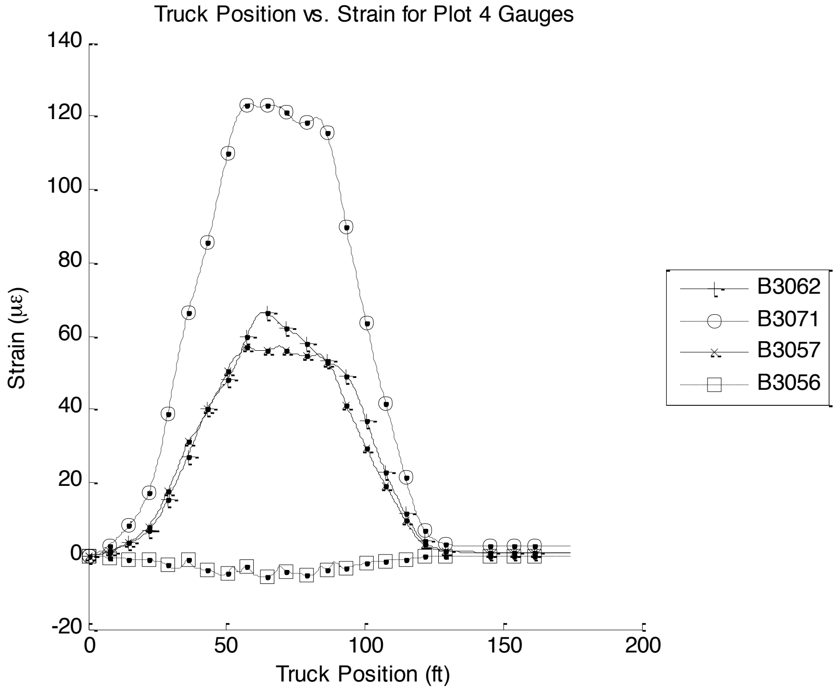

Figure 78 - Strain plots for Node 4 gages during Test 5.

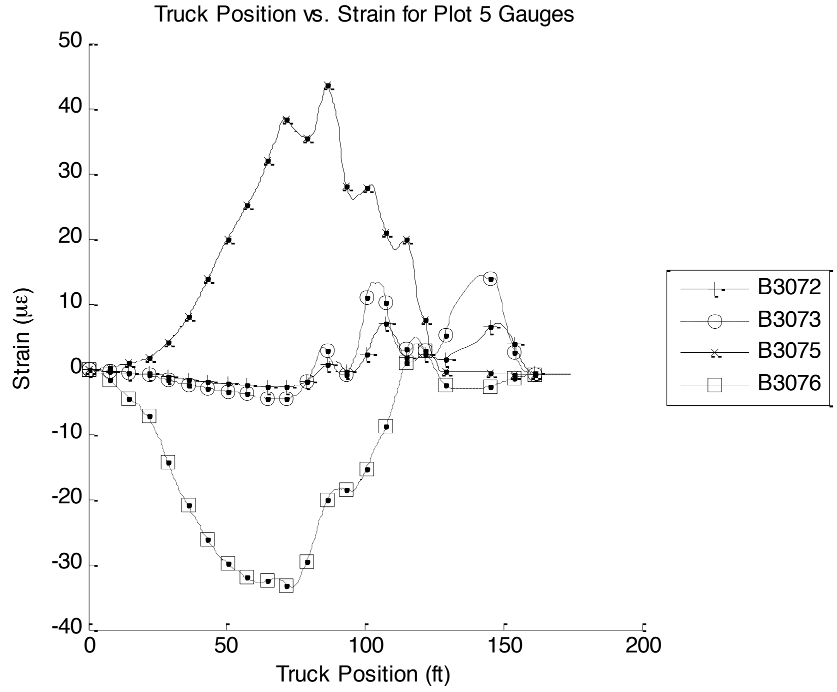

Figure 79 - Strain plots for Node 5 gages during Test 5.

