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Live Load Tests of Alabama's HPC Bridge

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Prepared by

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16. Abstract

Live load tests were performed on Alabama's HPC bridge. Load distribution factors, deflections, strains and stresses from the tests were compared with values calculated using design methods from AASHTO *LRFD* and AASHTO *Standard Specifications*. Distribution factors from both AASHTO specifications were found to be conservative. Deflections calculated by assuming all girders deflect the same amount, as suggested by AASHTO *LRFD*, matched best with the measured deflections and were larger than the maximum measured deflections by 20% or less. Stresses at the bottom of the girders near midspan calculated using both AASHTO specifications were higher than the measured values. For interior girders, stresses calculated using AASHTO *LRFD* matched best with the test results. For exterior girders, the AASHTO *LRFD* equations and the lever rule of the AASHTO *Standard Specifications* produced approximately the same results. AASHTO *LRFD* also requires a special analysis to determine the distribution factor for exterior girders by assuming the bridge cross section deflects downward and rotates as a rigid cross section. Overall, using the AASHTO *LRFD*, except for the special analysis, provided calculated stresses that were larger than the measured stresses by less than 40%. Use of the special analysis for exterior girders was not justified by the test results for this bridge.

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CHAPTER ONE

INTRODUCTION

BACKGROUND

Bridges with precast, prestressed concrete girders and reinforced concrete decks are common in new bridge construction due to a lower initial cost relative to other bridge systems and relatively low maintenance costs through the life of the structure. In recent years the Federal Highway Administration (FWHA) has stimulated the development and implementation of High-Performance Concrete (HPC). The use of HPC in bridge design offers a way to utilize higher compressive strength while ensuring long-term durability in these already popular bridges. Increased span lengths and fewer structural components result in cost savings during construction, while the bridge's longer service life results in lower life-cycle costs.

An HPC bridge was constructed in Alabama as part of a FHWA program to fund research of HPC, promote construction of HPC bridges, and host Showcases in various regions across the United States. The purpose of each HPC Showcase was to disseminate the latest knowledge on technological developments and applications of HPC in bridge construction for the promotion of more durable and cost effective concrete construction. Alabama's Showcase was held at Auburn University, and the technical presentations are collected in the *Participant Notebook* (1999). A summary of the Showcase events was provided by Stallings and Mayo (1999). A discussion of the construction of

Alabama's HPC bridge, concrete and materials research results, and data quantifying the strength and durability of the concrete used in constructing the bridge are given by Glover and Stallings (2000). An investigation of camber and prestress losses of the AASHTO BT-54 girders used in the HPC bridge is documented by Stallings and Eskildsen (2001). Here, results of live load tests of the HPC bridge are presented.

OBJECTIVE AND SCOPE

The primary objectives of this report are to document the live load tests performed on Alabama's HPC bridge, to present the results of those tests, and to provide comparisons of the field measurements and values calculated using standard AASHTO procedures. Measurements of strains in the girders and deck were made using instrumented bars that were placed prior to casting the concrete. Vertical deflections of the girders were also measured. Measured deflections, strains in the girders and longitudinal strains in the deck are compared to values calculated using procedures from the AASHTO *Standard Specifications* (1996) and from the AASHTO *LRFD Specifications* (1998). Measurements of strains in the deck reinforcement transverse to the roadway are also reported. Comparison of the strains in the transverse reinforcement to analytical results is beyond the scope of the project.

CHAPTER TWO

DESCRIPTION OF BRIDGE AND INSTRUMENTATION

LOCATION OF HPC BRIDGE

The bridge over Uphapee Creek on Alabama Highway 199 in Macon County, Alabama was the subject of the live load tests. The Uphapee Creek Bridge is a replacement for a bridge that was built in the 1940's that had suffered from streambed scour resulting from sand and gravel mining downstream. The Uphapee Creek Relief Bridge and Bulger Creek Bridge were also a part of the HPC construction and research projects, but live load tests were not performed on those bridges. A map showing the location of the bridges is provided in Figure 2.1.

DESCRIPTION OF HPC BRIDGE

The Uphapee Creek Bridge consists of seven, simply supported, prestressed concrete girder spans with a 7 in. thick cast-in-place concrete deck. The overall length of the bridge is 798 ft. There are five AASHTO BT-54 girders per span, spaced at 8.75 ft, giving a 40 ft wide roadway. The girders are on neoprene bearings supported by reinforced concrete bents on drilled shafts. One span of the bridge was instrumented and subjected to live load tests. This was the span between Bent 5 and Bent 6 on the North side of Uphapee Creek. The overall length of the span is 114 ft, and the length between the centerlines of the bearings is 112.25 ft. Plan and elevation views of the bridge and a typical cross-section taken from the construction drawings are shown in Figure 2.2 and 2.3. A more detailed plan view is shown in Figure 2.4.

HPC GIRDERS

The AASHTO BT-54 girders are 54 inches deep and are pretensioned with 42 strands. The strands are 0.6 in. diameter, low-relaxation, 270 ksi, 7-wire steel strand. Details of the girder reinforcement are shown in Figures 2.5 and 2.6. The compressive strength of the girder concrete was specified as 8000 psi at release and 10,000 psi at 28-days. Strength test results were higher than these specified values for each of the girders in the instrumented span. The modulus of elasticity of the girder concrete was measured in 32 individual tests of 4 in. diameter, match-cured cylinders at ages from release to 56-days. Due to the high release strength, any gain in modulus of elasticity with age was not obvious relative to the scatter in the measured values. A modulus of elasticity of the girder concrete of 5,740,000 psi was determined as the average of all 32 individual test results. Results of the individual tests are reported by Glover and Stallings (2000).

HPC DECK

Cast-in-place substructure and superstructure HPC was used on all three bridges in this project. The specified compressive strength of the cast-in-place concrete was 6,000 psi. Design calculations for the cast-in-place concrete members were based on a compressive strength of 4,000 psi. While the higher compressive strength of the cast-in-place concrete was not fully utilized in the design, HPC was specified to provide enhanced performance and durability characteristics.

The modulus of elasticity of the deck concrete was measured for three 6 in. diameter cylinders at each age of 7, 28, 56 and 91 days. The three specimens at each age were sampled from three different deck pours made during construction of the Uphapee Creek Relief bridge. The modulus of elasticity illustrated a clear increase with age. At the age of 91 days, the average of the three test results was 6,650,000 psi. This value is used in this report as the modulus of elasticity of the deck concrete.

INSTRUMENTATION

Strains were measured with strain gages that were placed in the girders and in the deck prior to casting the concrete. Electrical resistance and vibrating wire strain gages were used. Locations of the gages are illustrated by Figures 2.4, 2.7 and 2.8.

A plan view of the deck showing the general location of the strain gages in the deck, overall geometry, and girder numbering is shown in Figure 2.4. Electrical resistance gages were used in the deck. Those gages were mounted on pieces of 0.5 in. diameter reinforcing bar 4 ft in length. These pieces of reinforcing bar were installed parallel and transverse to the girders near the top and bottom of the deck slab as shown in Figures 2.4 and 2.7. Figure 2.7 shows a transverse cross section through the deck slab. The location of the #4 bars shown matches the location of the instrumented bars that were oriented parallel to the girders. The location of the #5 bars shown matches the location of the instrumented bars that were oriented perpendicular to the girders.

The girders were instrumented with vibrating wire and electrical resistance strain gages. Locations of the gages in the girder cross section are shown in Figure 2.8. Vibrating wire gages were installed in the bottom and top flange of each girder at midspan and in Girders 3, 4, and 5 at the quarter span. The vibrating wire gages were supplied by the manufacture attached to a length of 0.5 in. diameter reinforcing bar approximately 4 ft long. Electrical resistance gages were mounted on these bars for the gage locations in the bottom girder flanges at midspan.

Vertical deflections were measured at midspan of each girder and at quarter span for Girders 3, 4 and 5. These vertical deflections were measured using deflectometers that were calibrated to determine the deflection to the nearest 0.01 in. The deflectometers were attached to the bottom of the girder and measured the relative displacement between the bottom of the girder and the ground underneath the bridge. These vertical displacements include compression of the neoprene bearings and deformation of the bridge piers, but are considered here to be estimates of the girder deflection.

Two separate systems were used for data acquisition and storage. All vibrating wire strain gage data was acquired and stored using a Campbell Scientific CR-10X. These measurements were made only for static loading. All deflection and electrical resistance strain gage measurements were made using a MEGADAC 3108 high-speed data acquisition system.

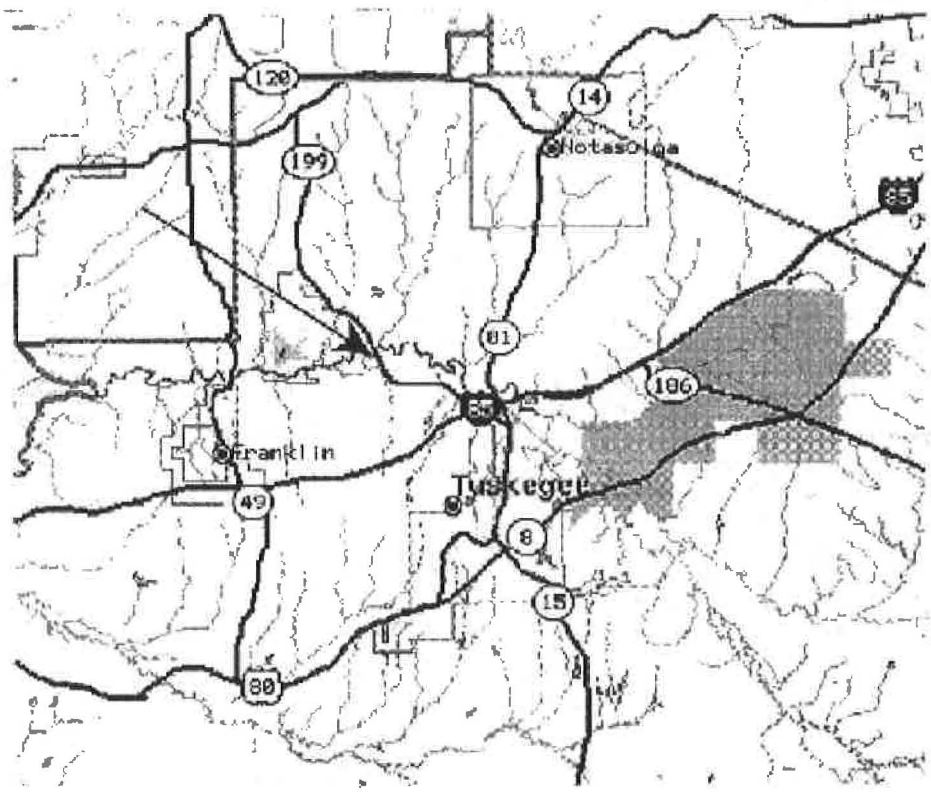


Figure 2.1. Location of HPC Bridges

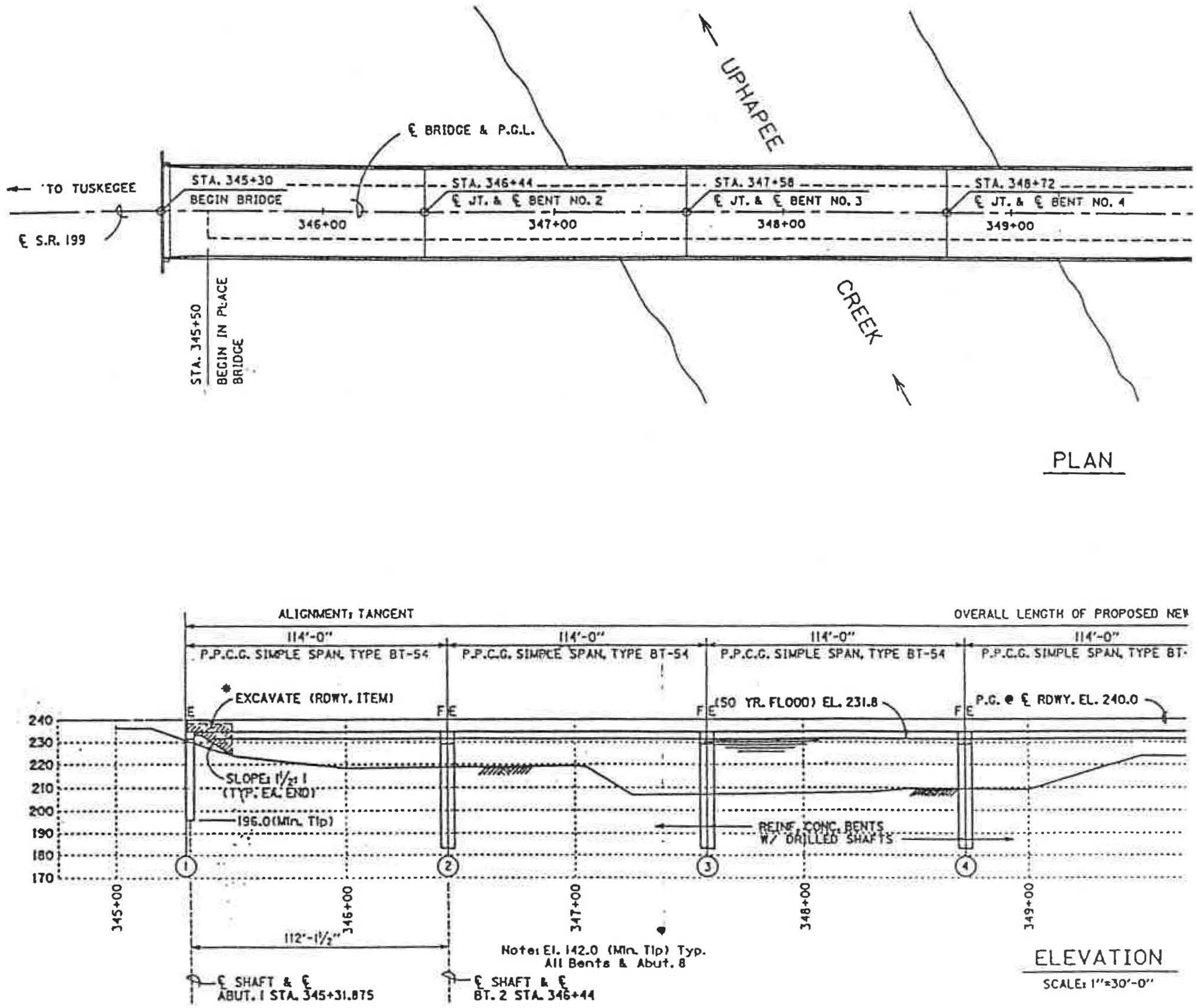


Figure 2.2. Plan and Elevation Views of the Uphapee Creek Bridge

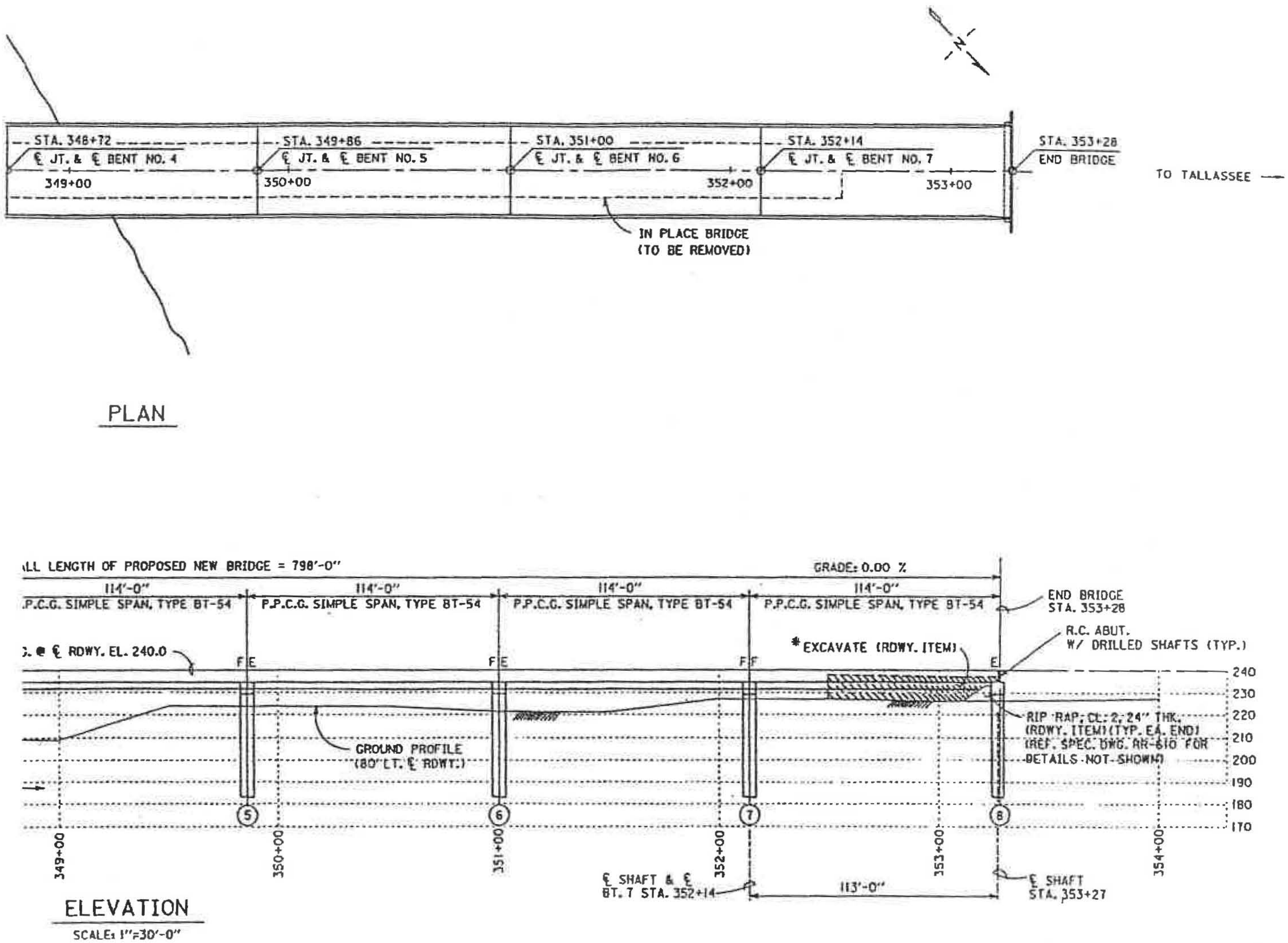


Figure 2.2. (continued) Plan and Elevation Views of the Uphapee Creek Bridge

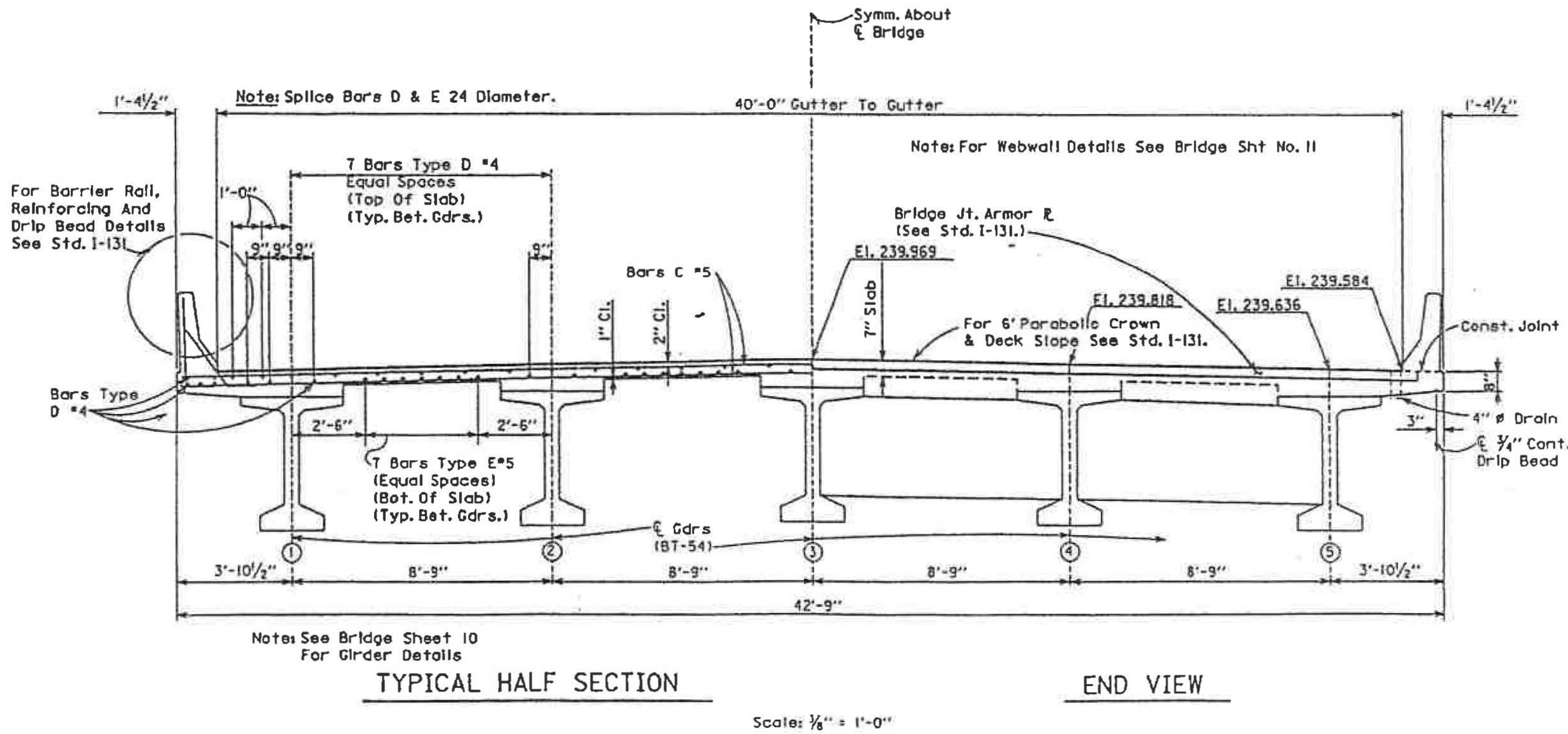


Figure 2.3. Cross Section of the Uphapee Creek Bridge

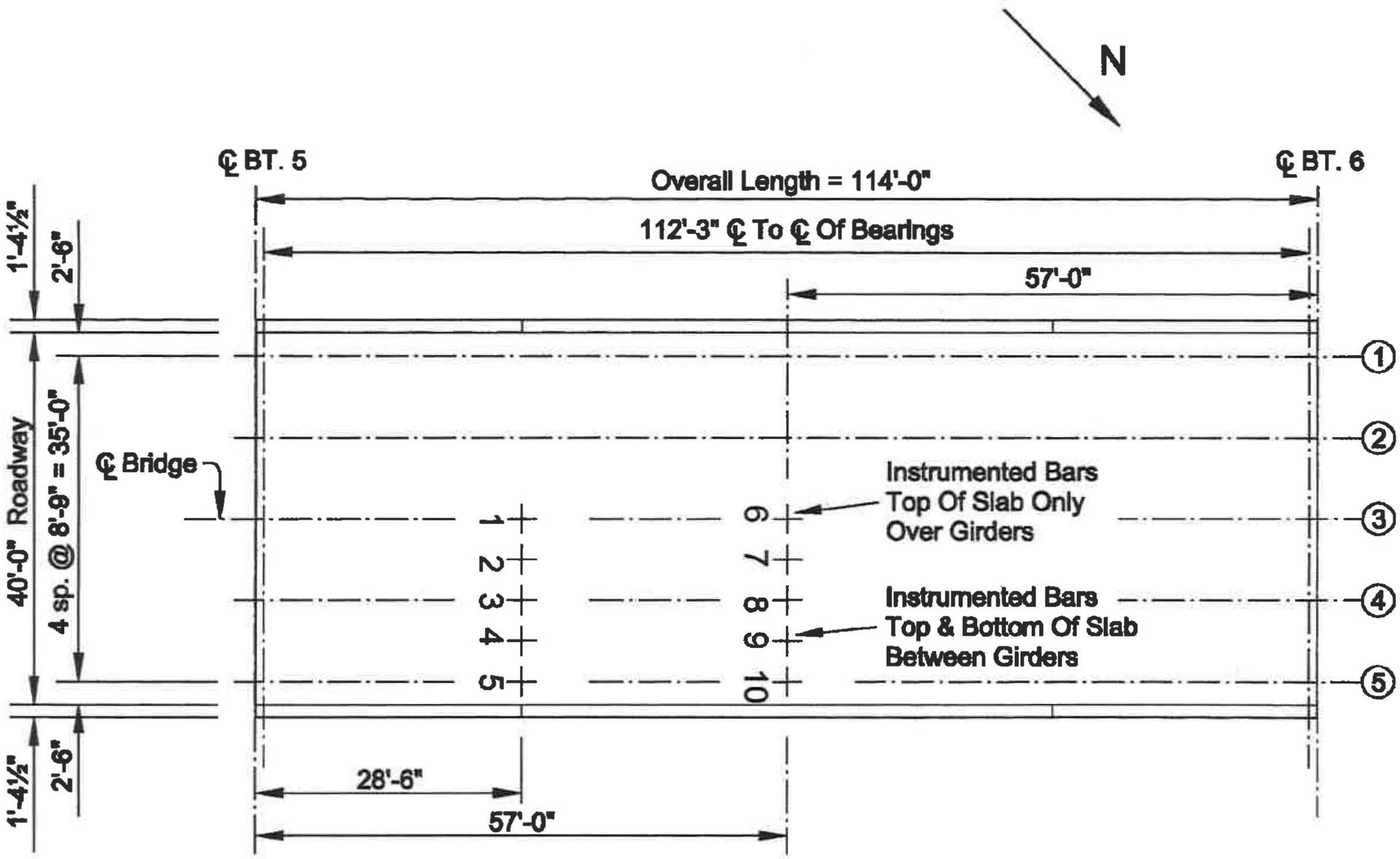


Figure 2.4. Bridge Geometry and Locations of Electrical Resistance Strain Gages in Deck