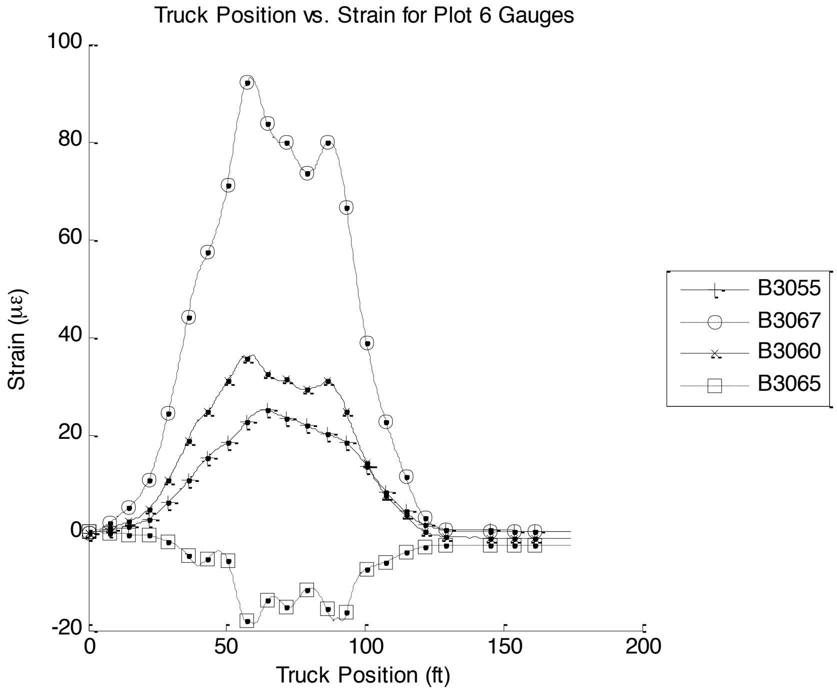

Figure 80 - Strain plots for Node 6 gages during Test 5.

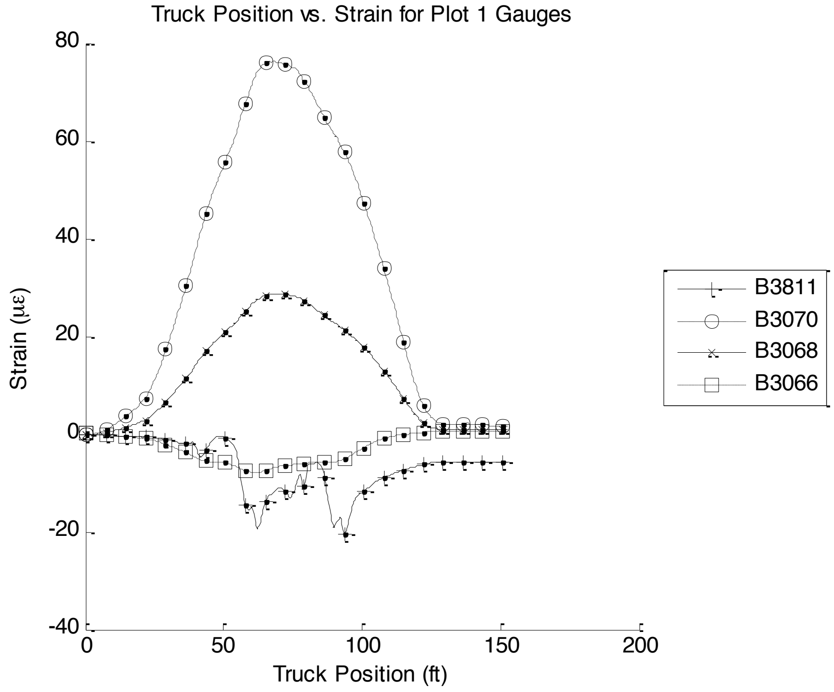

Figure 81 - Strain plots for Node 1 gages during Test 6.

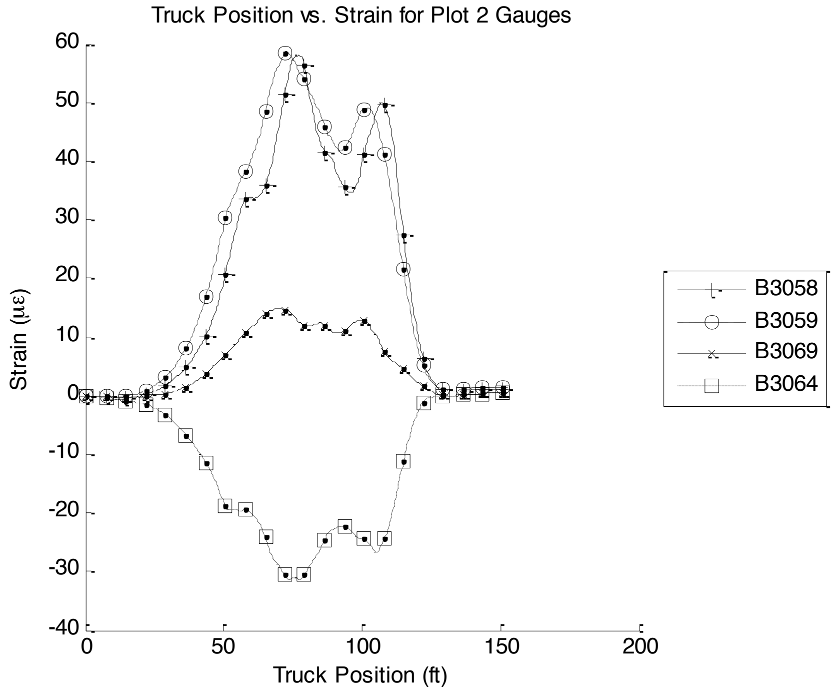

Figure 82 - Strain plots for Node 2 gages during Test 6.

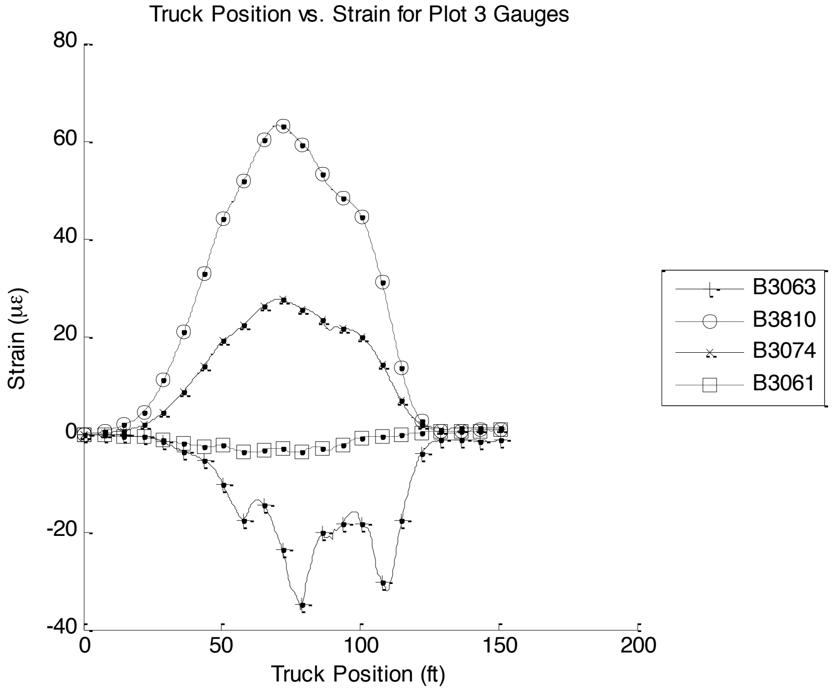

Figure 83 - Strain plots for Node 3 gages during Test 6.