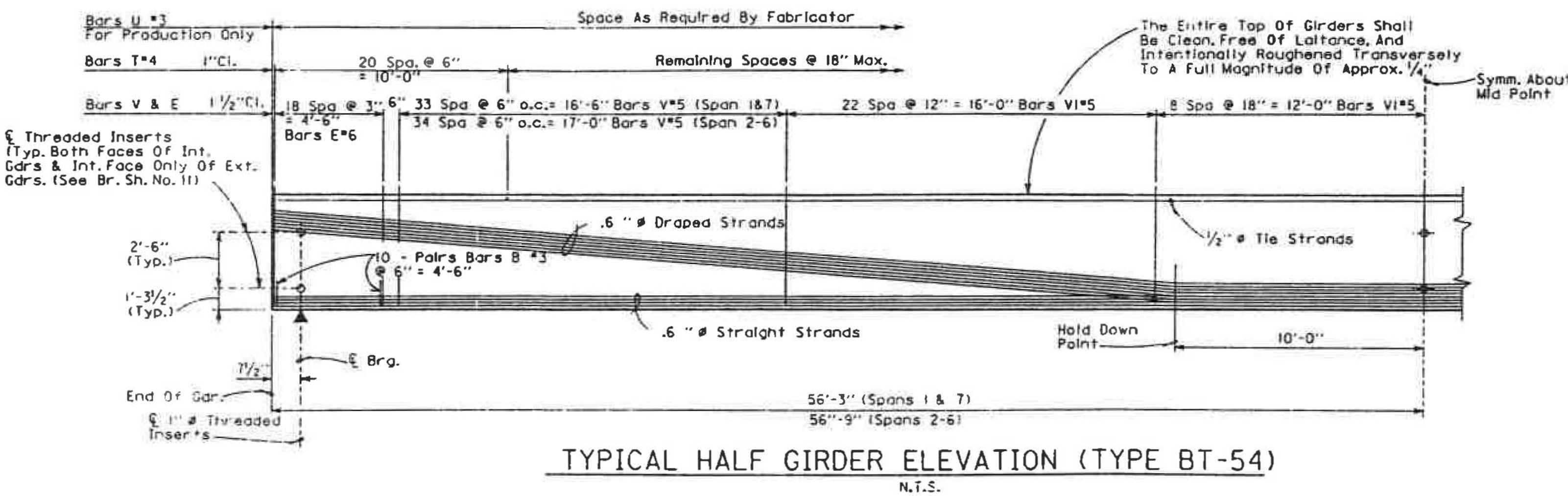
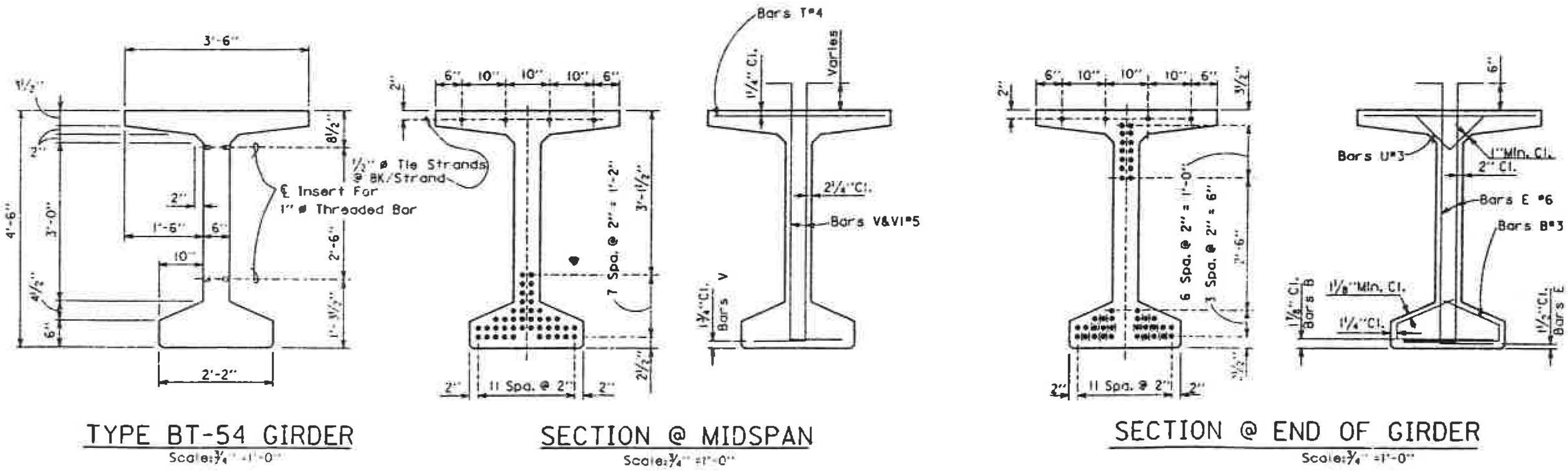


Figure 2.5. Typical Elevation of HPC Girder



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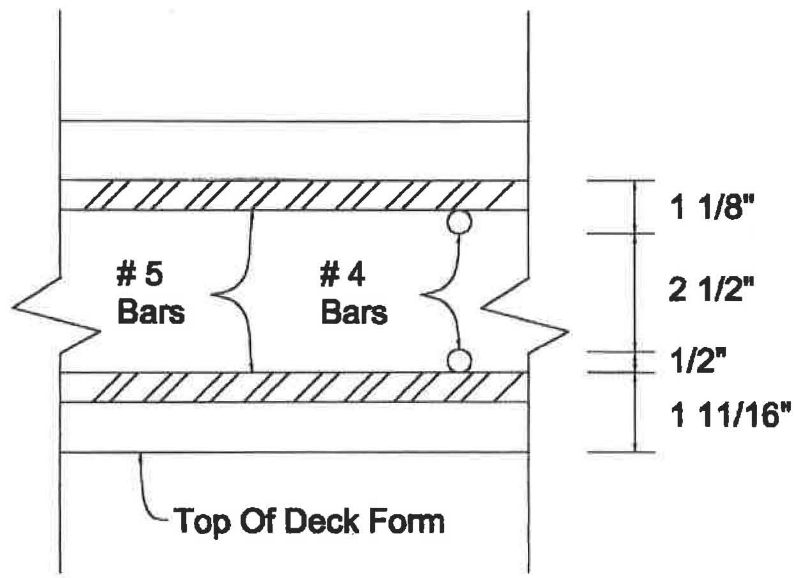

Figure 2.7. Locations of Reinforcement in Deck

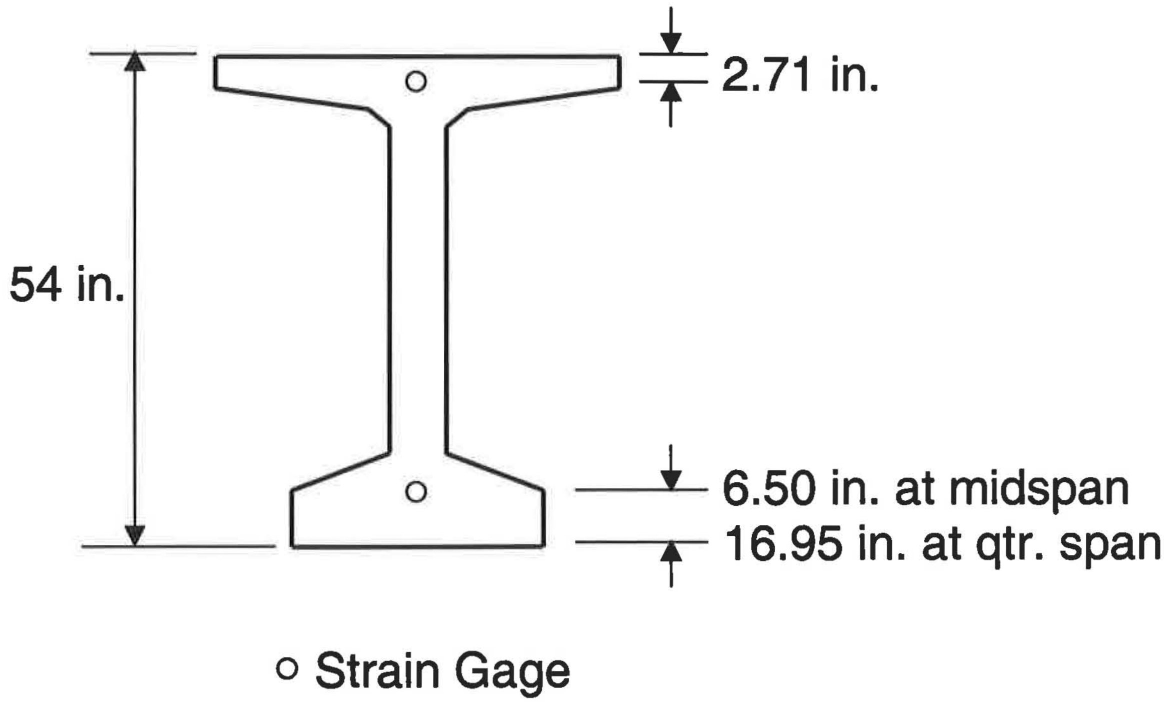

Figure 2.8. Locations of Strain Gages

CHAPTER THREE

TESTS AND TEST RESULTS

TESTS

Live load tests were performed using two identical load test trucks owned and operated by ALDOT. These trucks had a three axle configuration and a gross weight of 77,780 lbs. The axle spacing and weights are illustrated in Figure 3.1.

Static and dynamic tests were performed. Static tests included loading by a single truck and by two trucks simultaneously. Transverse positions of the trucks during the static tests are illustrated in Figure 3.2 and 3.3. For each transverse position, the truck(s) were positioned longitudinally with the interior axle at the quarter span and then at midspan. The quarter span and midspan cross sections are where strain gages and deflectometers were located. The trucks were oriented as if traveling toward the North during all static tests. Each truck was stopped at the positions indicated in Figure 3.2 within plus or minus three inches in the longitudinal and transverse directions. The series of static loadings consisting of all transverse and longitudinal load positions was repeated three times.

Dynamic tests consisted of passing both trucks over the bridge side-by-side with each truck centered in a traffic lane. This was repeated for a total of two passes toward the North and two passes toward the South.

Strain and deflection data from each repetition of each static test along with average values are tabulated in Appendix A. Also given in Appendix A are peak strains and deflections measured in the dynamic tests. In this chapter summaries of the average values from the tests are presented. Comparisons of the test results with calculated values are made in the next chapter.

STATIC TEST RESULTS

Plots of the deflections and strains measured in the prestressed girders due to load positions with two test trucks, and the combinations of load positions defined in Figure 3.4, are shown in Figure 3.5 through 3.18.

Figures 3.5 to 3.18 allow a comparison of strains measurements from the electrical resistance gages and the vibrating wire gages at midspan. Inspection of the figures shows that the electrical resistance gages and vibrating wire gages agreed within 10 microstrain for most cases. Vibrating wire strain gage measurements for the bottom flange of Girders 1 and 2 are unavailable due to a data acquisition problem.

Figures 3.11 and 3.12 Illustrate the girder responses to Load Position 4 which was symmetric about the centerline of the roadway. Both these figures illustrate that the deflection and strains at the exterior girder, Girder 5, were lower than the values at the other exterior girder, Girder 1. Since this trend is present in the deflection and strain data, this trend is concluded to be a real characteristic of the bridge response instead of a measurement error. The reason for the lack of symmetry in the exterior girder responses in unknown. Except for the exterior girders, the bridge response is reasonably symmetric for the symmetric loading of Position 4. This observation provides confidence in the measurements.

A plan view of the deck illustrating locations of instrumented rebar is shown in Figure 2.4. At each location there were bars to measure longitudinal and transverse strains. Longitudinal and transverse strains were measured at he bottom and top of the slab at the locations midway between girders. Shorthand descriptions of the gage locations are used in this report and are illustrated by the following examples. Gage 6-BT is at location 6 defined in Figure 2.4, at the bottom of the slab and measures strain transverse to the roadway. Gage 10-TL is at location 10, at the top of the slab measures strain in the longitudinal direction (parallel to the roadway).

A summary of maximum strains measured at each gage location for load positions with two trucks is provided in Table 3.1. Gages failed at some locations, so measurements for those locations are not available. Results in Table 3.1 indicate that strains measured in the deck were generally small.

Plots of deck strains similar to an influence line were made for the single truck load positions. These were facilitated by plotting the strain measurements as a function of the distance of the test truck from the curb as defined in Figure 3.19. Deck strains are plotted in Figures 3.20 through 3.27.

DYNAMIC TEST RESULTS

Typical plots of strains and deflections measured in a northbound pass of the side-by-side test trucks are provide in Figures 3.29 through 3.34. The plots illustrate that the bridge response is very similar to test results from other simple span bridges. The natural period taken from strain and deflection time histories recorded after the test trucks crossed the bridge, while the bridge was in free vibration, was approximately 0.32 seconds.

Peak strains and deflections measured in the both of the northbound and both of the southbound truck passes are tabulated in Appendix A. The average values for each direction are listed in Table 3.2. Average strains are listed in Table 3.2 only for gages where the static and peak dynamic strains had a magnitude of 10 microstrain or larger.

The average values, in Table 3.2, for the southbound dynamic and northbound dynamic indicate a slightly higher peak response for the southbound trucks. The reason for this is unclear, but this type result is not uncommon. In the static tests the trucks were oriented northbound, so comparisons of the static test results are made only with the dynamic test results for northbound trucks. A comparison of the static test results for load Position 4 and the northbound dynamic tests illustrates the impact effect of the moving trucks. This illustration facilitated by the ratio of the dynamic to static results shown in the last column of Table 3.2. The ratios in Table 3.2 are generally lower than the dynamic load allowance factor (1+IM) of 1.33 defined in AASHTO *LRFD*. The only exceptions are at gages 1-TL and 2-BL which measured longitudinal strains in the deck. Comparisons of the static and peak dynamic strains for northbound trucks are also made in Figure 3.34 and 3.35. The girder strains and midspan deflections in those figures follow similar trends in both the static and dynamic tests.

Table 3.1. Maximum Deck Strains from Static Loading by Two Trucks

Gage	Maximum Strain	Load Position
1-TL	-38	Q4
2-BL	-34	Q1, Q4
4-TL	-49	Q1
5-TL	-47	Q1
6-TL	-49	M4
7-TL	-42	M1
7-BL	-46	M2, M3**
8-TL	-55	M2
9-TL	-57	M1
9-BL	-36	M3
10-TL	-73	M1
1-TT	14	Q2
2-TT	-10/17*	Q4/M1
2-BT	24	Q2
3-TT	3	M1
4-TT	-6/2	Q2/M1
4-BT	15	Q1
5-TT	23	Q1
6-TT	-13	Q4
7-TT	-4	M2
7-BT	-13	M2
8-TT	-10	Q2,Q3
9-TT	-2/2	M2/M3, M4, Q1
9-BT	5	M1
10-TT	40	M1

* Largest compressive strain/largest tensile strain
** Multiple positions created same maximum strain

Table 3.2. Comparison of Static and Peak Dynamic Response

Sensor	Southbound Dynamic	Northbound Dynamic	Northbound Static	Northbound Dyn./Static
(a) Strains from Electrical Resistance Gages (microstrain)				
Girder 1	87	91	74	1.23
Girder 2	91	93	82	1.13
Girder 3	90	84	80	1.05
Girder 4	100	98	95	1.03
Girder 5	67	56	48	1.17
1-TL	-38	-37	-21	1.76
2-BL	-36	-34	-24	1.42
4-TL	-29	-26	-24	1.08
5-TL	-25	-22	-20	1.10
6-TL	-47	-48	-49	0.98
7-TL	-35	-32	-34	0.94
7-BL	-45	-44	-45	0.98
8-TL	-41	-39	-30	1.30
9-TL	-42	-38	-38	1.00
9-BL	-28	-25	-25	1.00
10-TL	-45	-39	-38	1.03
10-TT	28	24	20	1.20
(b) Deflections at Midspan (in.)				
Girder 1	0.323	0.334	0.273	1.22
Girder 2	0.360	0.355	0.313	1.13
Girder 3	0.393	0.369	0.345	1.07
Girder 4	0.371	0.331	0.308	1.07
Girder 5	0.271	0.228	0.202	1.13
(c) Deflections at Quarter Span (in.)				
Girder 3	0.258	0.240	0.202	1.19
Girder 4	0.244	0.215	0.181	1.19
Girder 5	0.226	0.188	0.136	1.38

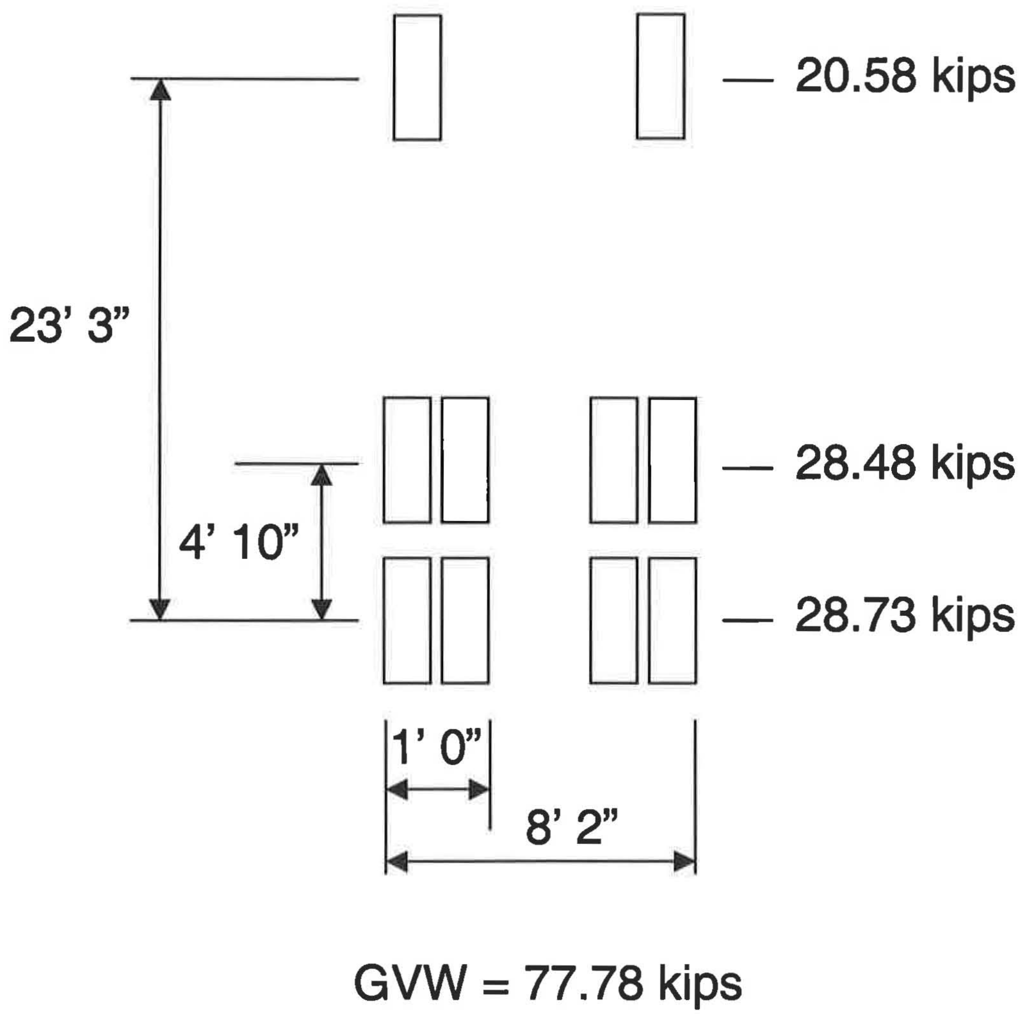

Figure 3.1. Load Truck Configuration

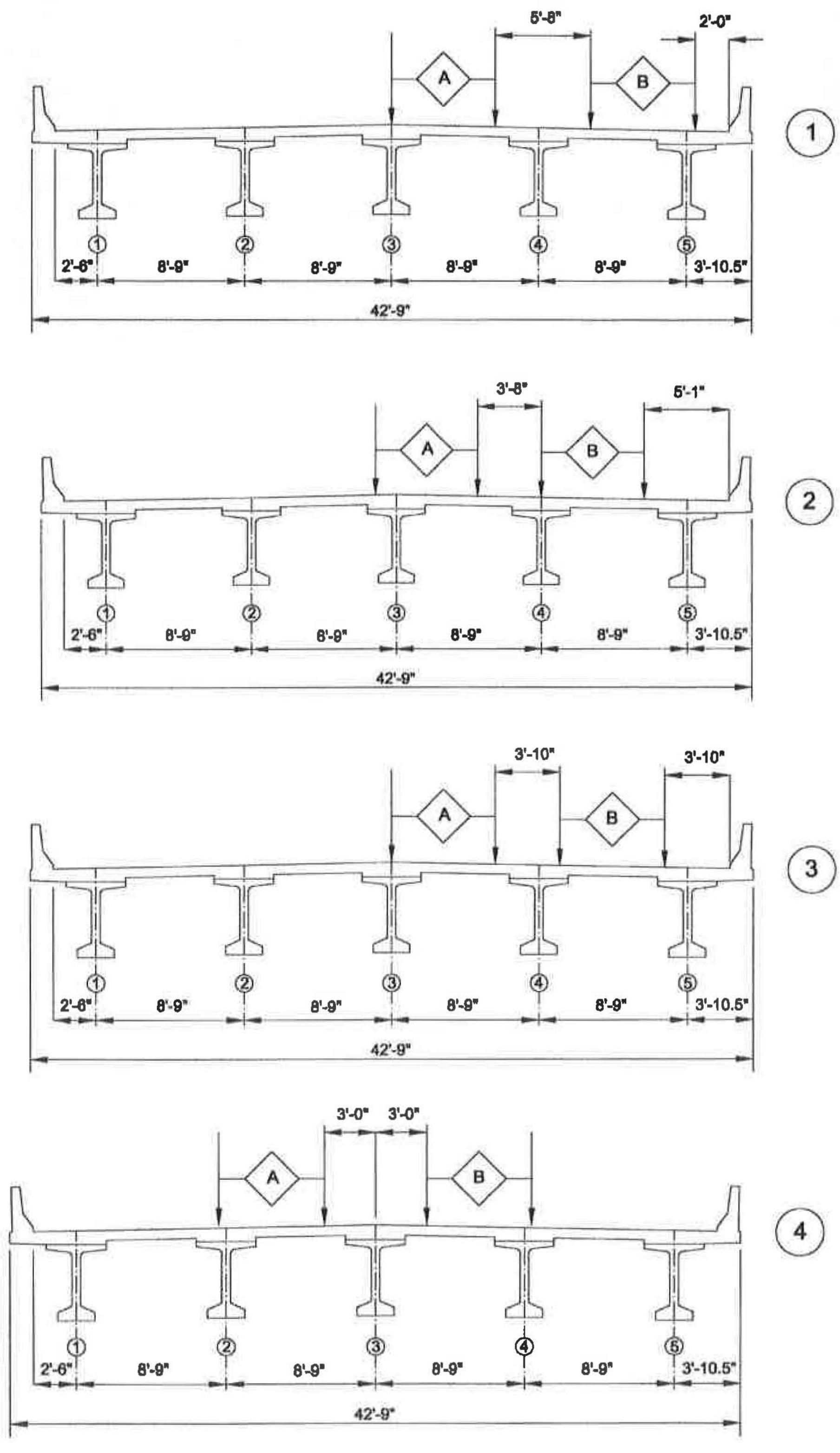

Figure 3.2. Transverse Positions for Two Trucks

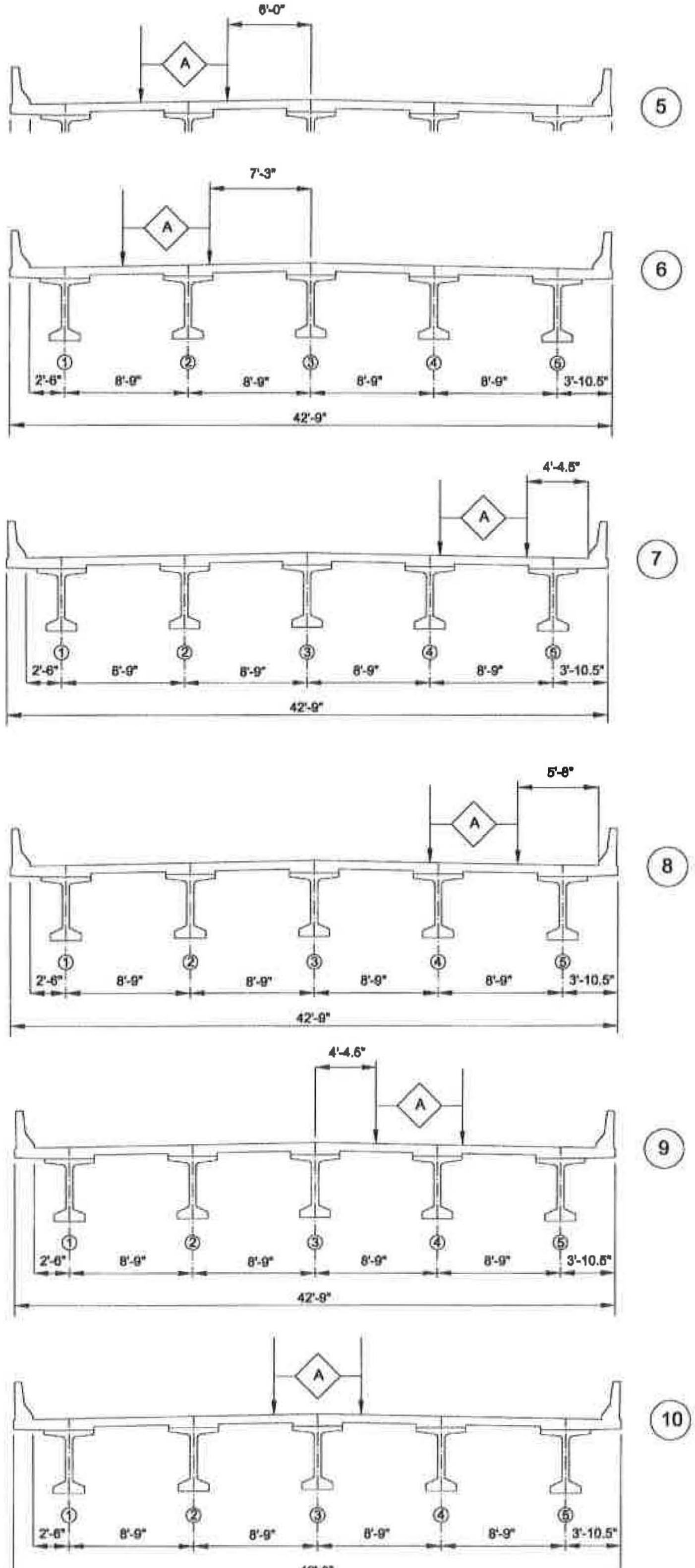

Figure 3.3. Transverse Positions for Single Trucks

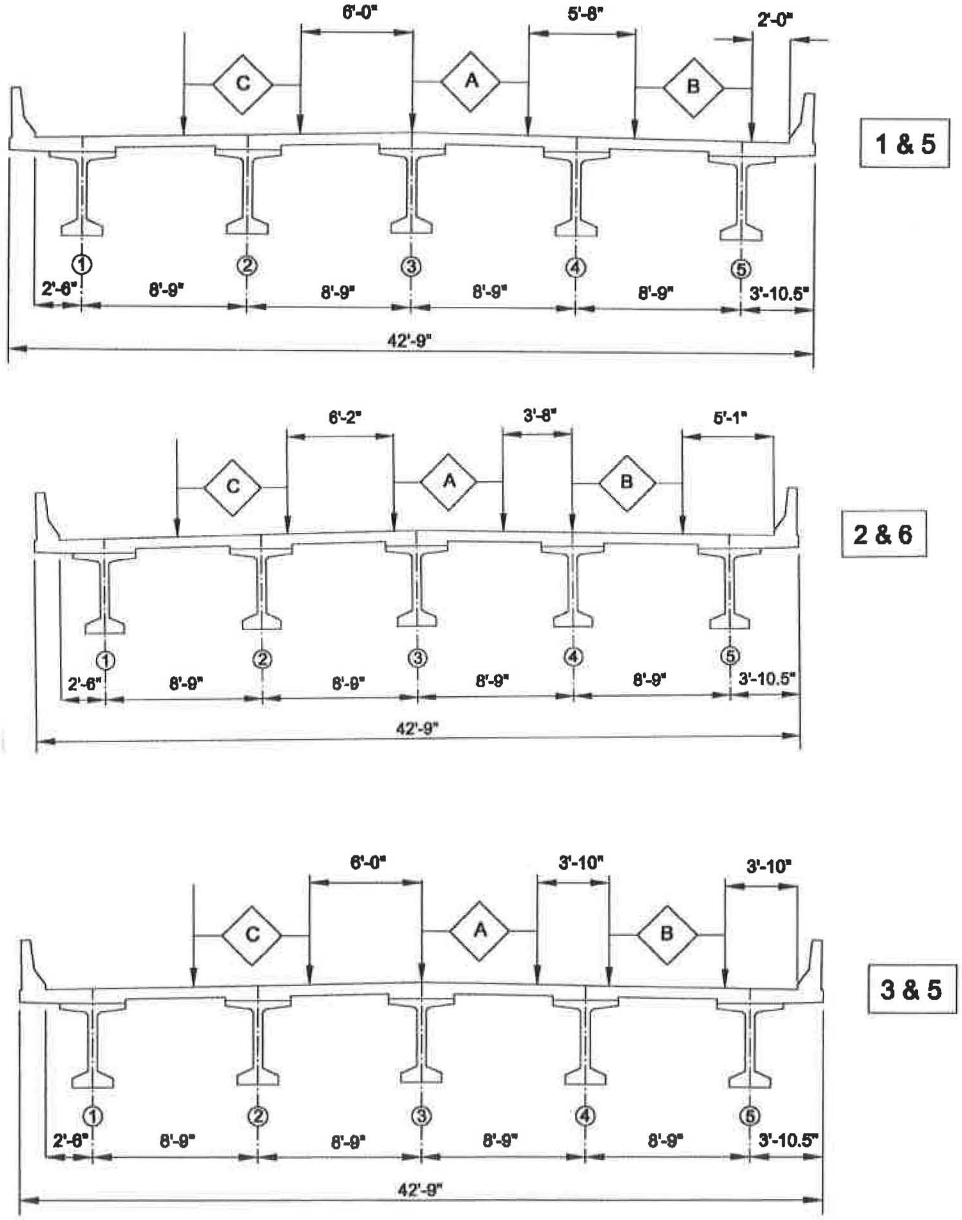

Figure 3.4. Combinations of Transverse Positions

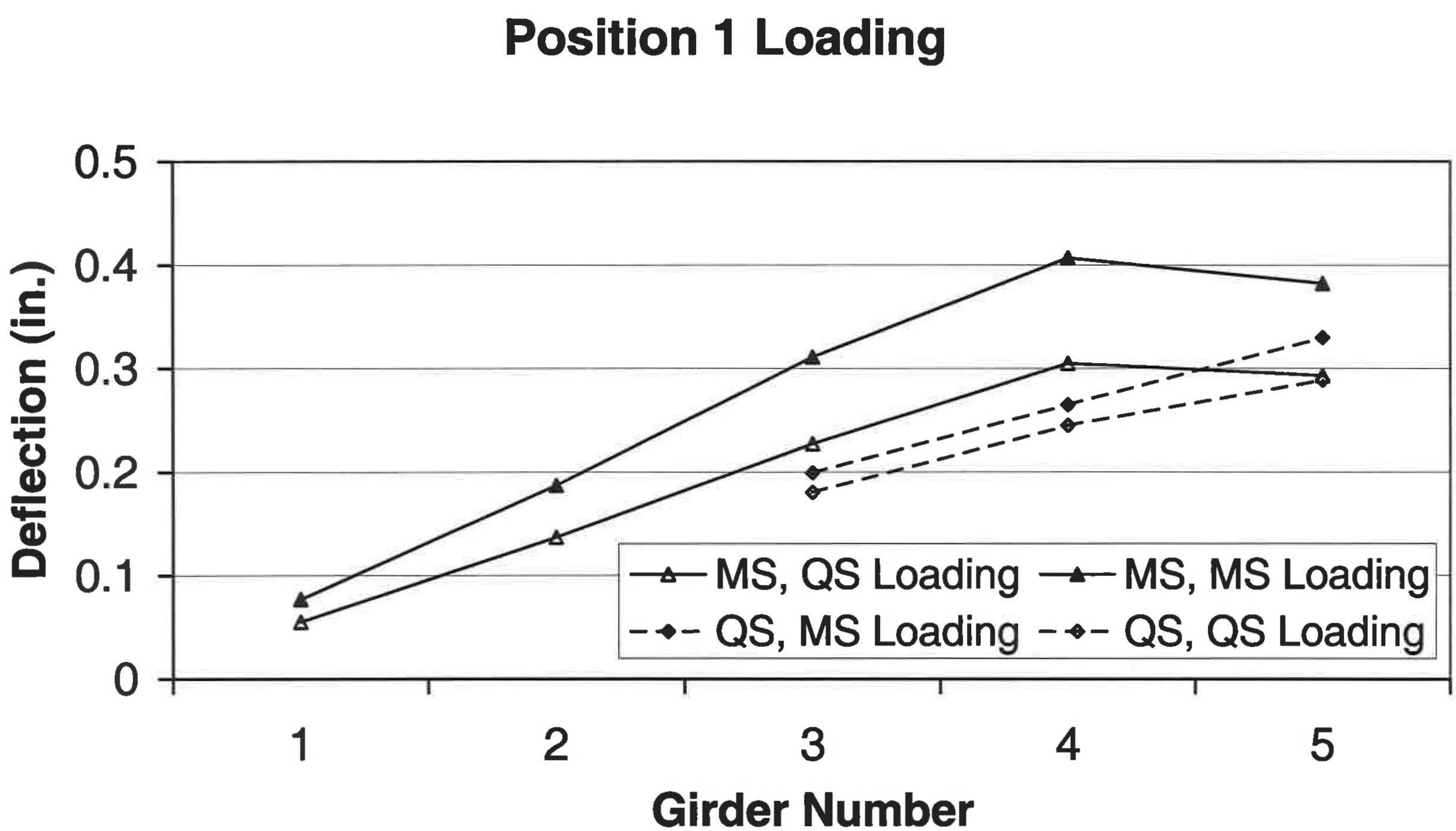

Figure 3.5. Deflections Due to Midspan and Quarter Span Loading in Position 1

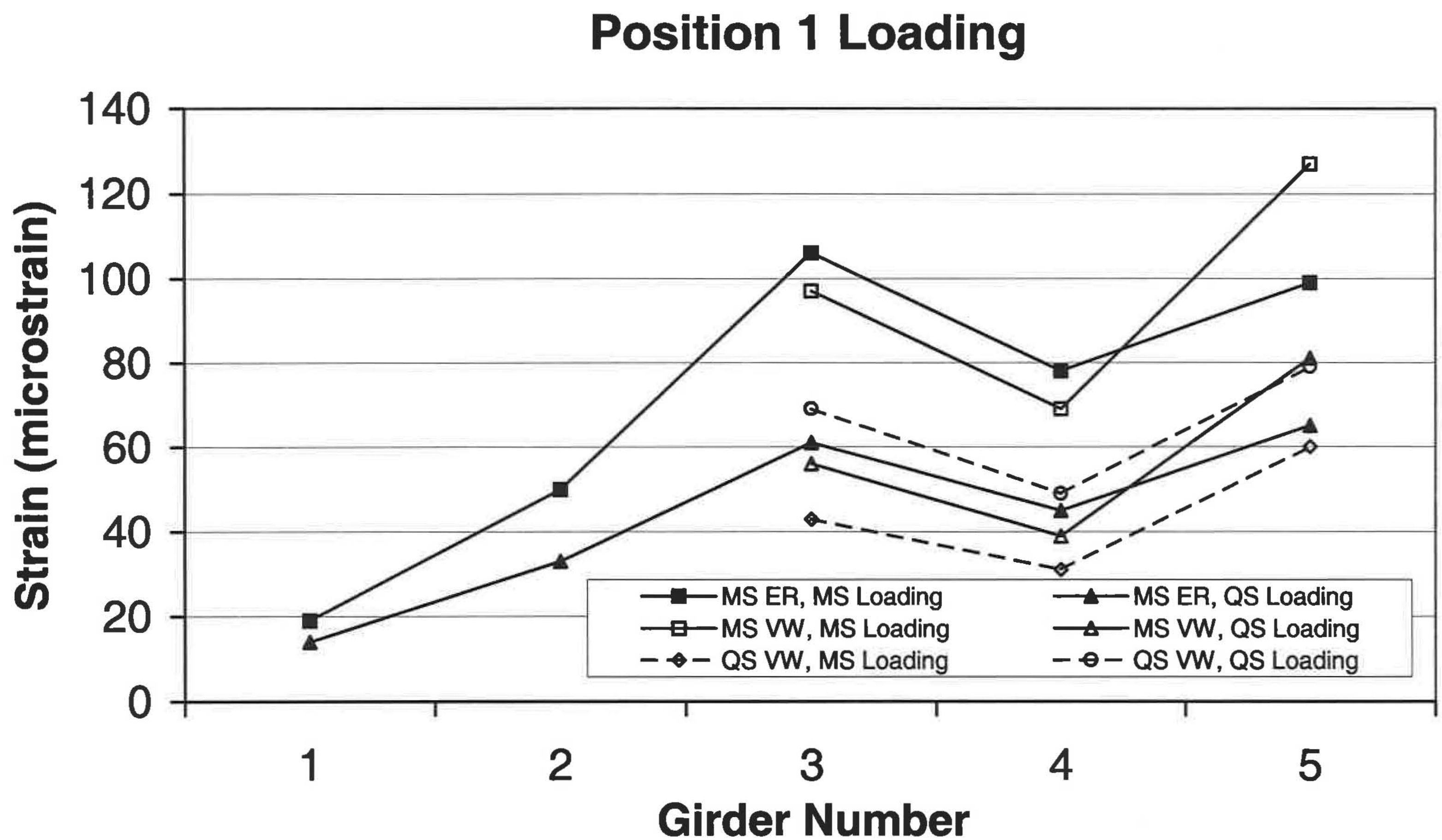

Figure 3.6. Strains Due to Midspan and Quarter Span Loading in Position 1

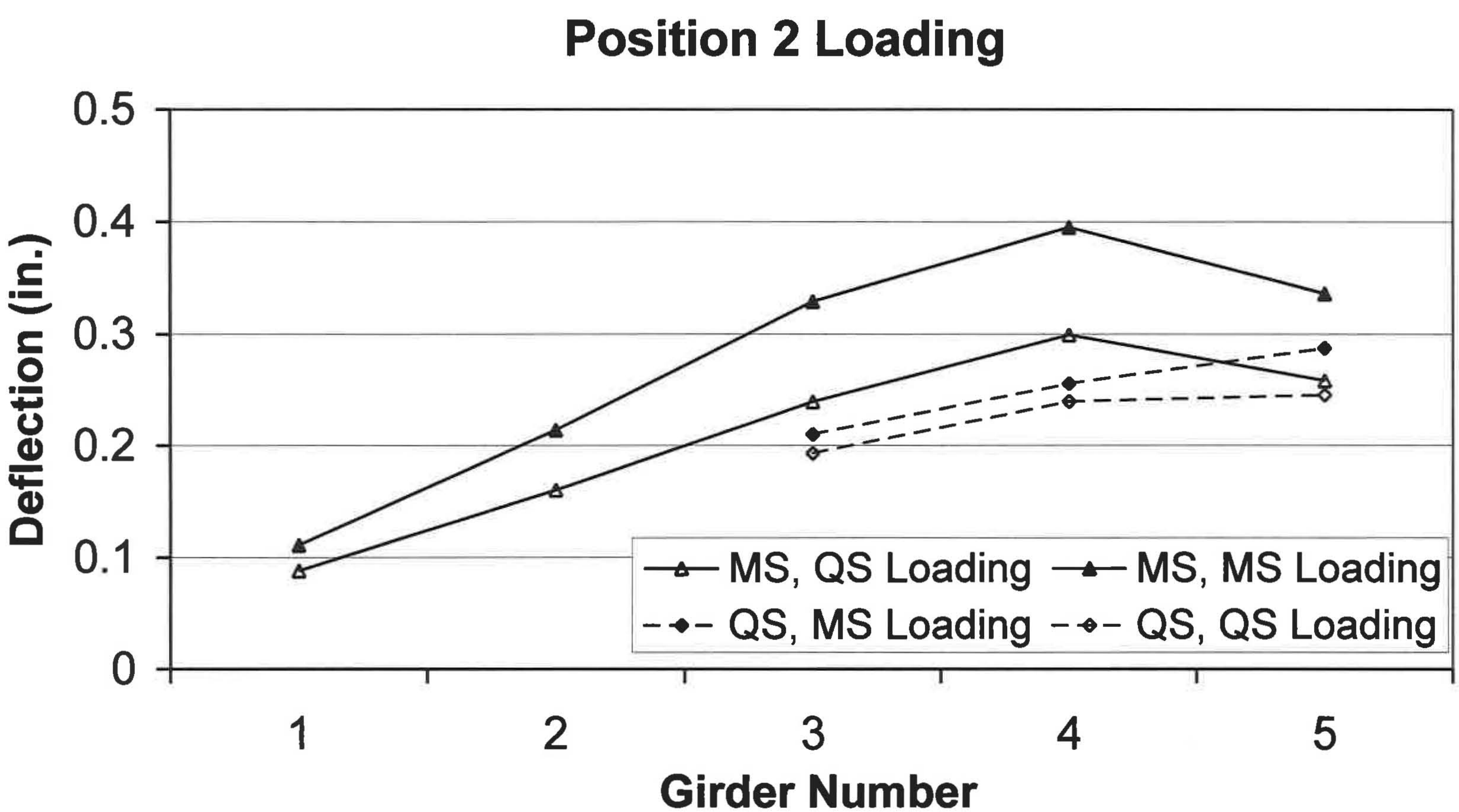

Figure 3.7. Deflections Due to Midspan and Quarter Span Loading in Position 2

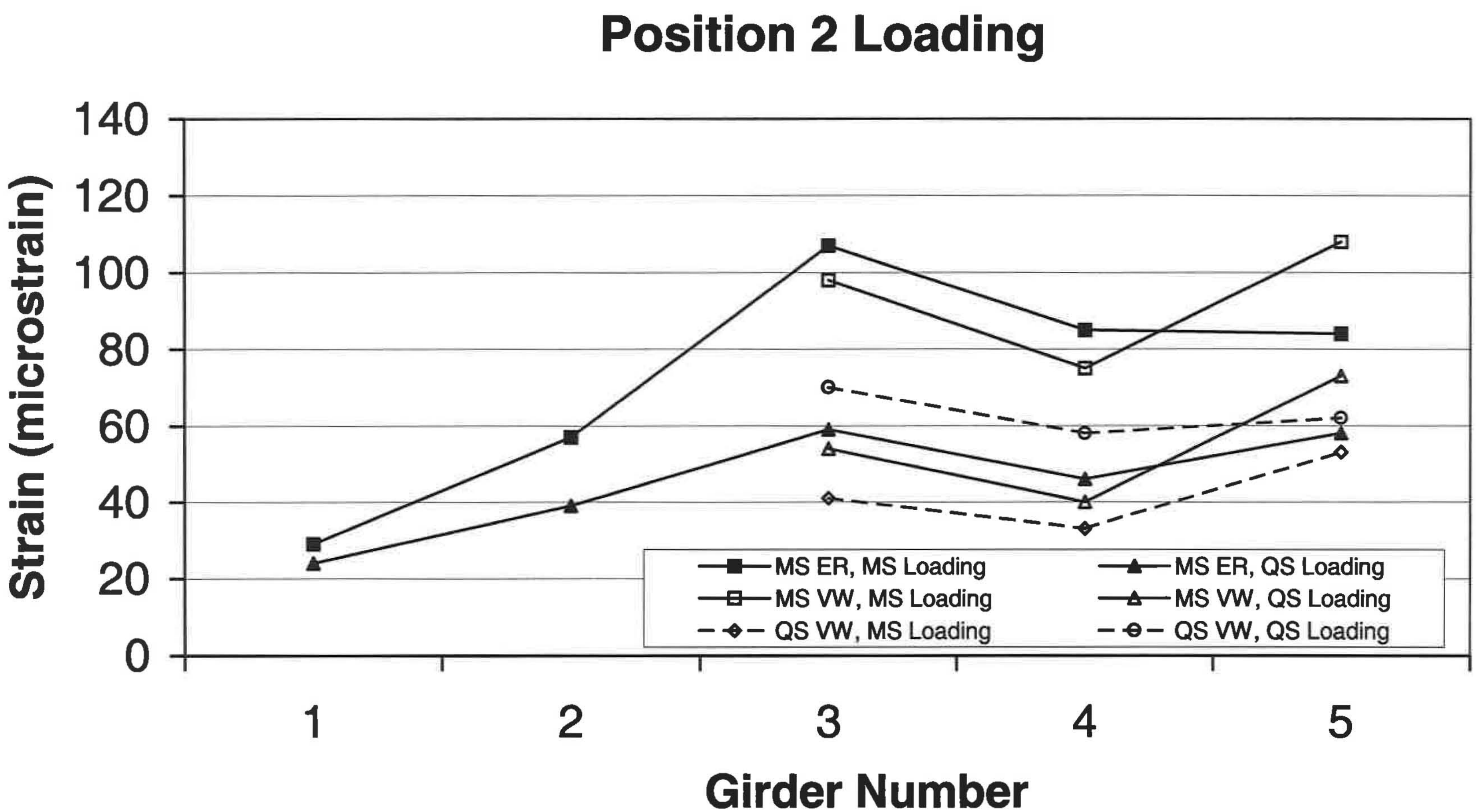