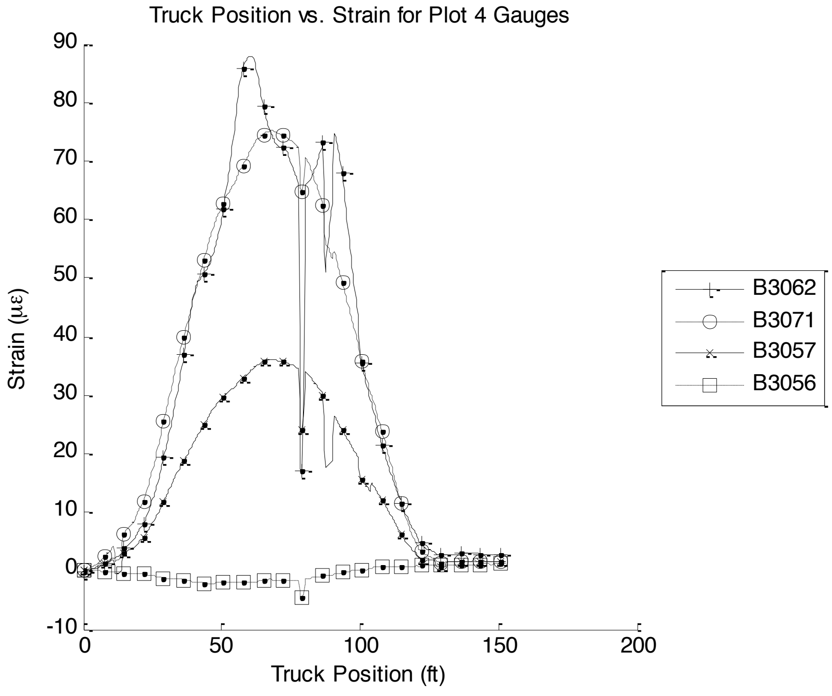

Figure 84 - Strain plots for Node 4 gages during Test 6.

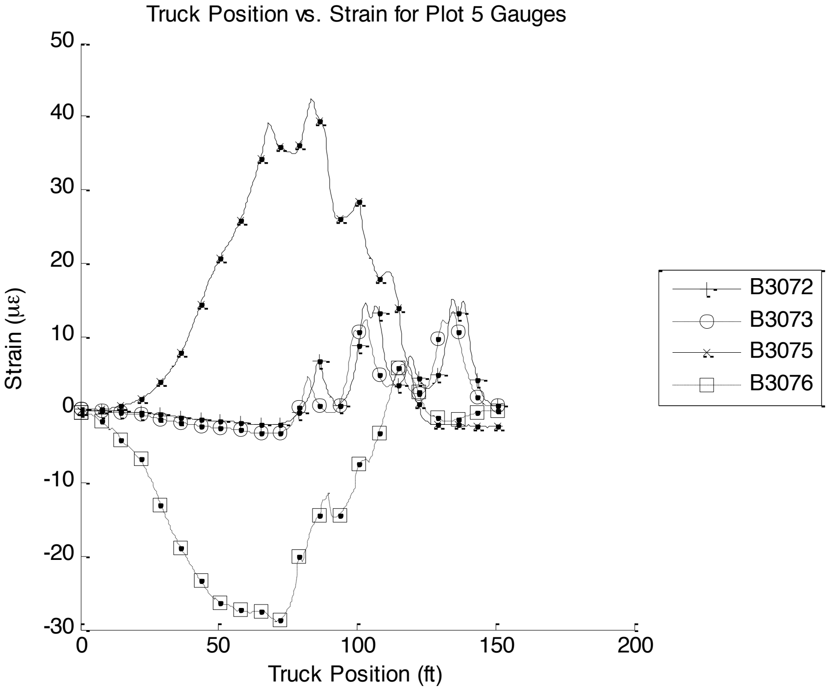

Figure 85 - Strain plots for Node 5 gages during Test 6.