Figure 3.8. Strains Due to Midspan and Quarter Span Loading in Position 2

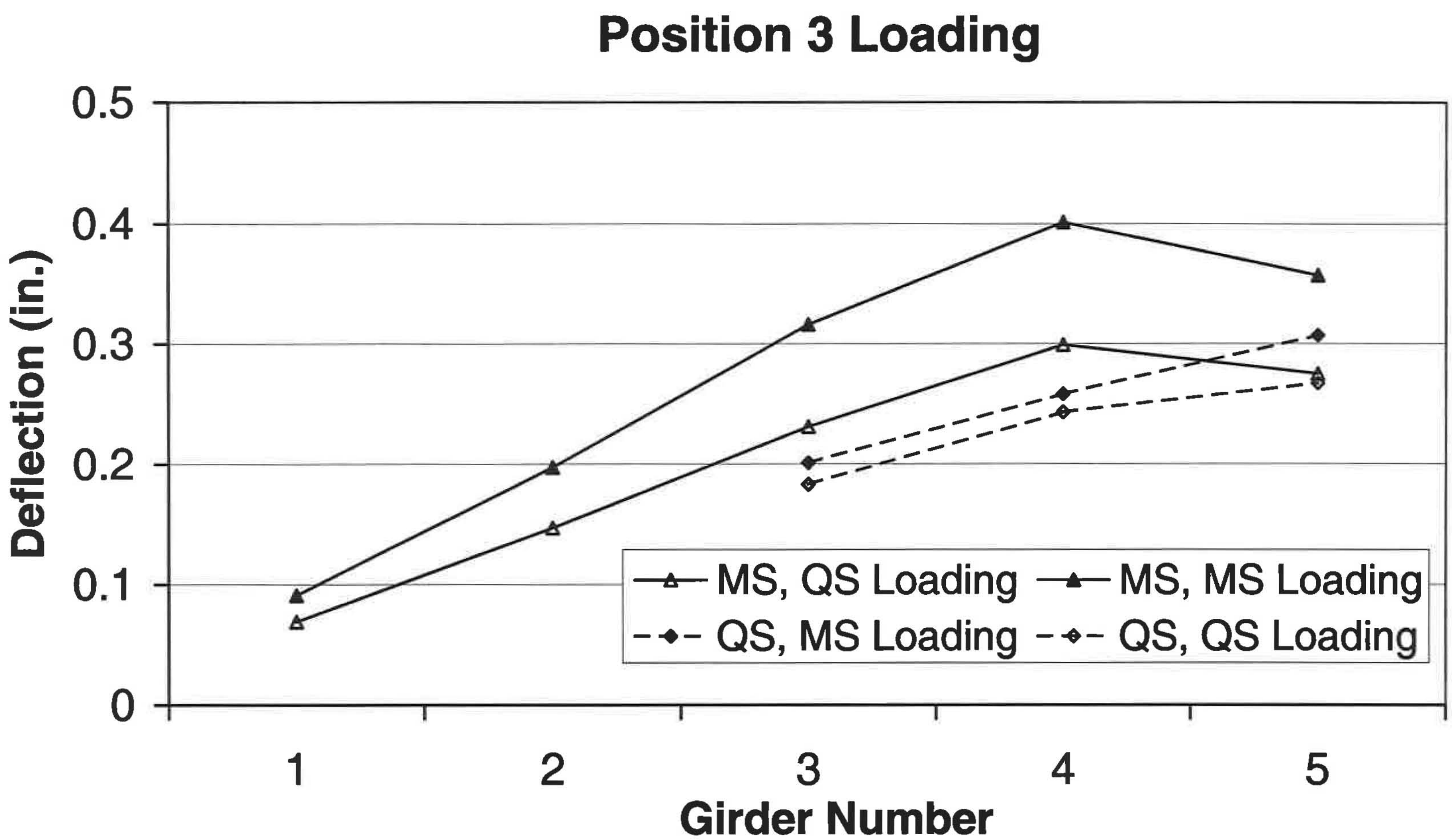

Figure 3.9. Deflections Due to Midspan and Quarter Span Loading in Position 3

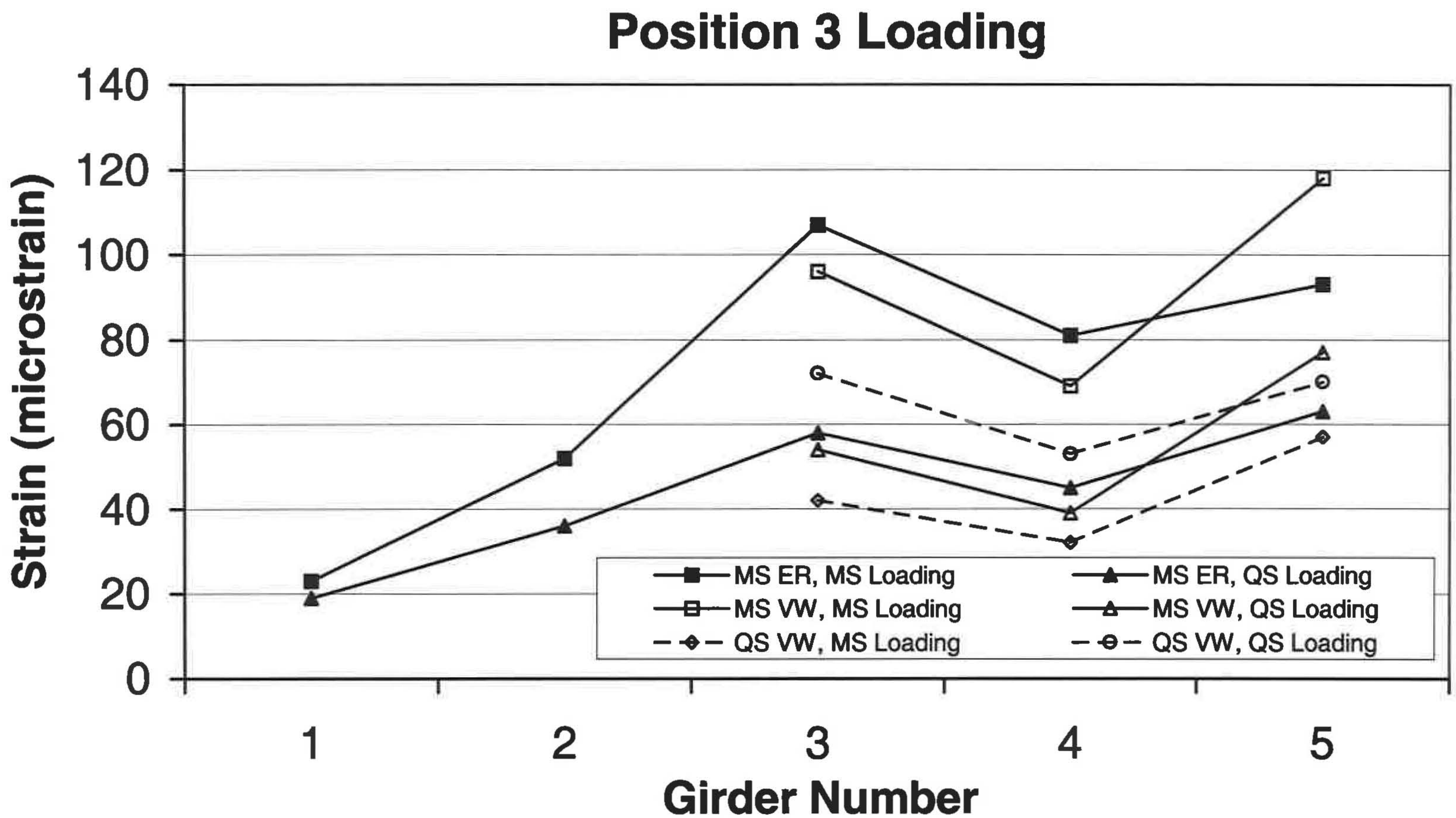

Figure 3.10. Strains Due to Midspan and Quarter Span Loading in Position 3

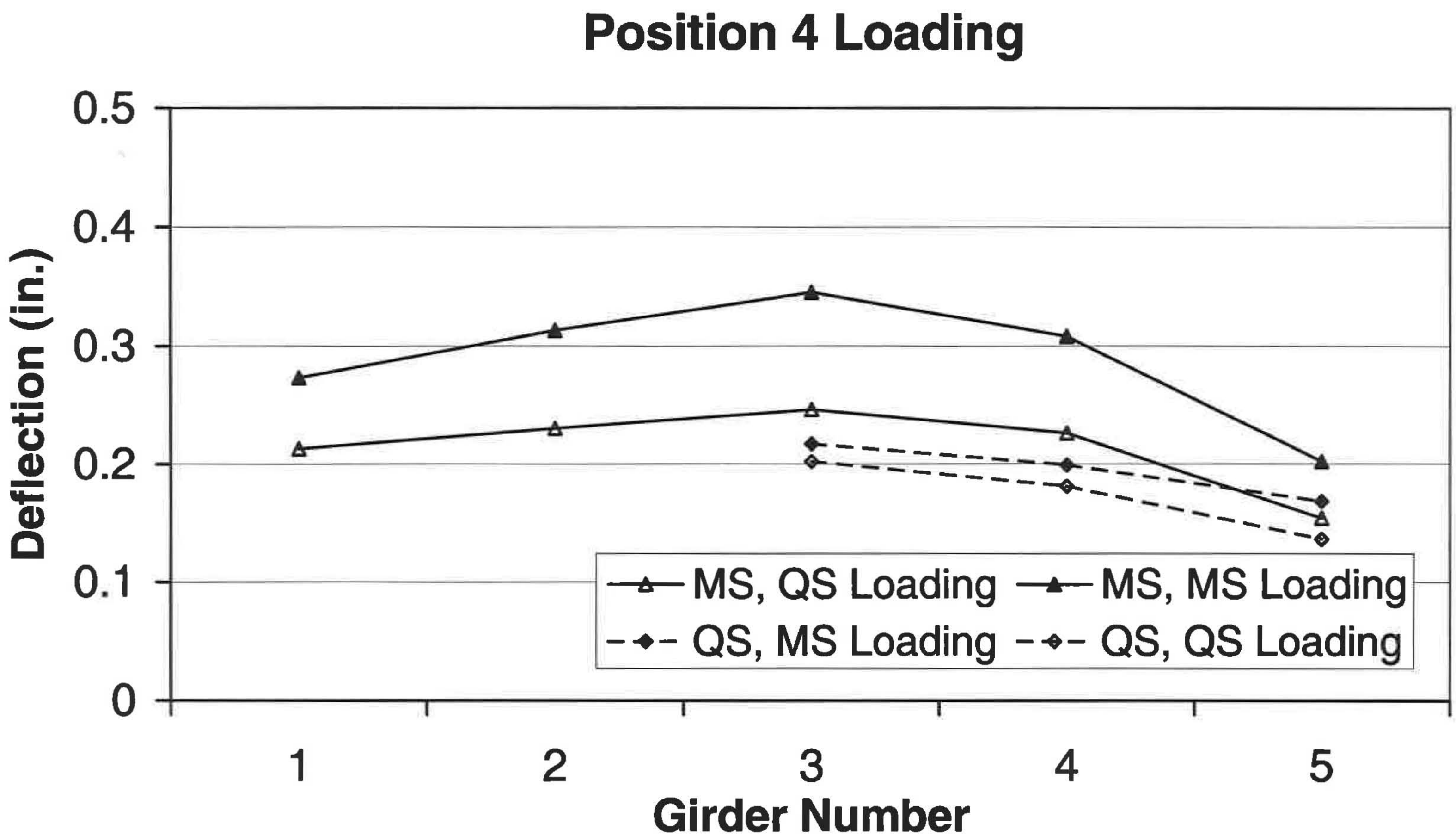

Figure 3.11. Deflections Due to Midspan and Quarter Span Loading in Position 4

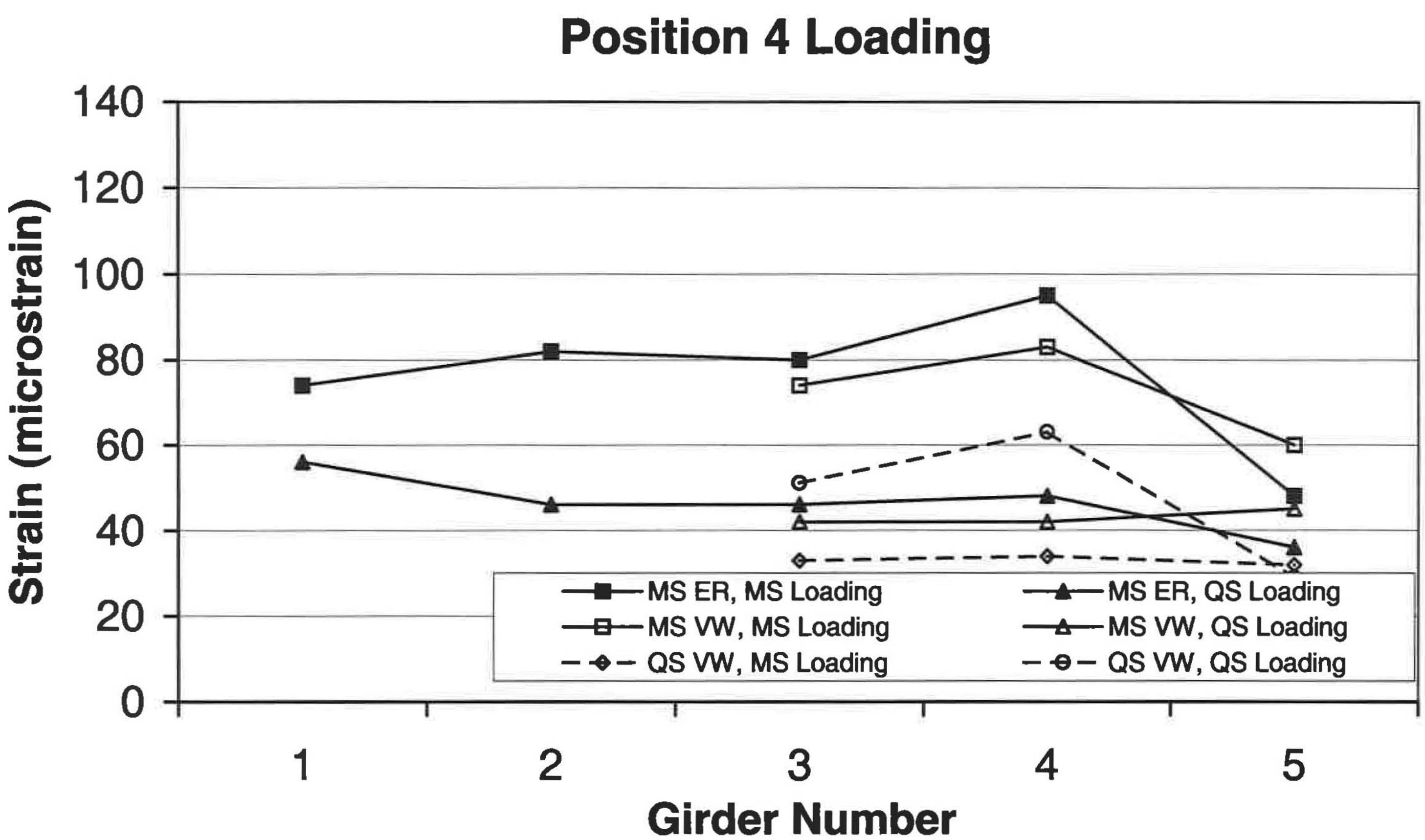

Figure 3.12. Strains Due to Midspan and Quarter Span Loading in Position 4

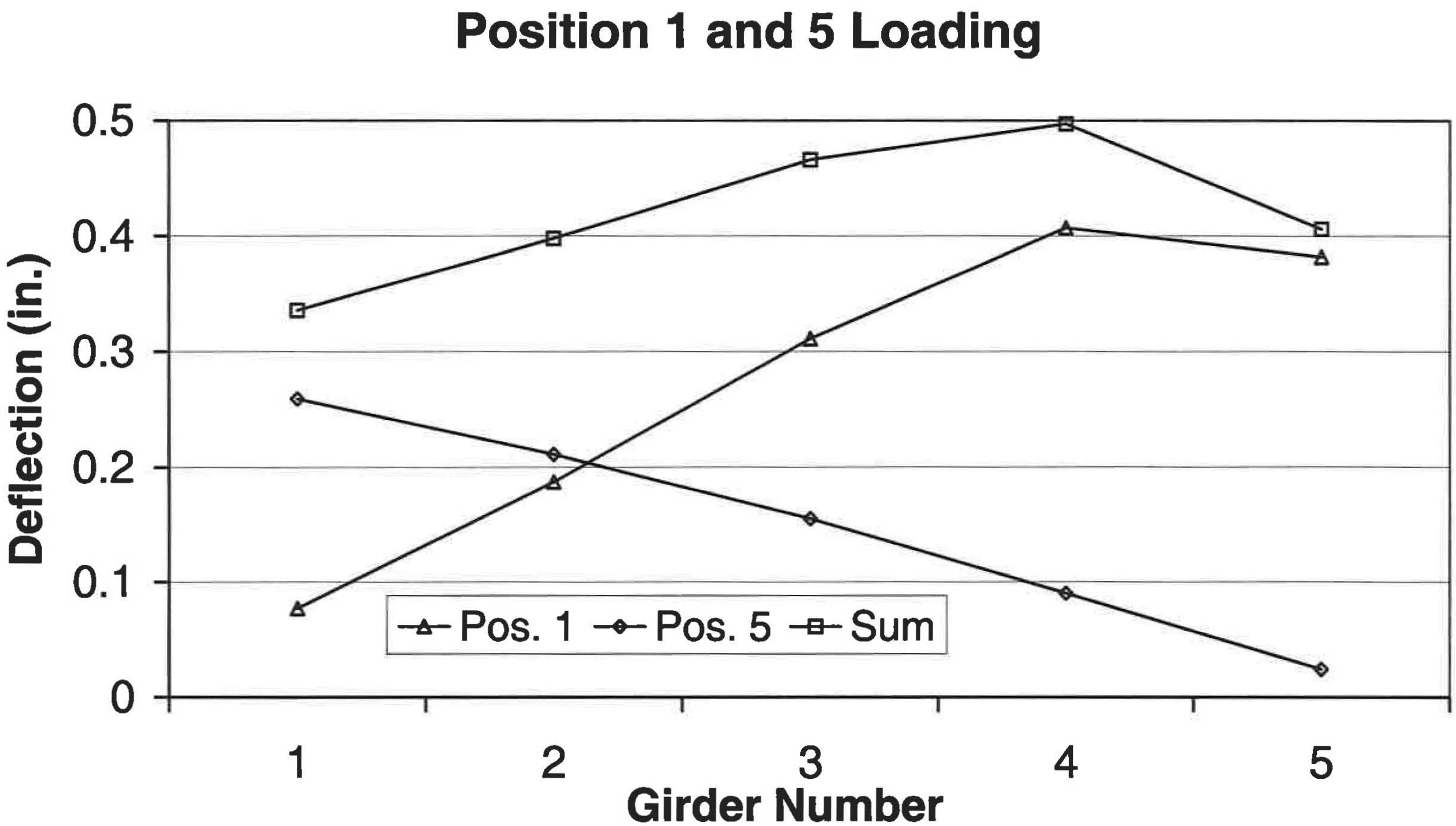

Figure 3.13. Midspan Deflections Due to Midspan Loading in Position 4

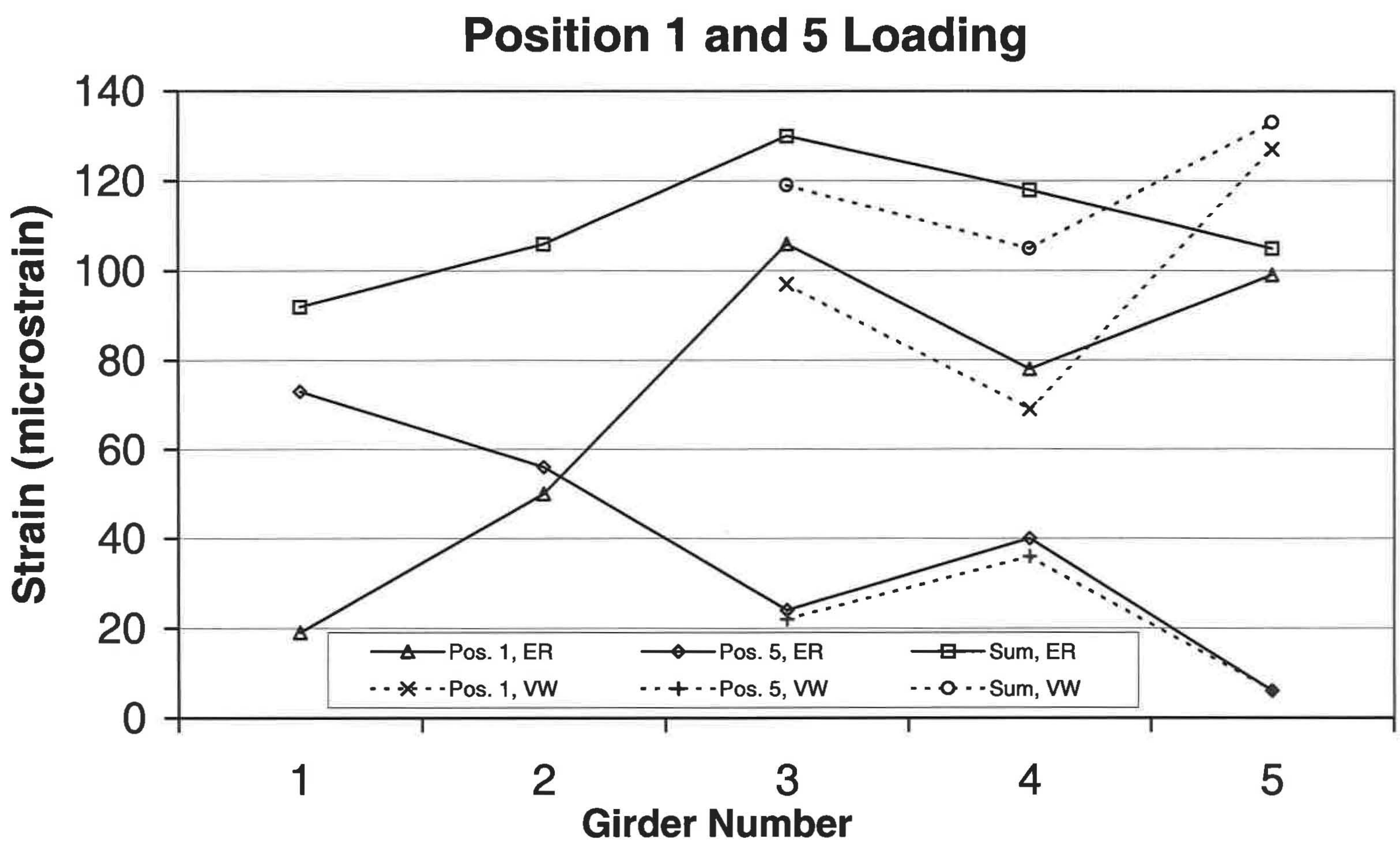

Figure 3.14. Midspan Strains Due to Midspan Loading in Positions 1 and 5

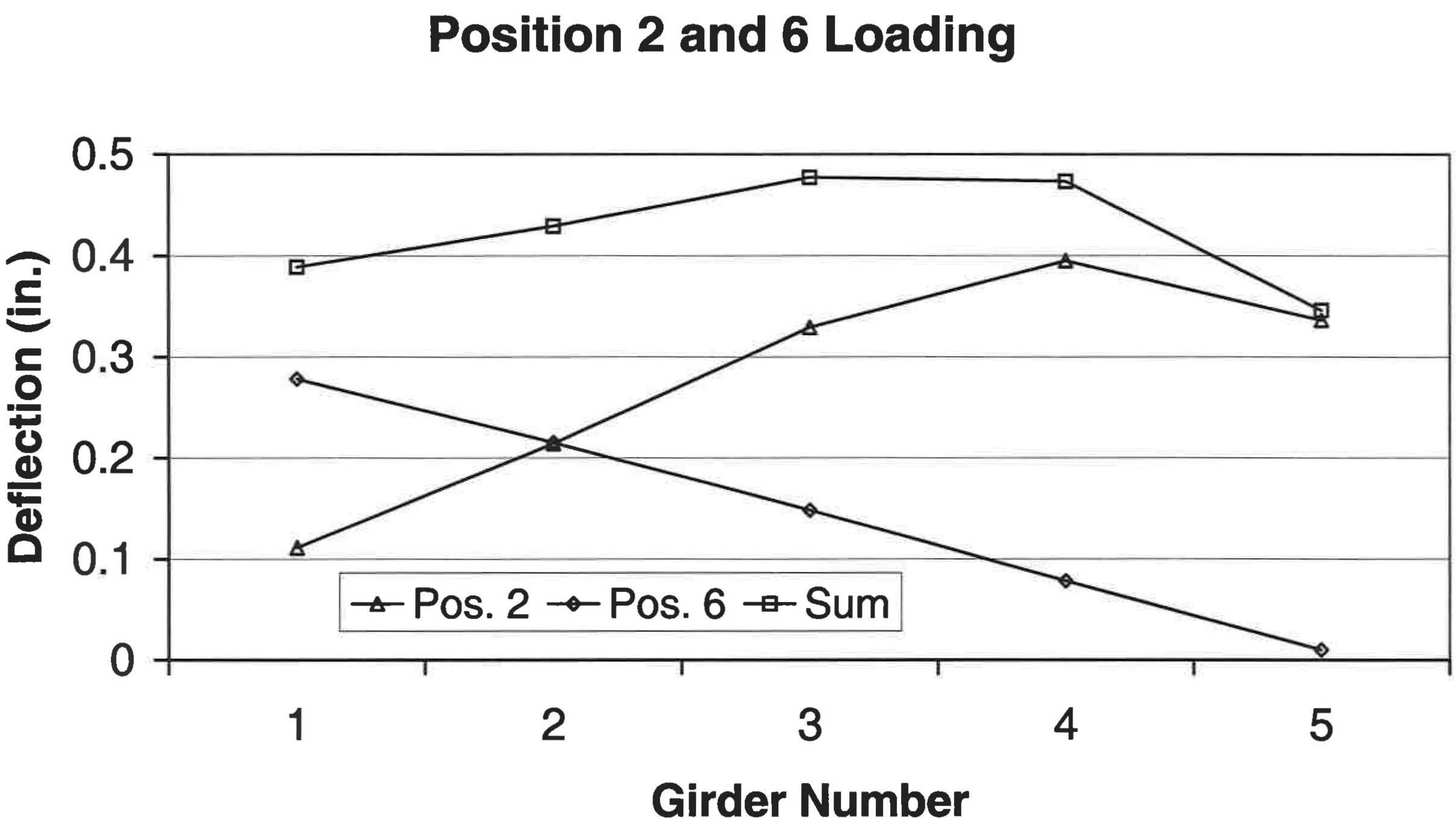

Figure 3.15. Midspan Deflections Due to Midspan Loading in Positions 2 and 6

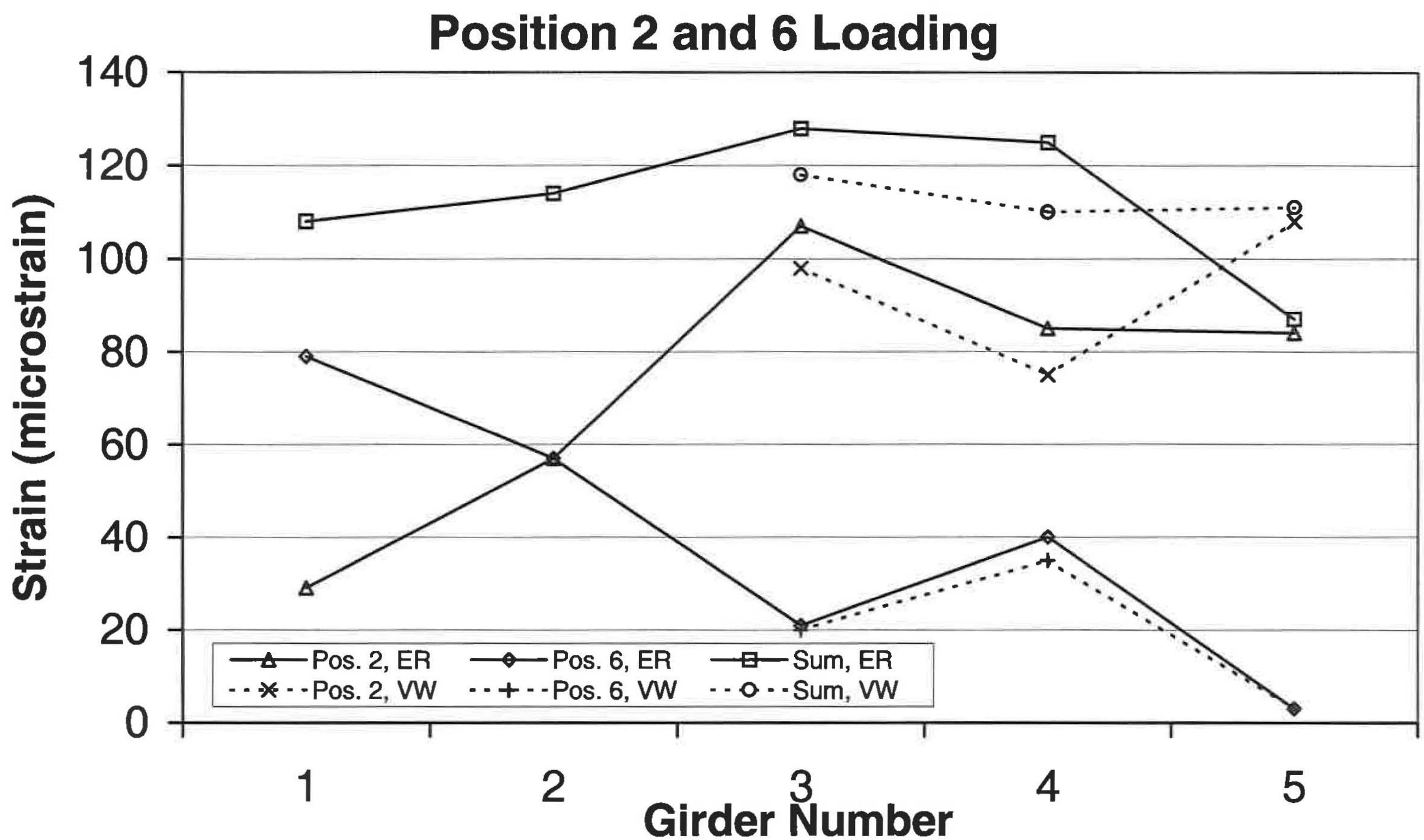

Figure 3.16. Midspan Strains Due to Midspan Loading in Positions 2 and 6

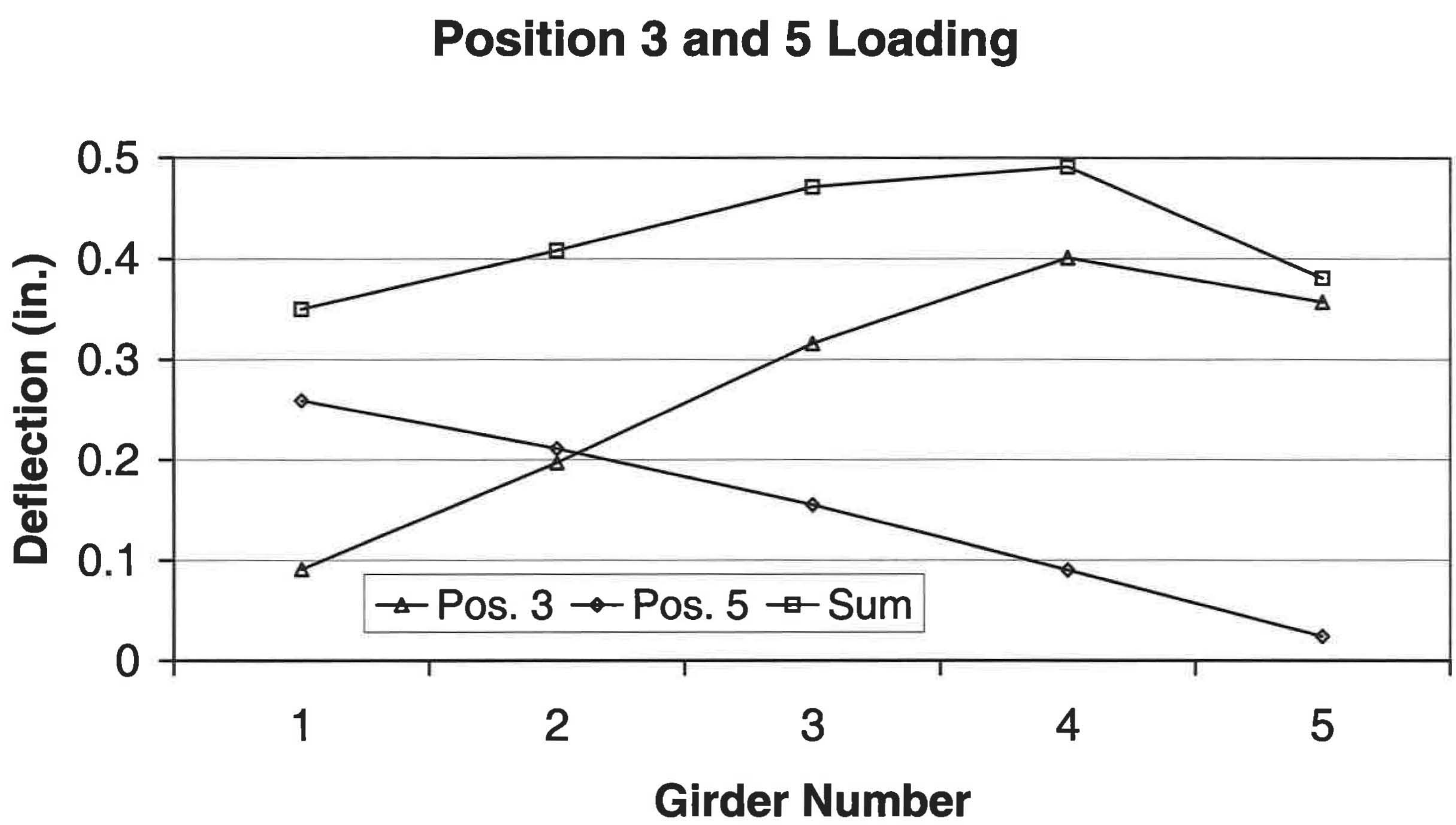

Figure 3.17. Midspan Deflections Due to Midspan Loading in Positions 3 and 5

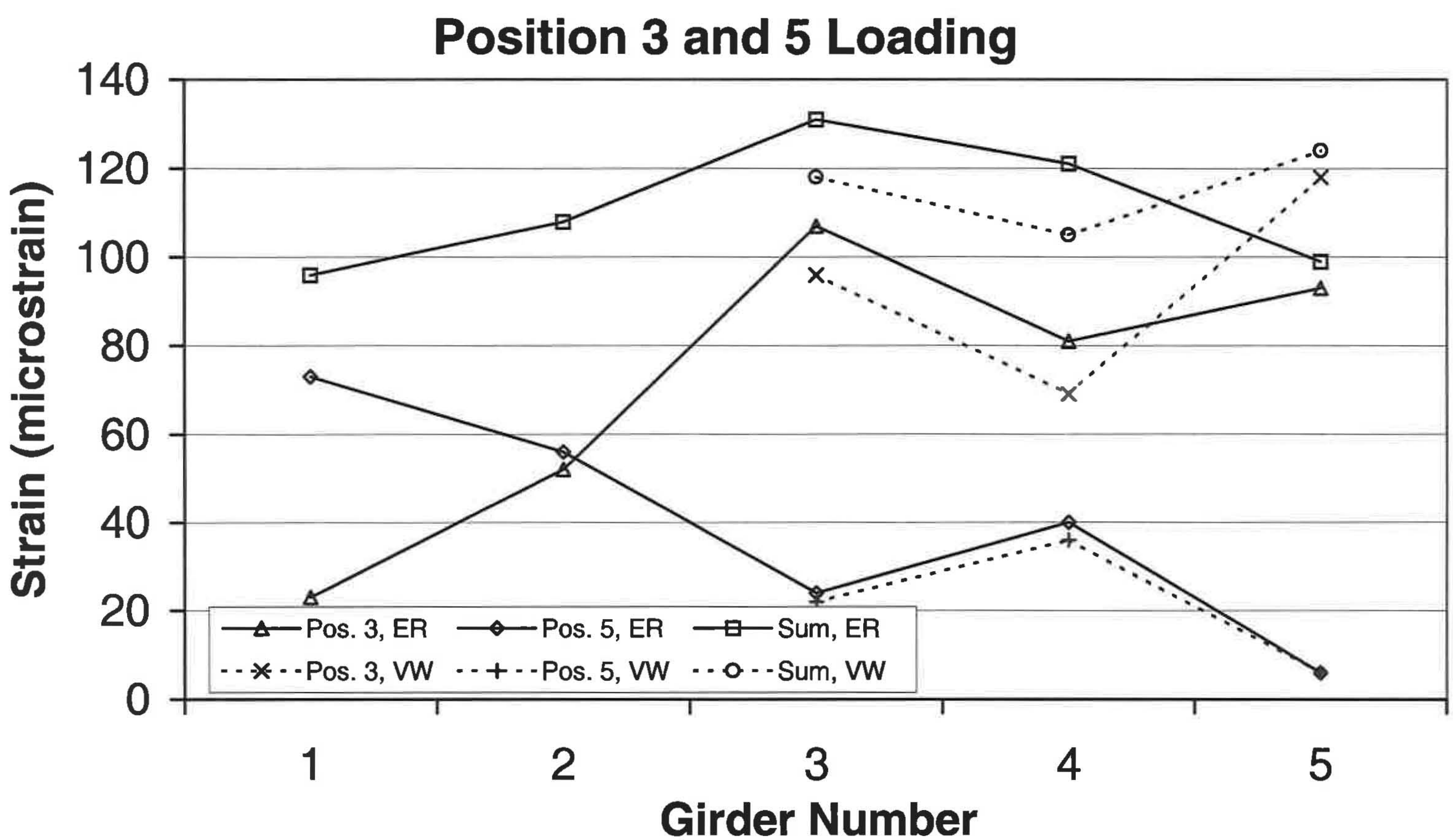

Figure 3.18. Midspan Strains Due to Midspan Loading in Positions 3 and 5

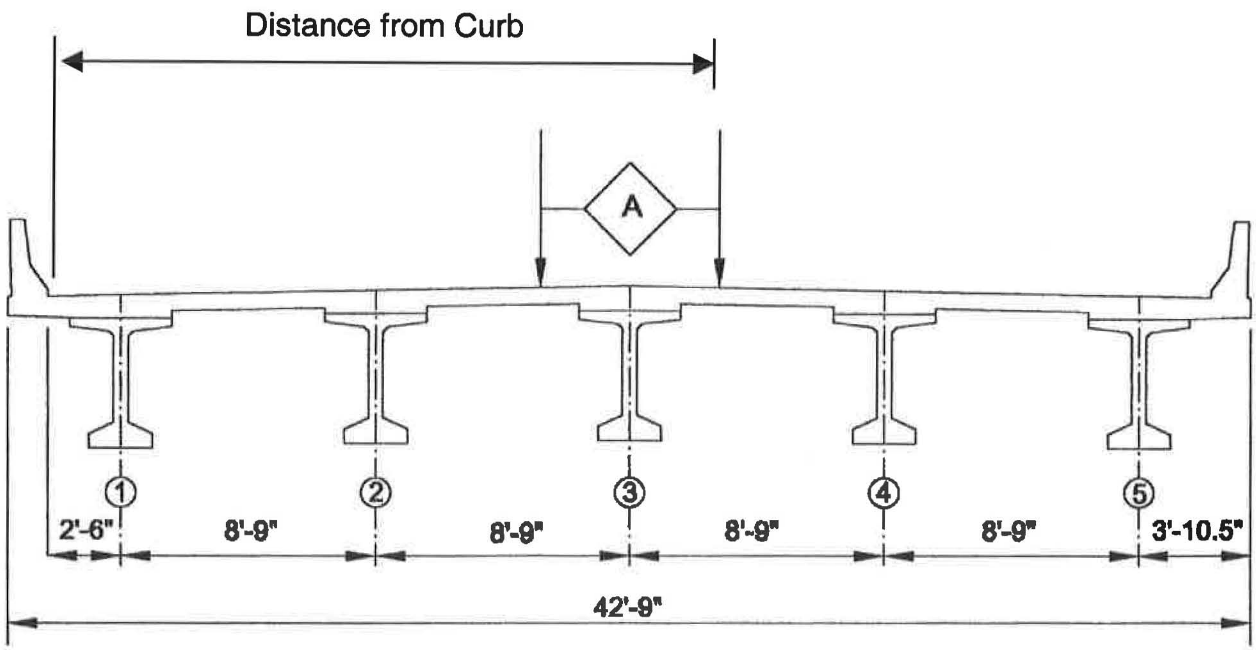

Figure 3.19. Definition of Test Truck Location for Figure 3.20 through 3.27

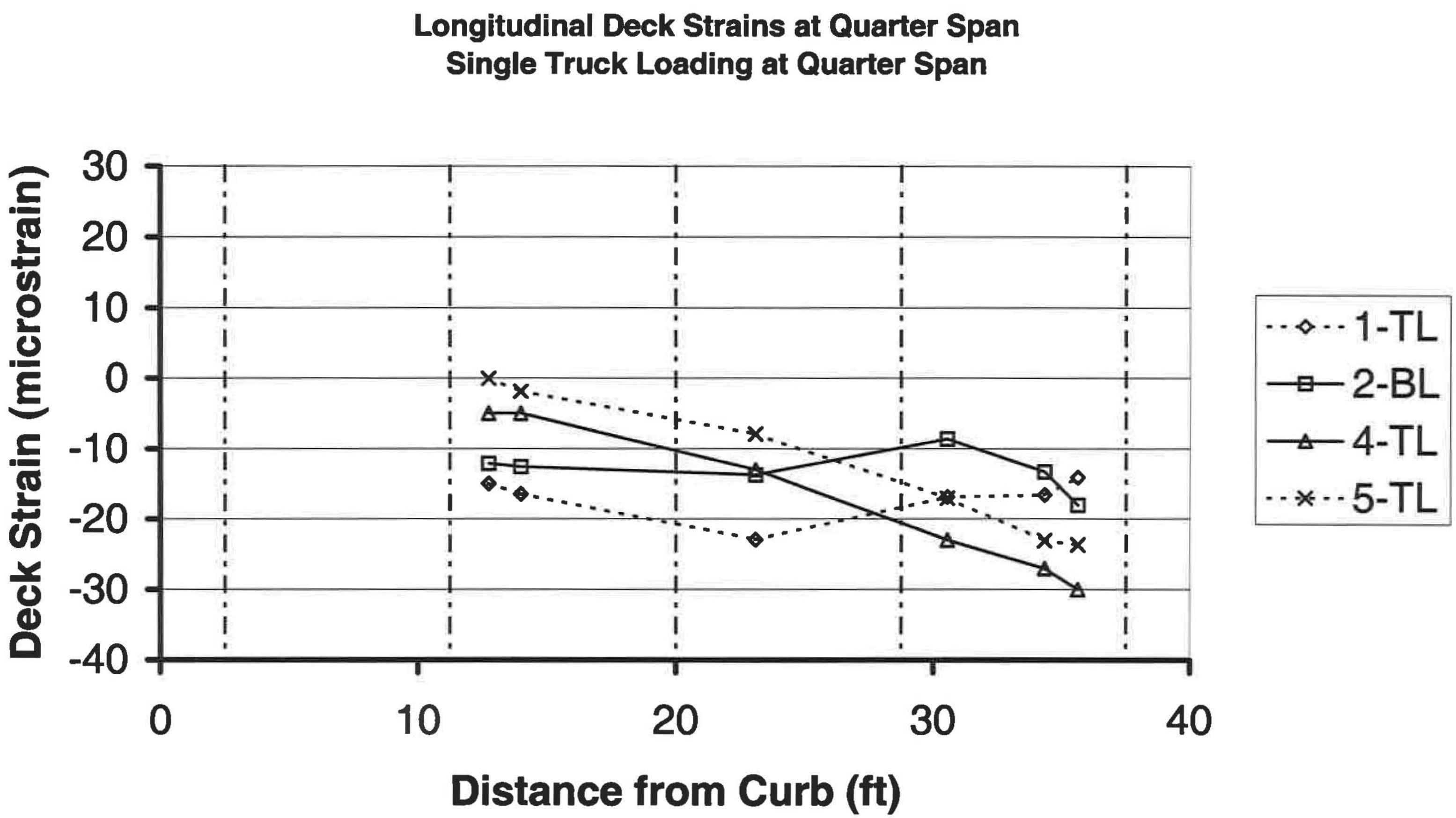

Figure 3.20. Longitudinal Deck Strains at Quarter Span from Single Truck Loading at Quarter Span