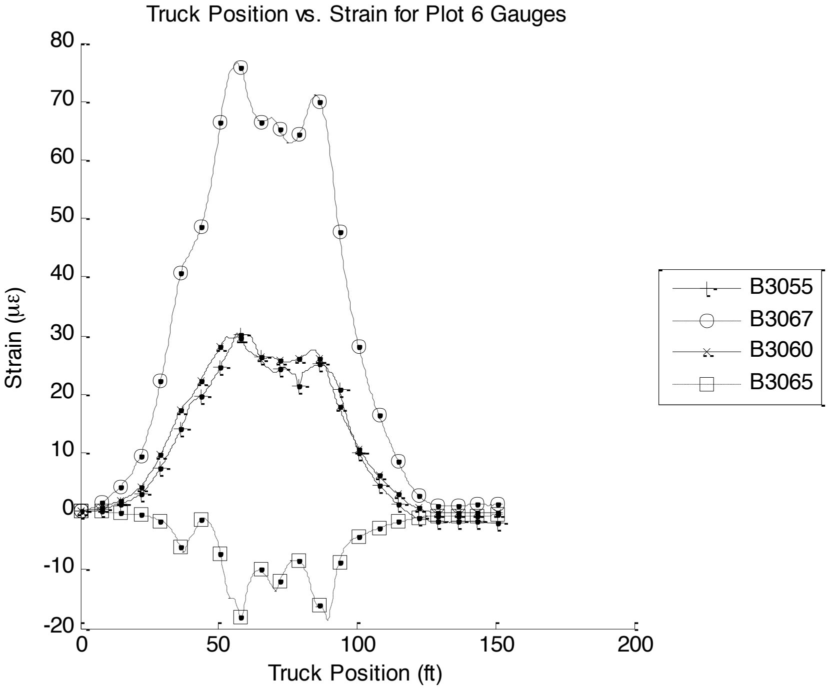

Figure 86 - Strain plots for Node 6 gages during Test 6.

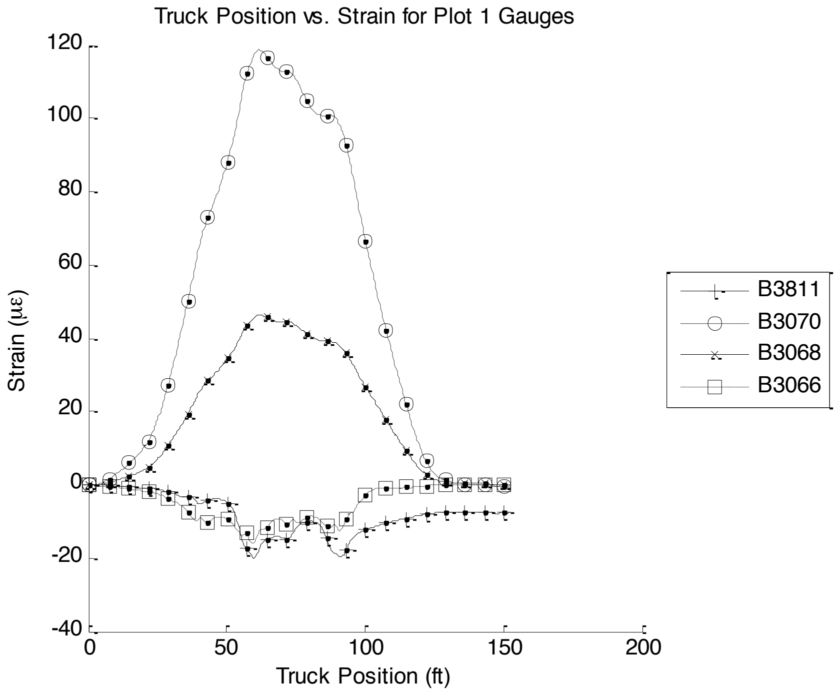

Figure 87 - Strain plots for Node 1 gages during Test 7.

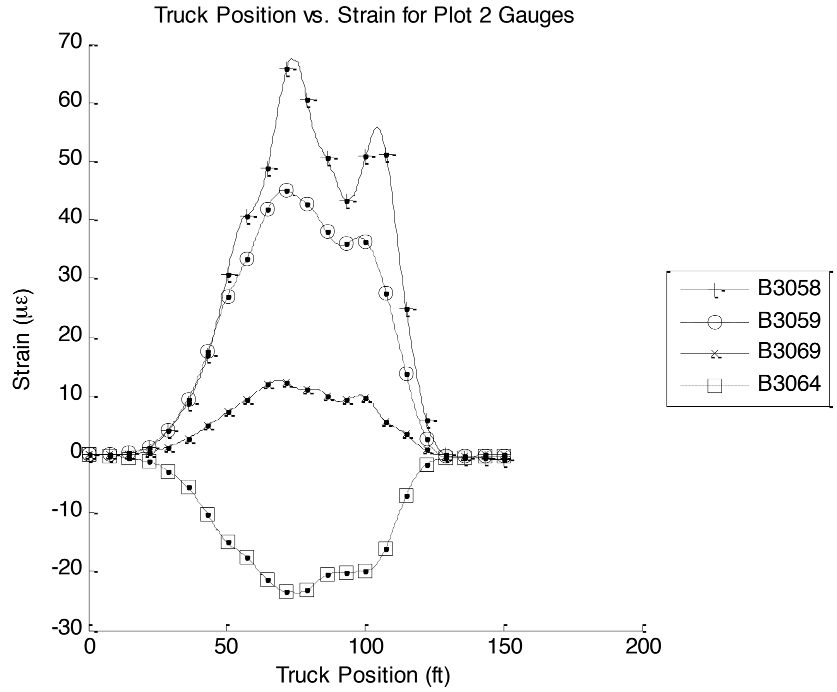

Figure 88 - Strain plots for Node 2 gages during Test 7.