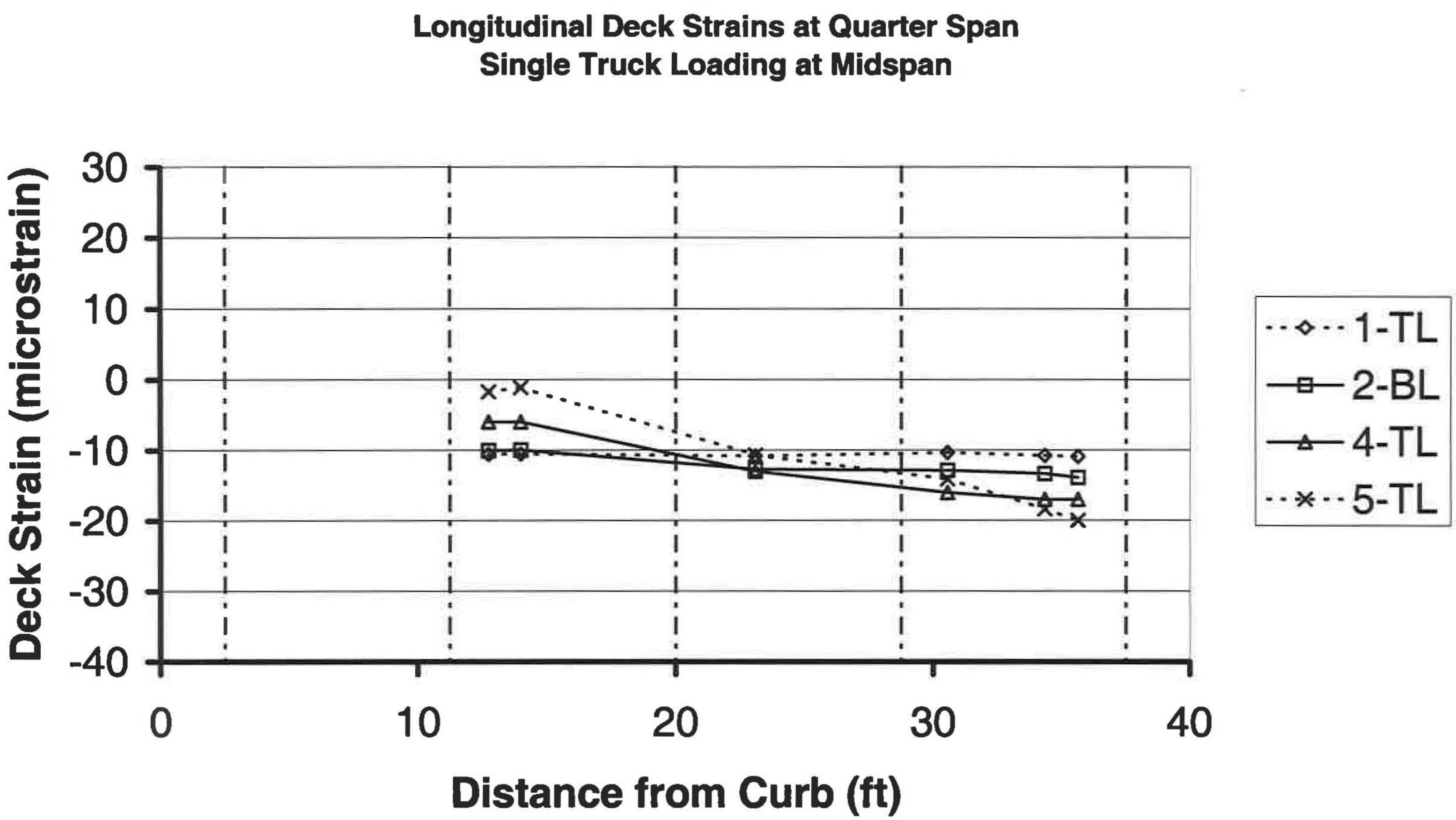

Figure 3.21. Longitudinal Deck Strains at Quarter Span from Single Truck Loading at Midspan

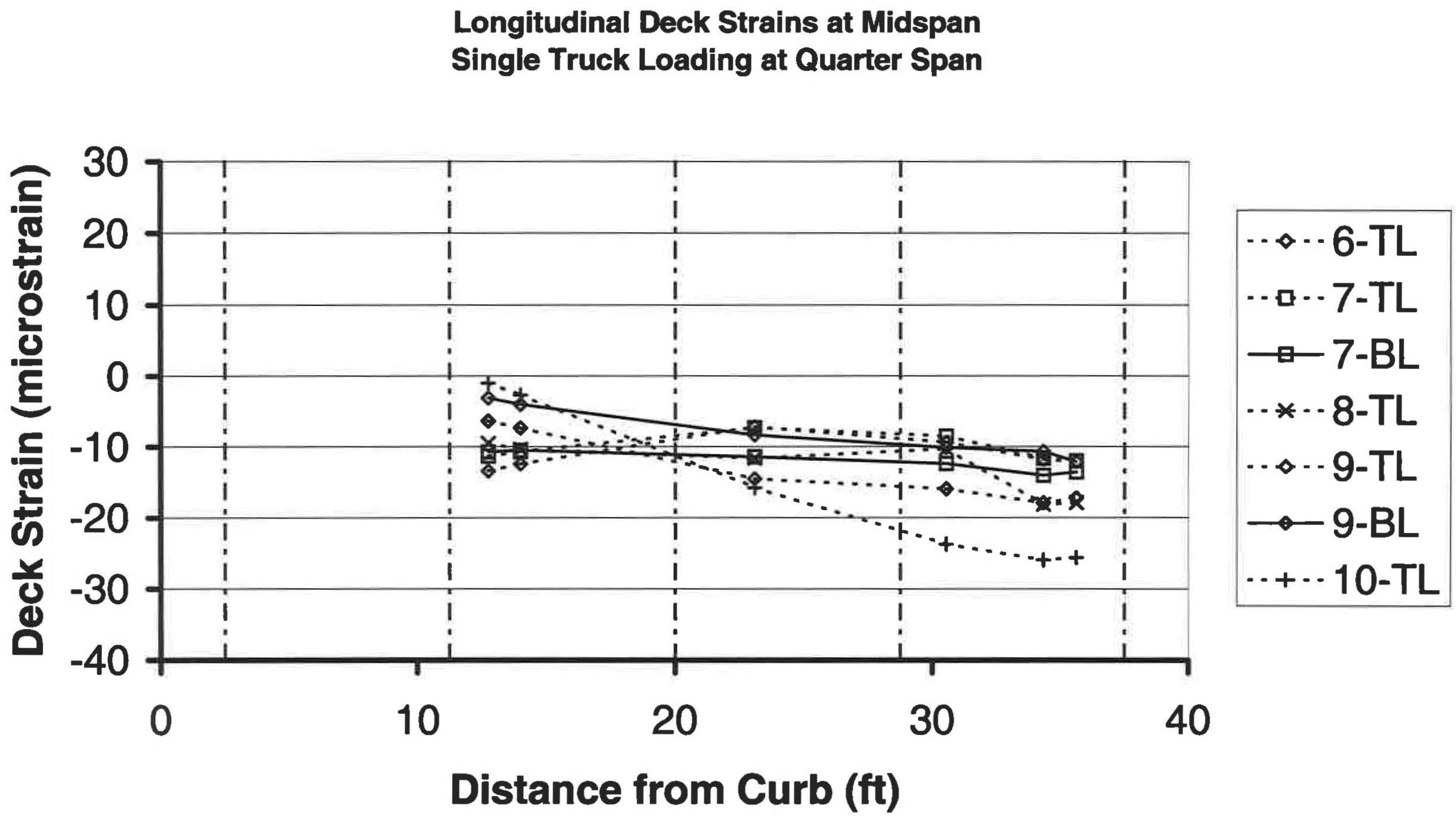

Figure 3.22. Longitudinal Deck Strains at Midspan from Single Truck Loading at Midspan

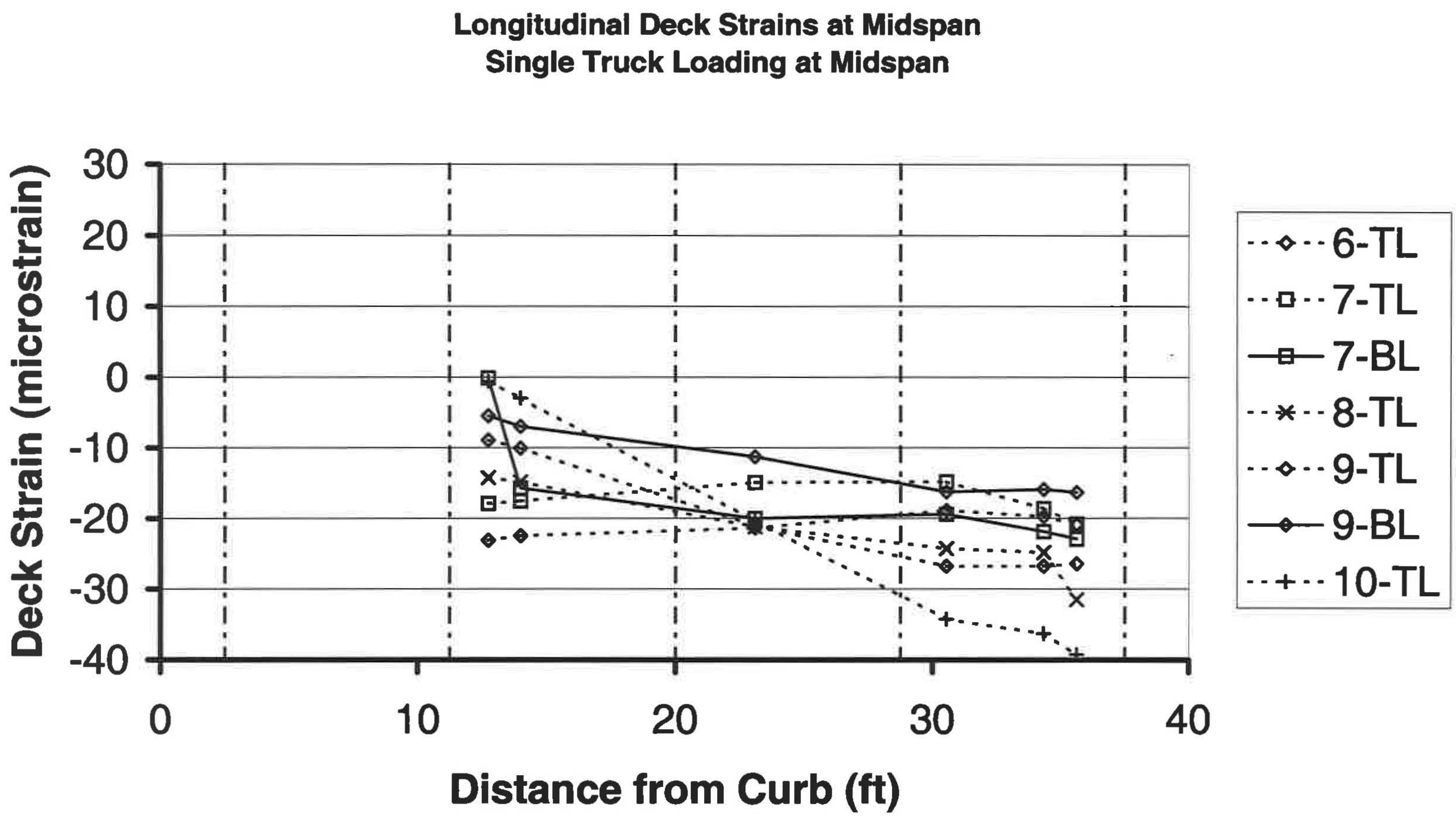

Figure 3.23. Longitudinal Deck Strains at Midspan from Single Truck Loading at Quarter Span

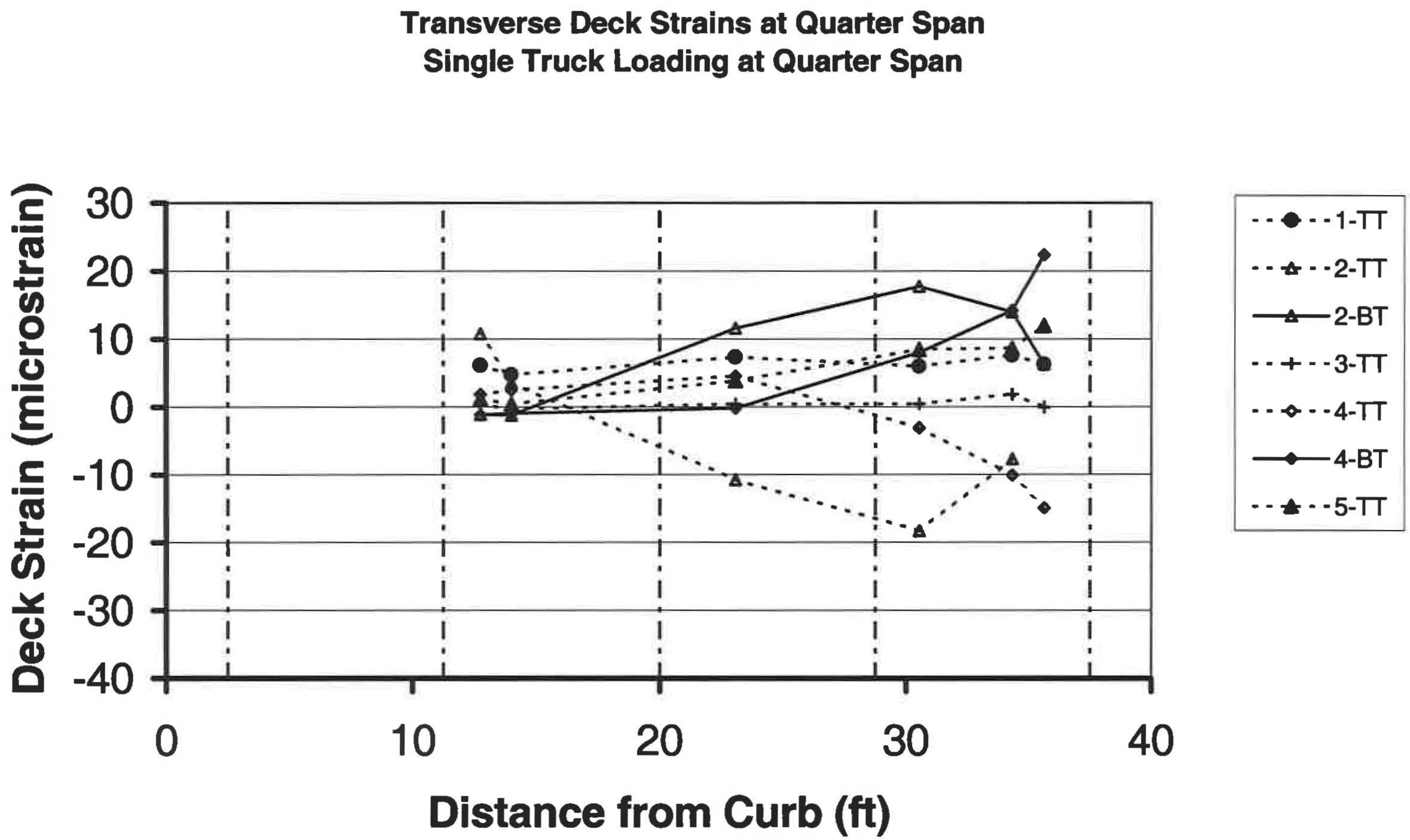

Figure 3.24. Transverse Deck Strains at Quarter Span from Single Truck Loading at Quarter Span

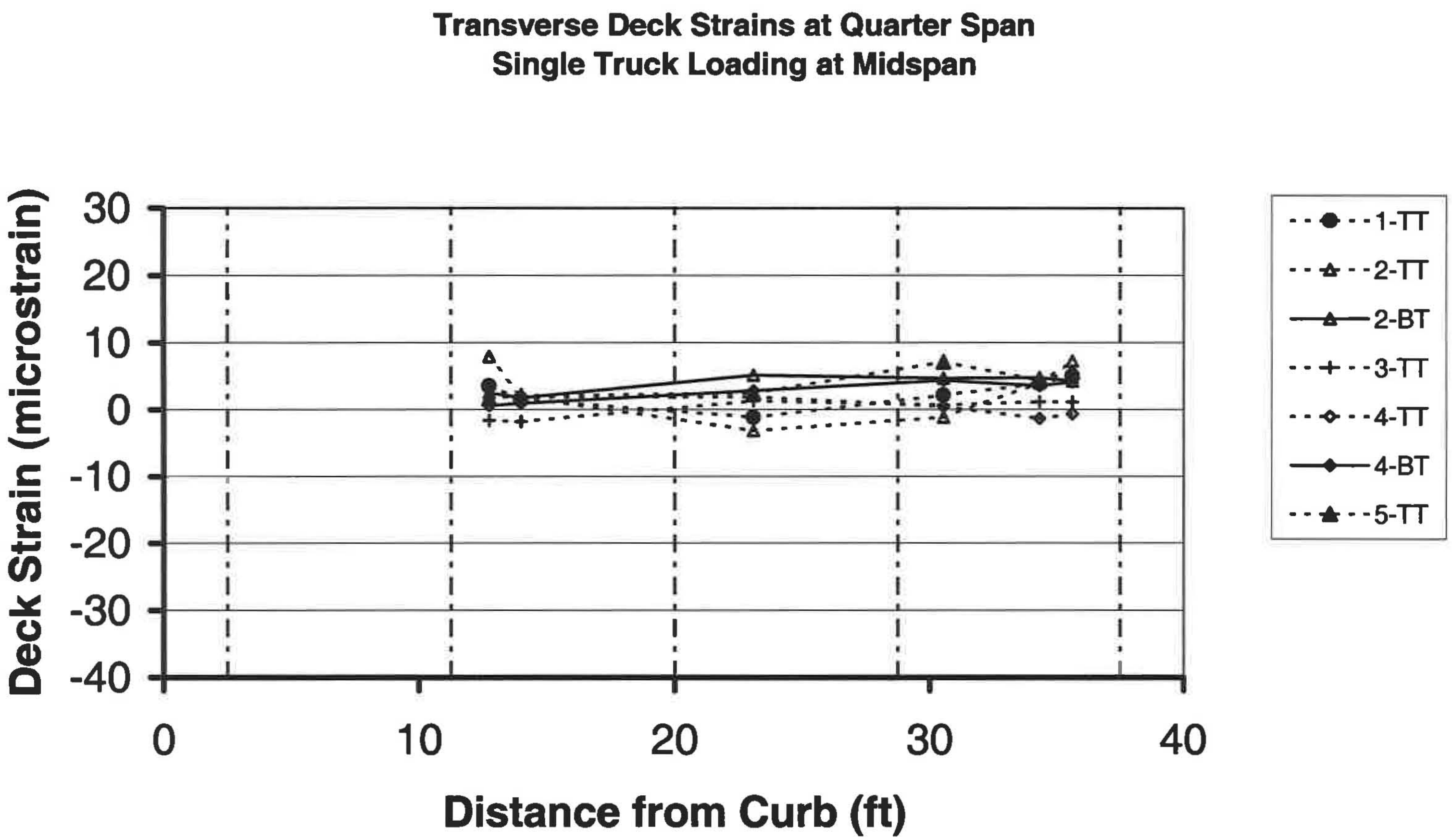

Figure 3.25. Transverse Deck Strains at Quarter Span from Single Truck Loading at Midspan

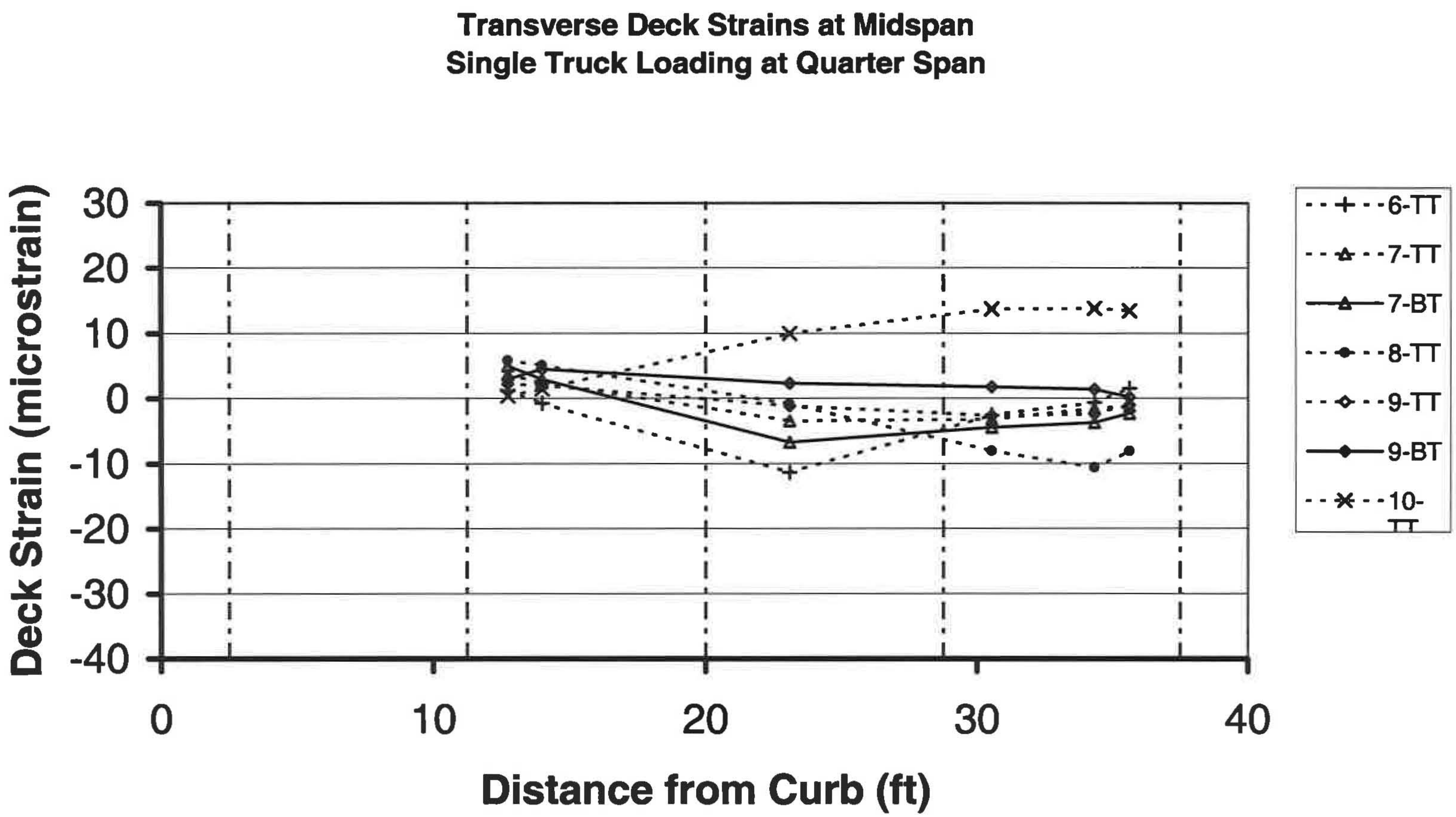

Figure 3.26. Transverse Deck Strains at Midspan from Single Truck Loading at Quarter Span