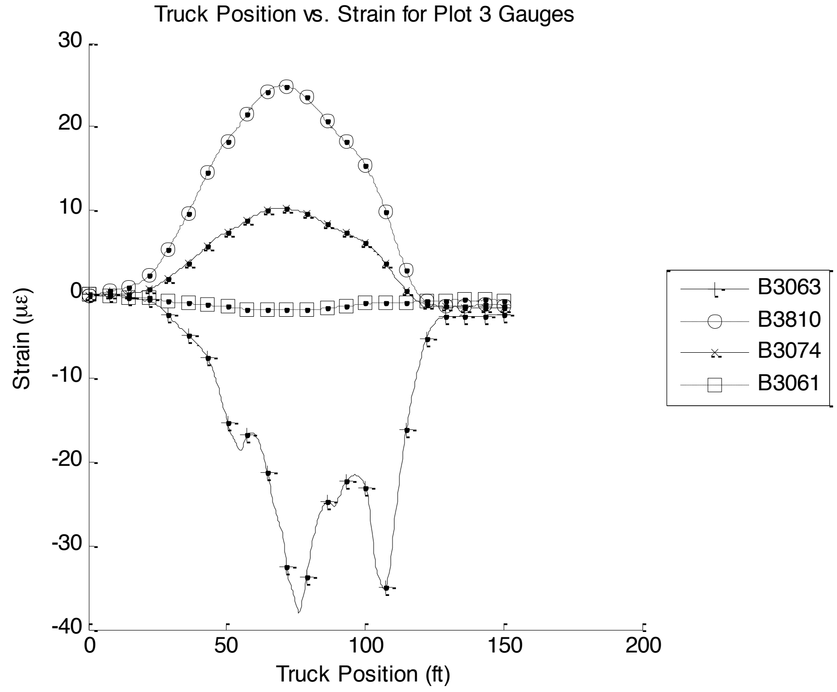

Figure 89 - Strain plots for Node 3 gages during Test 7.

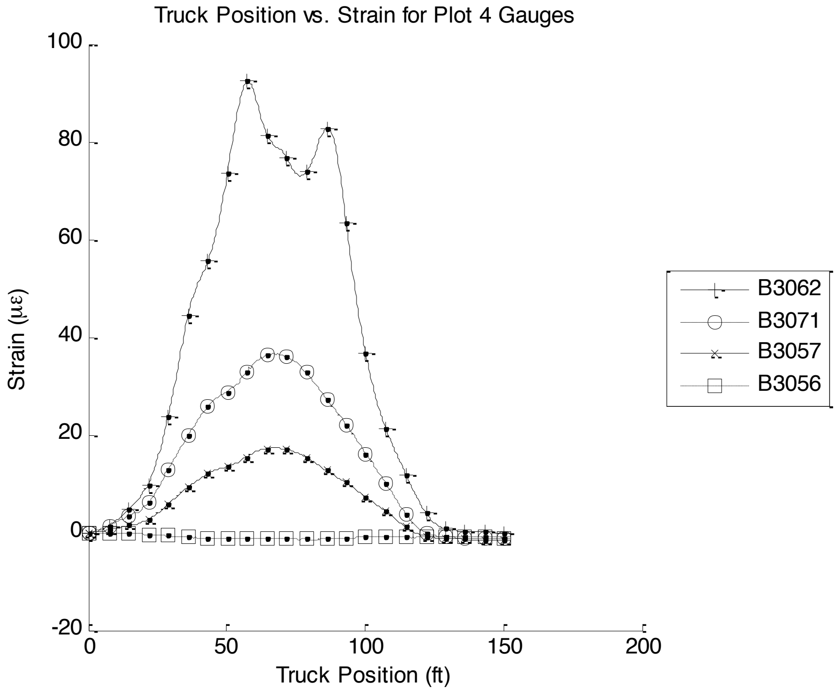

Figure 90 - Strain plots for Node 4 gages during Test 7.