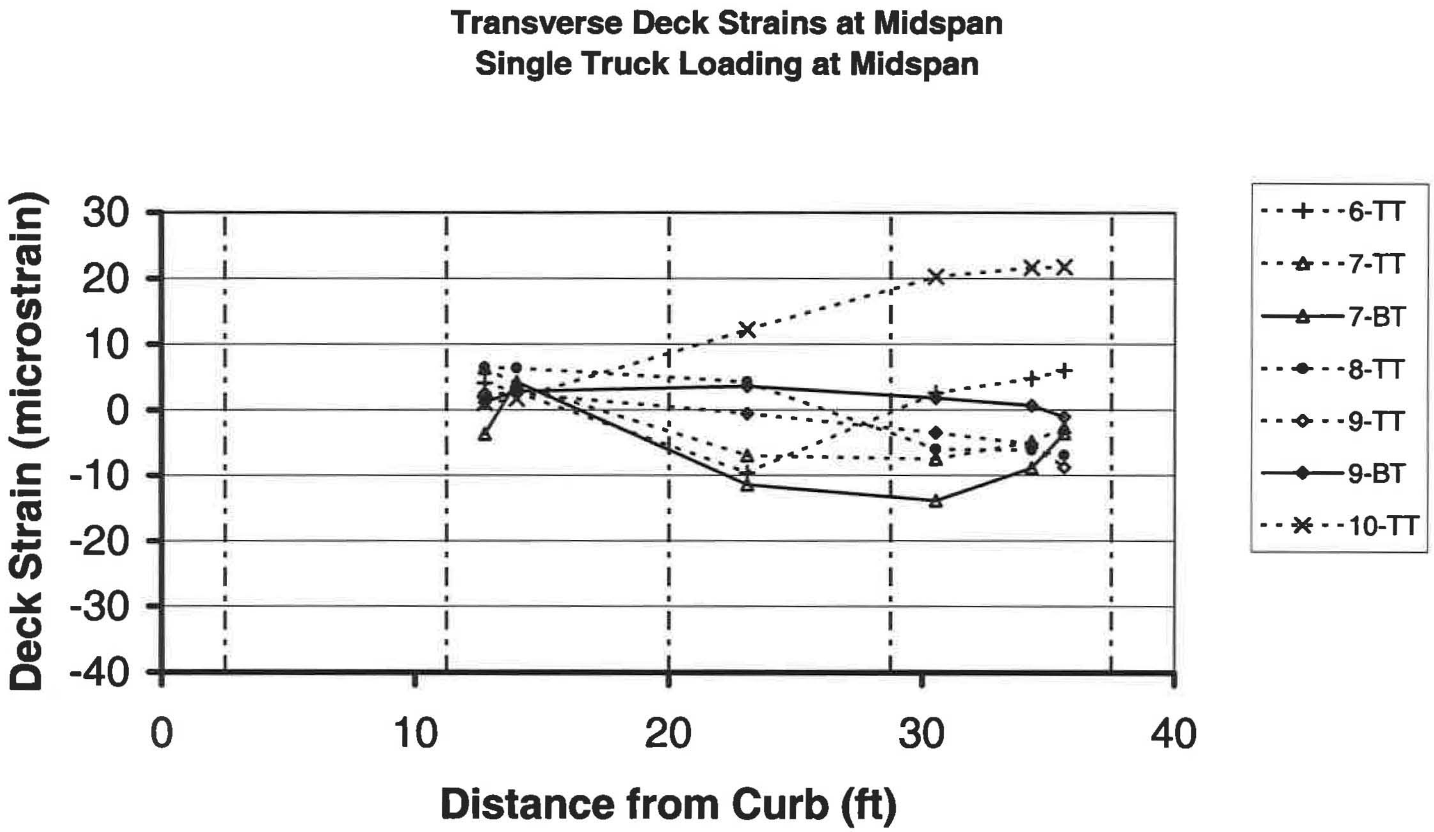

Figure 3.27.. Transverse Deck Strains at Midspan from Single Truck Loading at Midspan

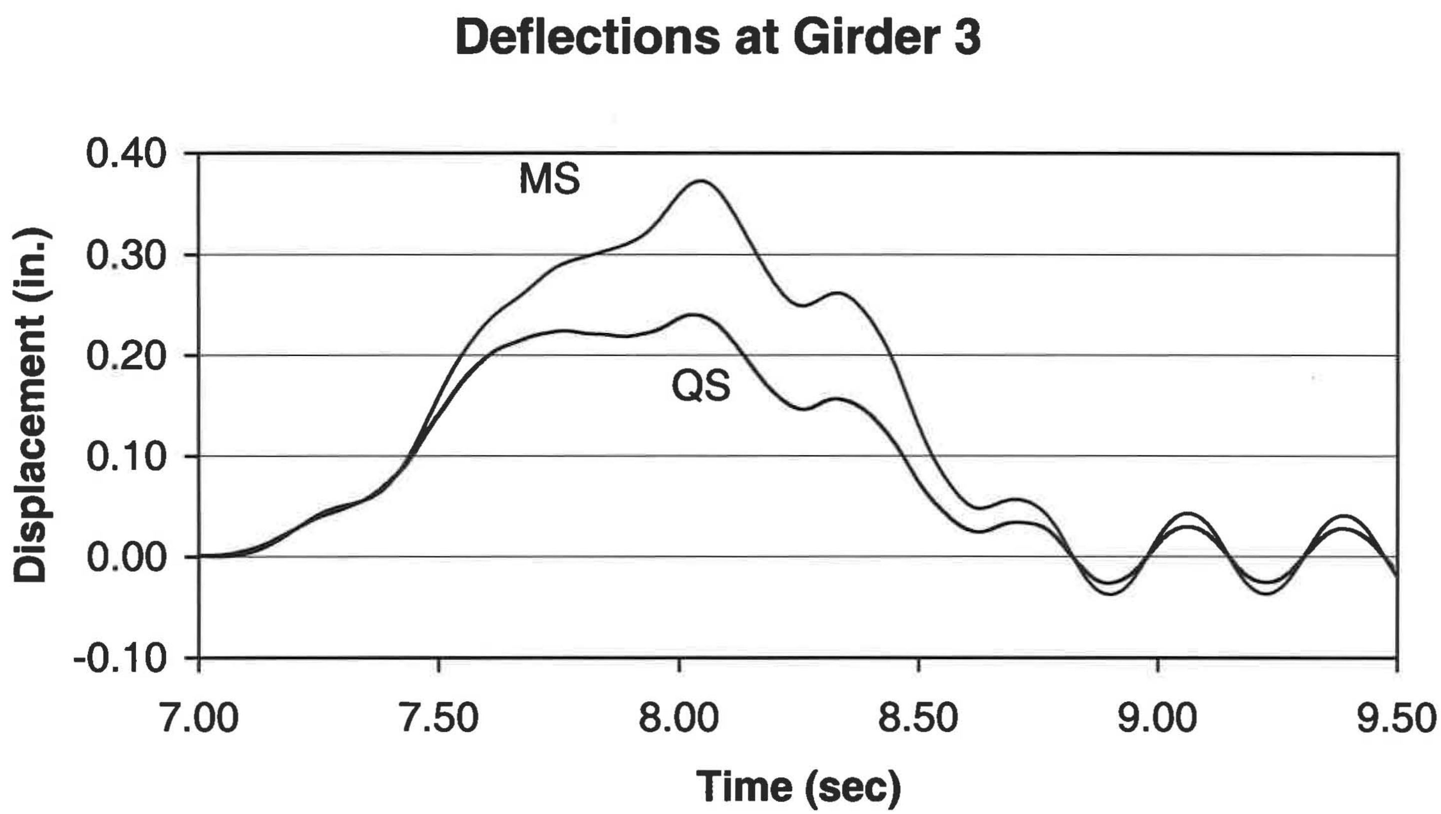

Figure 3.28. Deflections of Girder 3 Due to Northbound Test Trucks

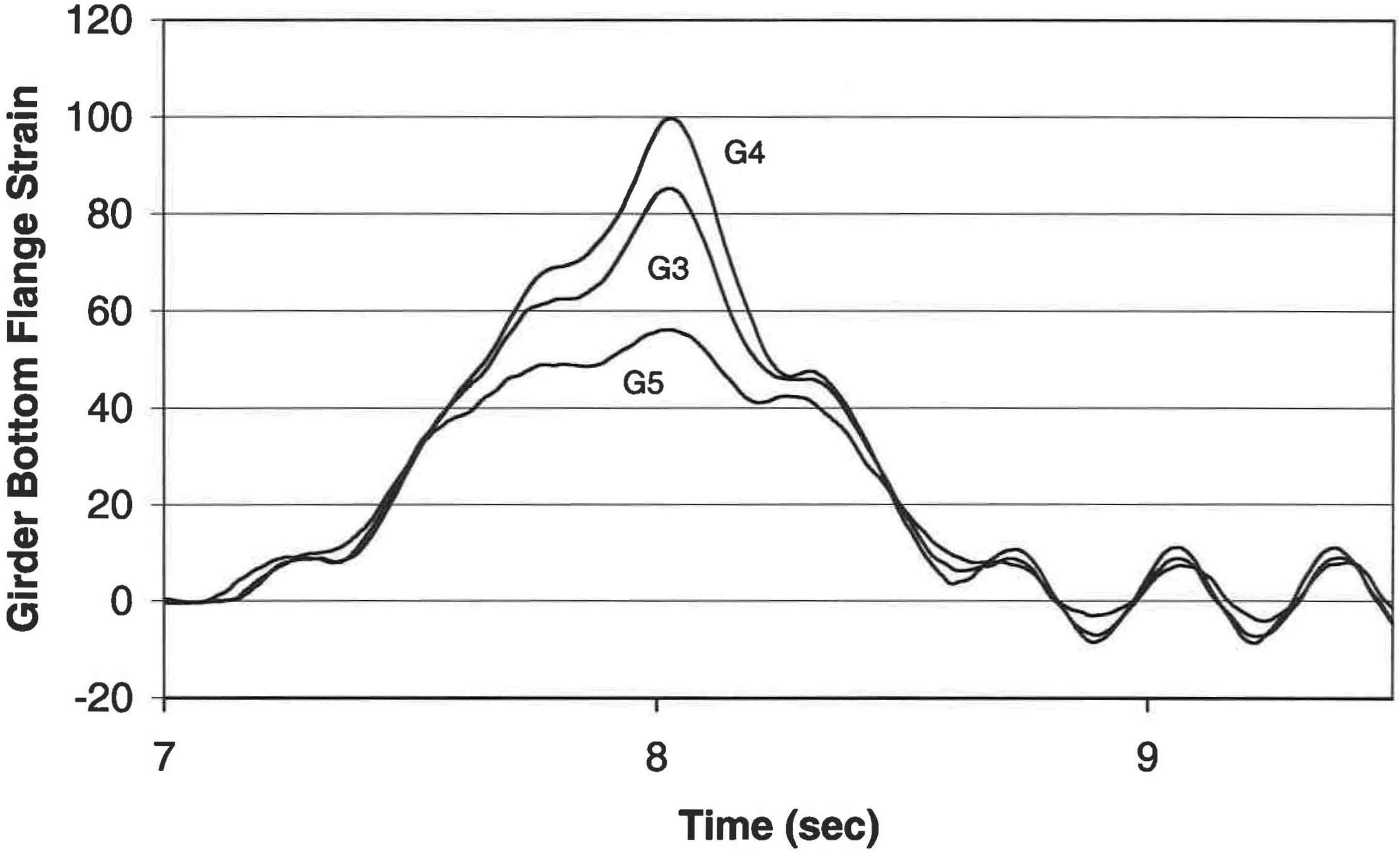

Figure 3.29. Bottom Flange Strains in Girder 3, 4 and 5 Due to Northbound Test Trucks

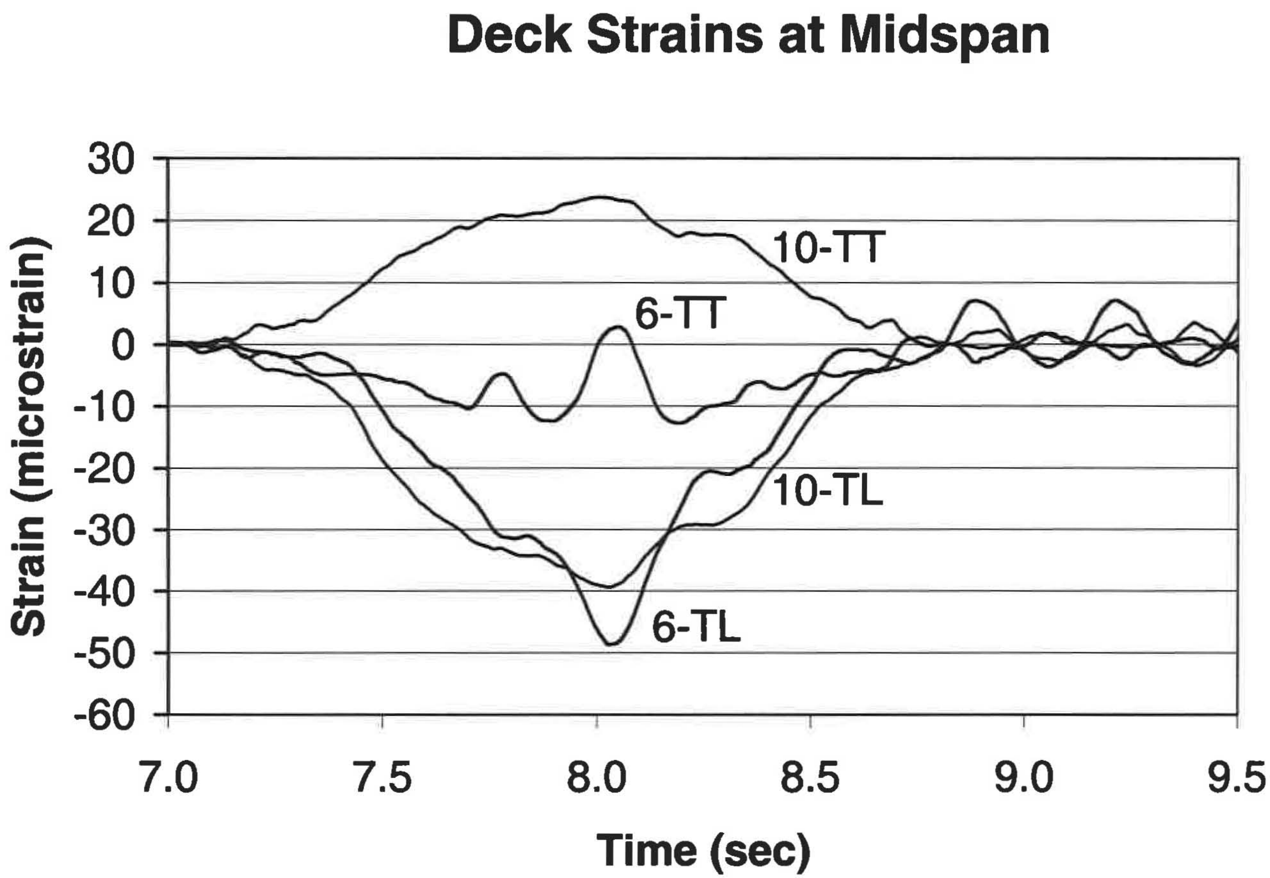

Figure 3.30. Deck Strains at Midspan Gage Locations at 6 and 10 Due to Northbound Test Trucks

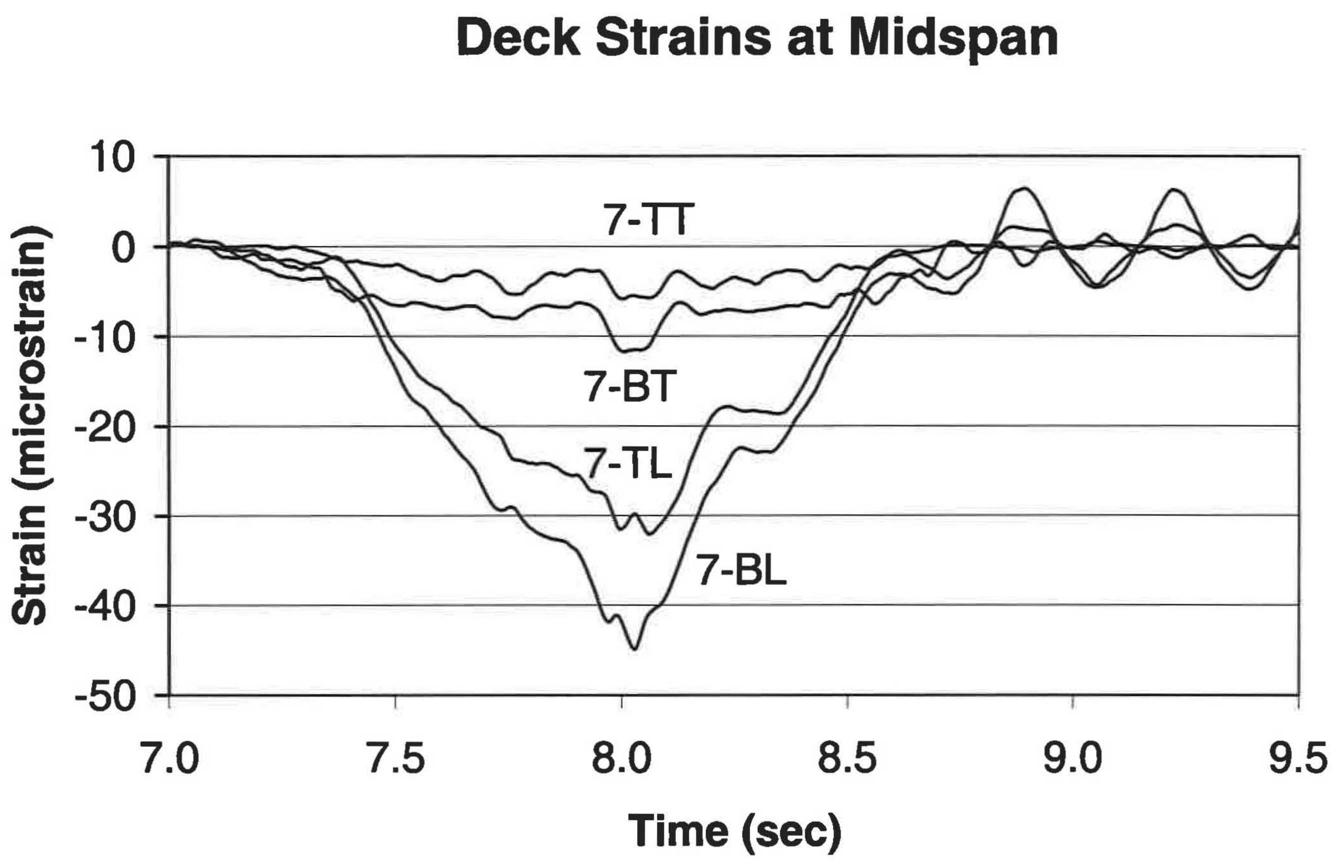

Figure 3.31. Deck Strains at Midspan Gage Location 7 Due to Northbound Test Trucks

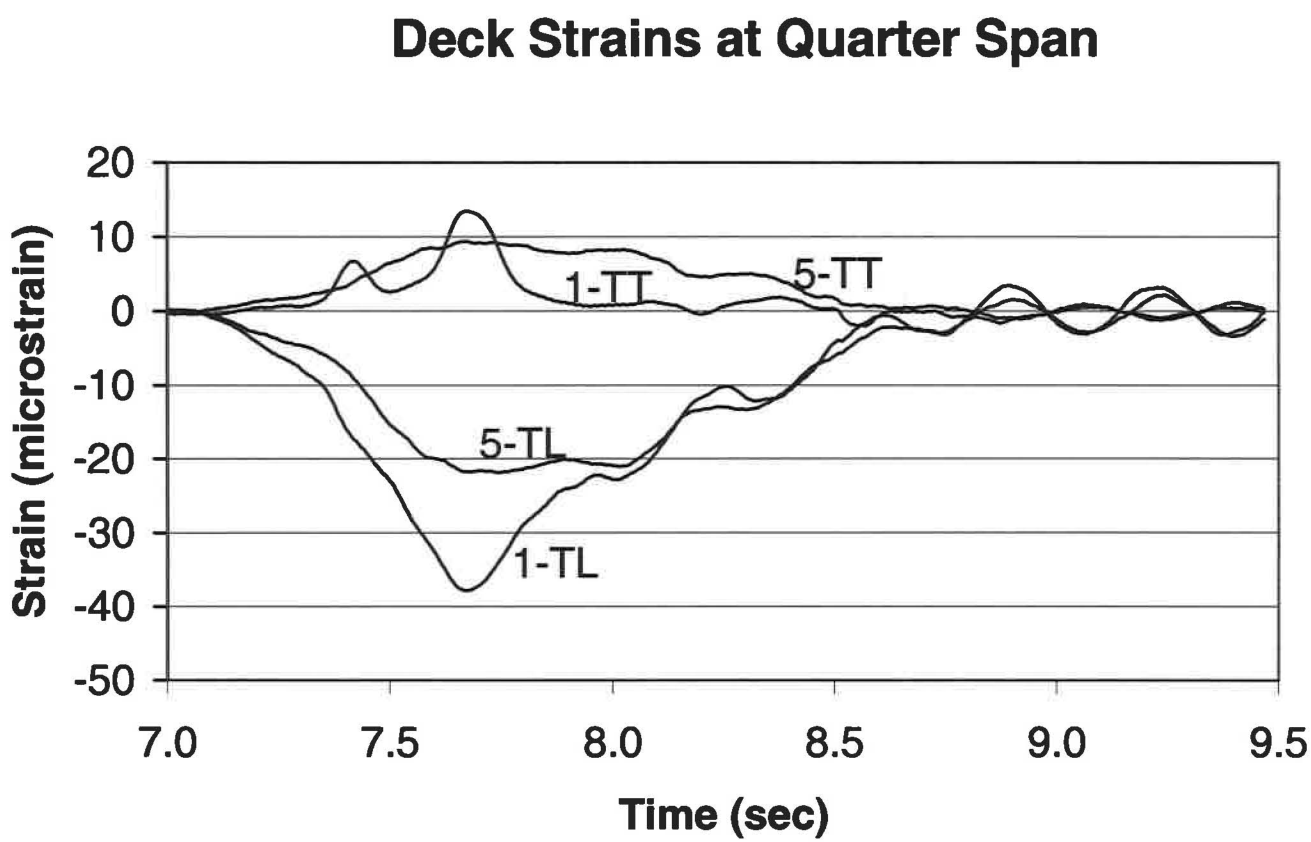

Figure 3.32. Deck Strains at Quarter Span Gage Locations 1 and 5 Due to Northbound Test Trucks