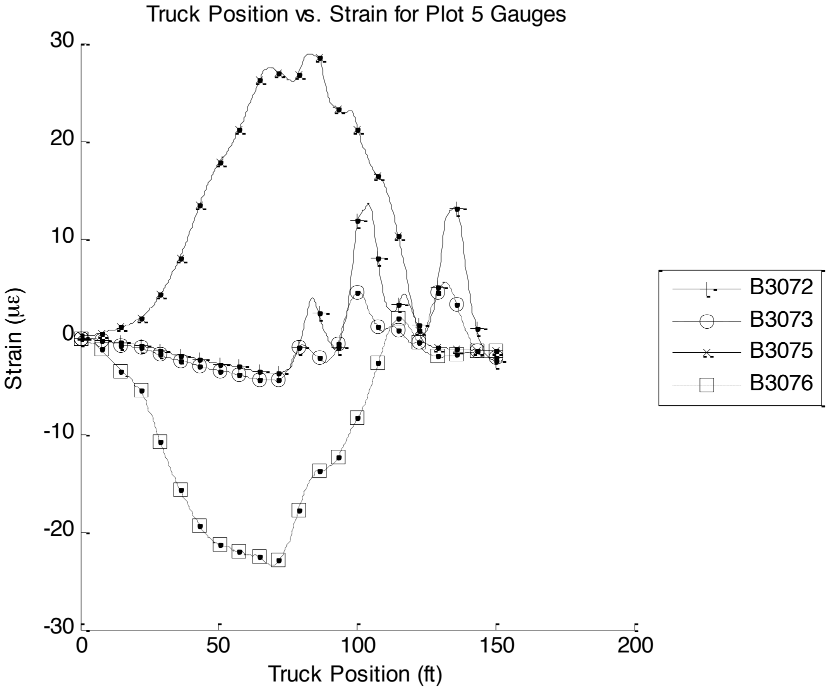

Figure 91 - Strain plots for Node 5 gages during Test 7.

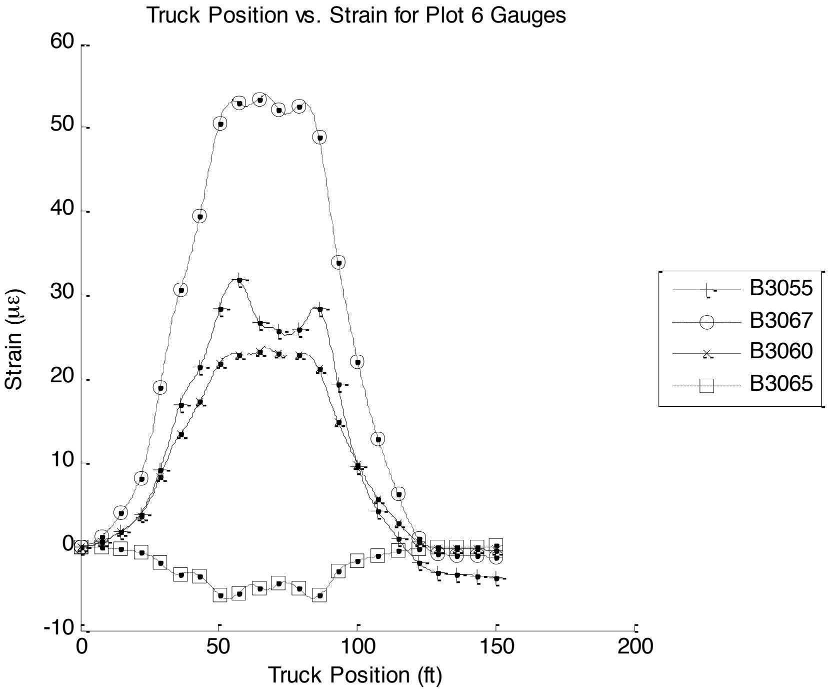

Figure 92 - Strain plots for Node 6 gages during Test 7.