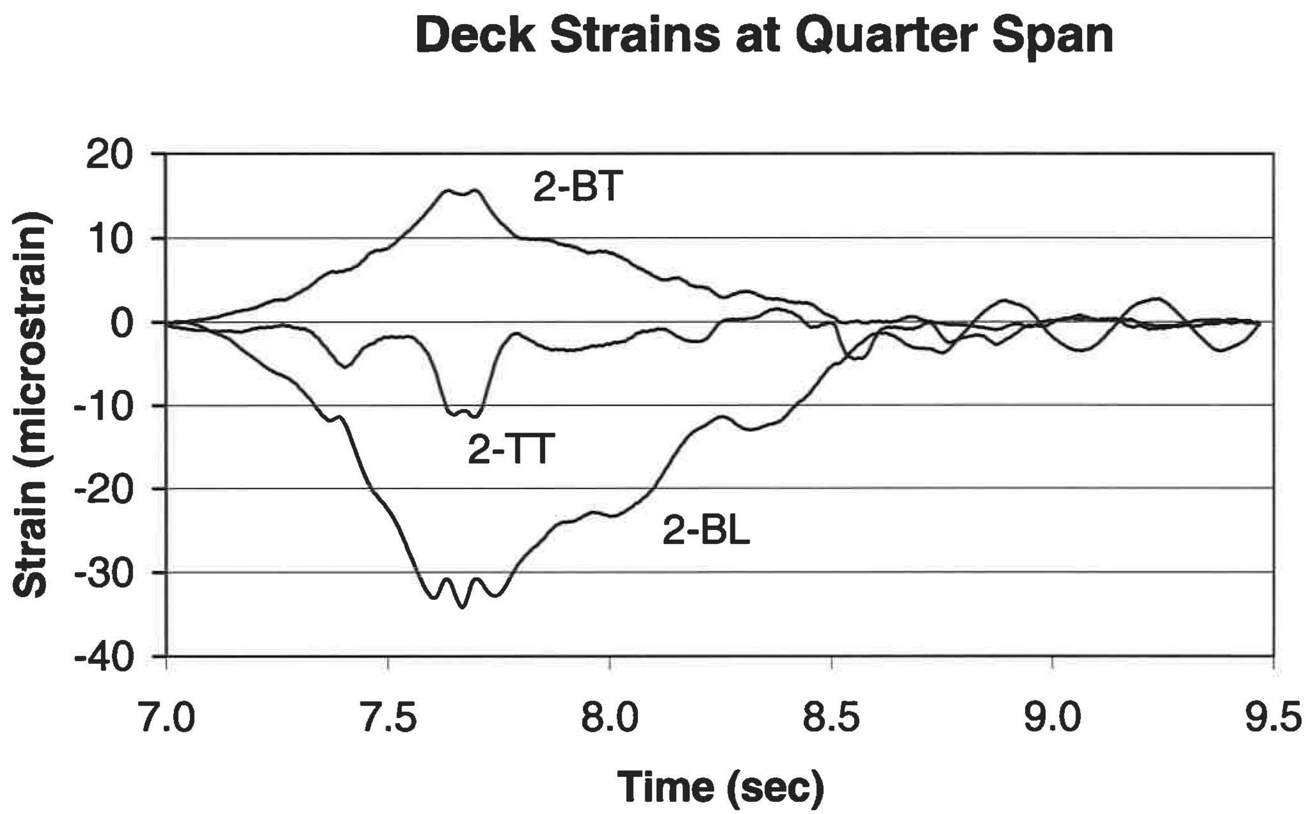

Figure 3.33. Deck Strains at Quarter Span Gage Locations 2 Due to Northbound Test Trucks

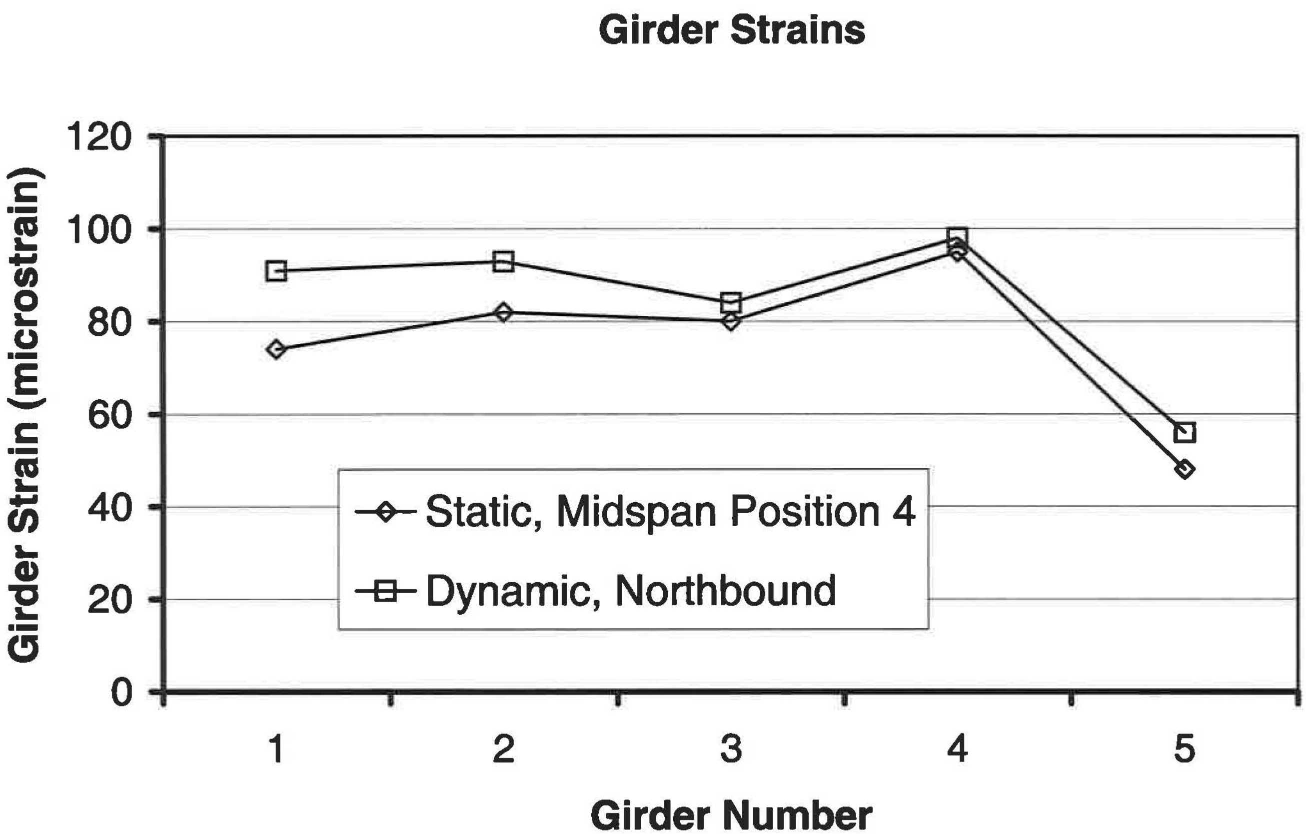

Figure 3.34. Static and Peak Dynamic Strains at Midspan

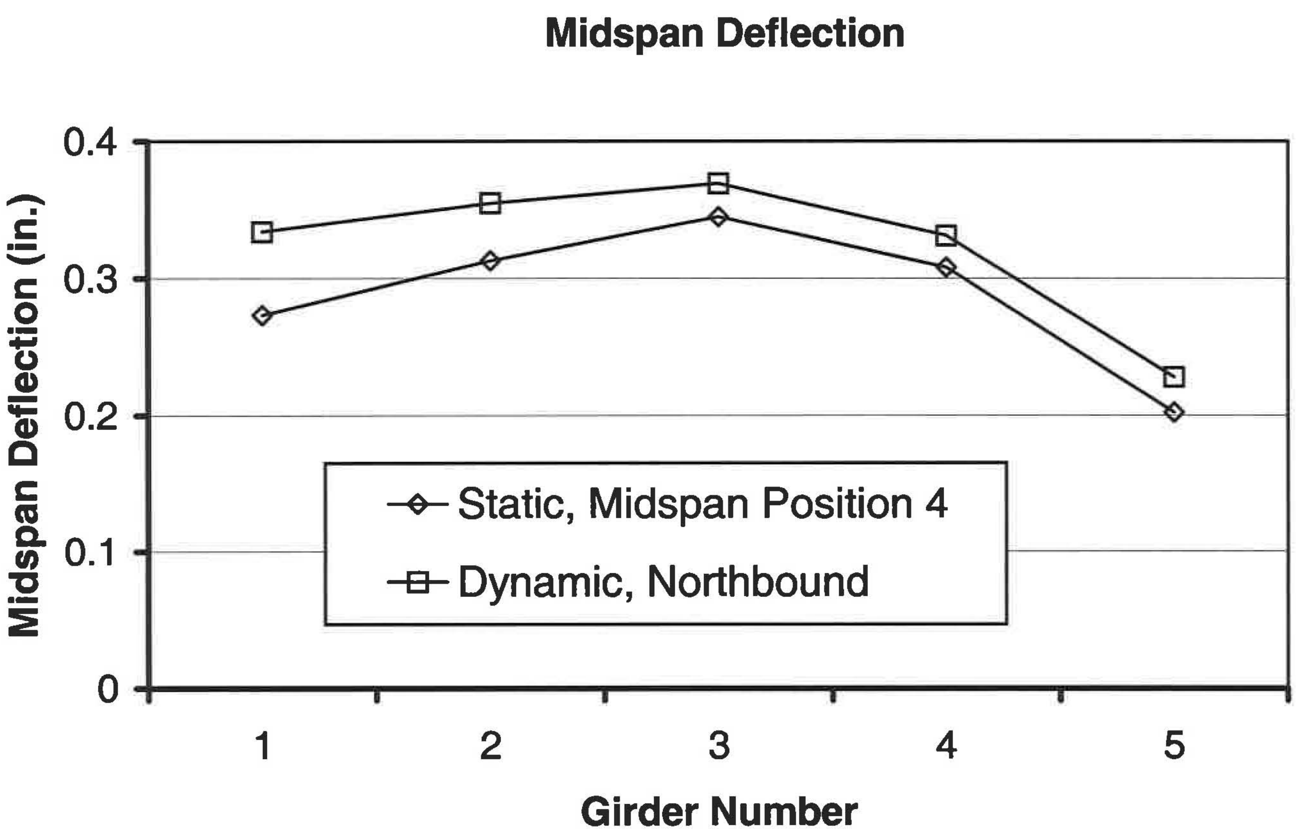

Figure 3.35. Static and Peak Dynamic Deflections at Midspan

CHAPTER FOUR

COMPARISON OF TEST AND CALCULATION RESULTS

The goal of this chapter is to compare the measurements made in the live load tests to values calculated using AASHTO design methods. Truck load distribution factors calculated using AASHTO methods are compared to distribution factors derived from the strain data from the tests. Stresses, strains, and deflections calculated for the test truck loading are also compared to measured values.

BENDING MOMENT DUE TO TEST TRUCKS

Bending moments due to a single test truck (see Figure 3.1) were calculated at midspan and quarter span for the midspan and quarter span truck positions used in the live load tests. These bending moments are shown in Figure 4.1.

DISTRIBUTION FACTORS

Distributions factors were used in a simplified bridge analysis to assign a portion of the calculated bending moments shown in Figure 4.1 was distributed to each composite girder cross-section. Distribution factors were calculated using the AASHTO *Standard Specifications* (AASHTO *Standard Specifications for Highway Bridges* (1996)) and AASHTO *LRFD* (AASHTO *LRFD Bridge Design*

Specifications (1994)). In the AASHTO *Standard Specifications*, Table 3.23.1 states that the distribution factor for interior girders of a bridge with two or more traffic lanes and girder spacing less than 14 ft is:

$$DF = \frac{S}{5.5} \qquad (1)$$

where:

DF = Distribution Factor

S = Spacing between girders (ft)

Because this distribution factor is defined for application to the bending moment produced by one wheel-line of truck loading, the value must be halved to facilitate a proper comparison to the truck load distribution factors obtained using AASHTO *LRFD*. For the bridge span subjected to live load tests, half the distribution factor calculated by Eqn. 1 is 0.795.

Distribution factors for exterior girders are found by assuming the deck between interior girders transfers load in the transverse direction as a series of simple spans and by allowing the deck to cantilever over the exterior girder. This is procedure is commonly referred to as the lever rule. For the bridge span subjected to live load tests, the truck load distribution factor for an exterior girder by the lever rule is 0.705. This distribution factor corresponds to the test loading Position 1 as defined in Figure 3.2. Because of the girder spacing, only one test truck contributes to the exterior girder distribution factor.

AASHTO *LRFD* gives the distribution factor for interior girders of prestressed concrete girder bridges in Table 4.6.2.2.2b-1 as follows:

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \quad (2)$$

DF = Distribution Factor

S = Spacing between girders (ft)

L = Span Length (ft)

t_s = Depth of concrete slab (in.)

K_g = Longitudinal Stiffness Parameter

where:

$$K_g = n\left(I + Ae_g^2\right) \quad (3)$$

n = Modular ratio between beam and deck materials

I = Moment of inertia of beam ($in.^4$)

e_g = distance between the centers of gravity between girder and deck

A = Area of girder

Using the geometric characteristics of the bridge span subjected to live load tests, the distribution factor by Eqn. 2 is 0.681.

The distribution factor for the exterior girders by AASHTO *LRFD* when two or more design lanes are loaded is given by:

$$DF_{ext} = eDF_{\text{int}} \quad (4)$$

DF_{ext} = Distribution Factor for exterior girder

DF_{int} = Distribution Factor for interior girder

e = Correction Factor

where:

$$e = 0.77 + \frac{d_e}{9.1} \geq 1.0 \qquad (5)$$

d_e = Distance between the center of exterior girder and the interior edge of traffic barrier (ft)

Using the geometric characteristics of the bridge span subjected to live load tests, the distribution factor by Eqn. 4 is 0.712.

AASHTO *LRFD* also includes a special analysis for the computation of the distribution factor for exterior girders of bridges with diaphragms or cross-frames. The special analysis assumes the bridge cross section deflects and rotates as a rigid cross section, and the distribution factor for the exterior girder is the reaction on the exterior girder produced by the design loading in the design lanes. The reaction is given by the following equation:

$$R = \frac{N_L}{N_b} + \frac{x_{ext} \sum^{N_L} e}{\sum^{N_b} x^2} \qquad (6)$$

R = Reaction on exterior girder in number of lanes

N_L = Number of loaded lanes

N_b = Total number of girders in span

e = Eccentricity of a lane from the center of gravity of the pattern of girders

x_{ext} = Horizontal distance from the center of gravity of the pattern of girders to each girder (ft)

The larger distribution factor from either Eqn. 4 or Eqn. 6 is used for design. The distribution factor of Eqn. 6 is also subject to reduction by multi-presence factors when there are more than two design lanes. Results from Eqn. 6 for the span subjected to live load tests are summarized in Table 4.1. Truck positions used in Eqn. 6 are the truck positions used in the live load tests as defined in Figure 3.2 and combinations of the load positions as defined in Figure 3.4. Multi-presence factors were not applied in this report since multiple trucks were actually present during the tests.

Distribution factors were calculated from the strains measured with the electrical resistance strain gages in the bottom flange of the girders by using the following equation:

$$DF = \frac{n w_i \varepsilon_i}{\sum_{j=1}^{k} \varepsilon_j w_j} \qquad (7)$$

DF = Distribution Factor

n = Number of lanes loaded, 3 for cases considered here

ε_i = Strain in i^{th} girder

k = Numbers of girders

ε_j = Strain in j^{th} girder

w_j = Ratio of section modulus of j^{th} girder to typical interior section modulus

To use Eqn. 7, the section modulus of the interior and exterior composite girder sections is used to calculate the ratio w. For interior girders, the ratio is equal to

1. For exterior girders of the span considered here, the ratio w is 0.996 if the barrier rail is neglected and 1.14 if the barrier rail is included in the composite cross section for the exterior girder. It has been common in past research to use w = 1 for all girders since the barriers are generally not included as part of the composite cross section in design of the exterior girders. That approach is taken here and distribution factors are only reported for w=1 for all girders.

A summary of the distribution factors calculated from the strain measurements is provided in Table 4.2. Distribution factors for exterior girders for loadings with two trucks at midspan, Positions 1, 2, and 3, are provided for comparison with results of the AASHTO *LRFD* special analysis. The other AASHTO methods for calculating distribution factors correspond to loading in all design lanes. There are three design lanes in the 40 ft roadway width of the span subjected to live load tests. To simulate loading in three lanes, the sum of the strains for combinations of truck positions 1 + 5, 2 + 6, and 3 + 5 (see Figure 3.4) were used to calculate distribution factors.

COMPARISON OF DISTRIBUTION FACTORS

The largest distribution factor at an exterior girder listed in Table 4.2 for each load position is shown in Table 4.1 under the heading "Tests." This provides a comparison in Table 4.1 of the distribution factors from the test results to those from the AASHTO *LRFD* special analysis. Generally it is seen that the special analysis overestimates the distribution factors from the test. For all cases shown in Table 4.1, the calculated distribution factors average 41% higher than the values determined from the measured strains.

Table 4.3 provides a comparison of distribution factors for interior and exterior girders. The largest distribution factors for interior and exterior girders calculated from the test results are 0.708 and 0.577, respectively. These values are listed in Table 4.3. The distribution factor 0.681 for an interior girder from AASHTO LRFD is 4% less than the value from the tests. This is very good agreement. The distribution factor of 0.795 from the AASHTO Standard Specifications overestimates the value from the tests by 12%. At the exterior girders, the distribution factors of 0.705 and 0.712 from the two AASHTO specifications, excluding the special analysis of *LRFD*, are approximately 22% higher than the value from the tests. The largest distribution factor for an exterior girder from the special analysis was 0.811, this value is 41% higher than the largest distribution factor for an exterior girder from the test results.

COMPARISONS OF STRAINS

Measured and calculated strains at midspan and quarter span are compared in Tables 4.4(a) and 4.4(b). The strains listed under the heading "Tests" are the sum of strains measured for load Position 1 plus Position 5. Strains are shown for the electrical resistance (ER) and vibrating wire (VW) strain gage locations in the deck and girders. Strains for an Interior Girder were measured at Girder 3, and strains at an exterior girder were measured at Girder

5. The strains measured at midspan of these girders for the combination of Position 1 plus Position 5 were the largest strains measured for all load combinations at an interior and exterior girder. Hence, these measured strains are appropriate for comparison to values calculated using the AASHTO distribution factors. At the bottom of Girder 3 and 5, strains were measured using both electrical resistance and vibrating wire gages. There is no clear reason to chose one set of measurements over the other, so the average of the two is listed in Table 4.4. Those average values are considered here to be the best estimates of the strains at the bottom of the girders, and these values are referred to in comparisons made below.

Distribution factors from the AASHTO *Standard Specifications* and AASHTO *LRFD* were applied to the bending moments shown in Figure 4.1, and the resulting bending moment was used to calculate stresses in the composite deck and girder cross section at the strain gage locations. Strains were calculated by dividing the stress by the modulus of elasticity of the transformed concrete section, 5,740,000 psi. For the exterior girder, the AASHTO *LRFD* distribution factor of 0.804 (see Table 4.1) from the special analysis for load Position 1 plus 5 was used. The rail was not included in the composite exterior girder cross section.

The largest strains were measured at the bottom of the girders, and this location typically controls the design for live load. For all cases shown in Table 4.4(a) and (b), the tension strain at the bottom of the interior and exterior girders was overestimated by the calculations. For the interior girder, the strains calculated using the distribution factor from AASHTO *LRFD* provided the best match to the measured strains. These calculated strains are within 40% of the measured strains. At the bottom of the exterior girder, the strains calculated using the distribution factor from the AASHTO *Standard Specifications* matched best with the test results. If the AASHTO *LRFD* distribution factor of 0.712 from Eqn. 4 is used instead of the one from the special analysis, all the calculated strains at the exterior girder are approximately the same as those listed in the column for the AASHTO *Standard Specifications* for which the distribution factor is 0.705. Using either of these distribution factors, 0.712 or 0.705, the calculated strains of the bottom of the exterior girders are larger than the measured strains by less than 40%.

The results in Table 4.4 justify the following conclusions for this bridge. Use of the distribution factors from AASHTO *LRFD* provides simplified bridge analysis results that are conservative, and the results are as good or better than those obtained using the AASHTO *Standard Specifications* if the AASHTO *LRFD* Special Analysis for exterior girders is not used. With this exception, strains at the bottom of the girders were larger than the measured strains by 40% or less.

COMPARISONS OF STRESSES

Measured and calculated stresses at midspan and quarter span are compared in Tables 4.5(a) and 4.5(b). The stresses shown are the stress in the concrete at the strain gage locations. Measured stresses were determined by multiplying the strains listed in Table 4.4(a) and (b) by the concrete modulus of elasticity, 5,740,000 psi for the girders and 6,650,000 psi for the deck. The tabulated stresses do not provide any new information beyond the comparisons made with strains in the previous section, but the numerical values of the stresses may be more familiar to the reader. Due to the rounding of the calculated strains in Table 4.4, the percentage differences between measured and calculated values are different in Tables 4.4 and 4.5 at some gage locations.

COMPARISONS OF DEFLECTIONS

A comparison of measured and calculated deflections is presented in Table 4.6. Deflections measured in field tests are listed for the combined loading of Position 1 plus Position 5. This load combination produced the largest deflections of the three load combinations with three trucks shown in Figure 3.4. Measured deflections are also shown in Table 4.6 for the symmetrical loading of Position 4 with two trucks. For each of the loadings, the maximum deflection measured at an individual girder is listed, and the average deflection measured at the five girders is also listed. Average deflections are not listed at the quarter span because quarter span deflections were measured at only three of the five girders.

The calculated deflections for each case are larger than the measured deflections. The deflections calculated using the smallest distribution factors match best with the measured values. AASHTO *LRFD* suggests that live load deflections be calculated by assuming that all girders deflect the same amount. This corresponds to the use of a distribution factor equal to the number of loaded lanes, N_L, divided by the number of girders, N_b. Values for this method are shown in rows labeled "(N_L/N_b)" for three lanes loaded, corresponding to Position 1 plus Position 5, and for two lanes loaded, corresponding to Position 4.

The deflections calculated using (N_L/N_b) as the distribution factor match well with the maximum measured deflections; for all cases the difference is within 20% of the measured deflection. The deflections calculated using (N_L/N_b) as the distribution factor match within 42% of the average midspan deflection for each case.

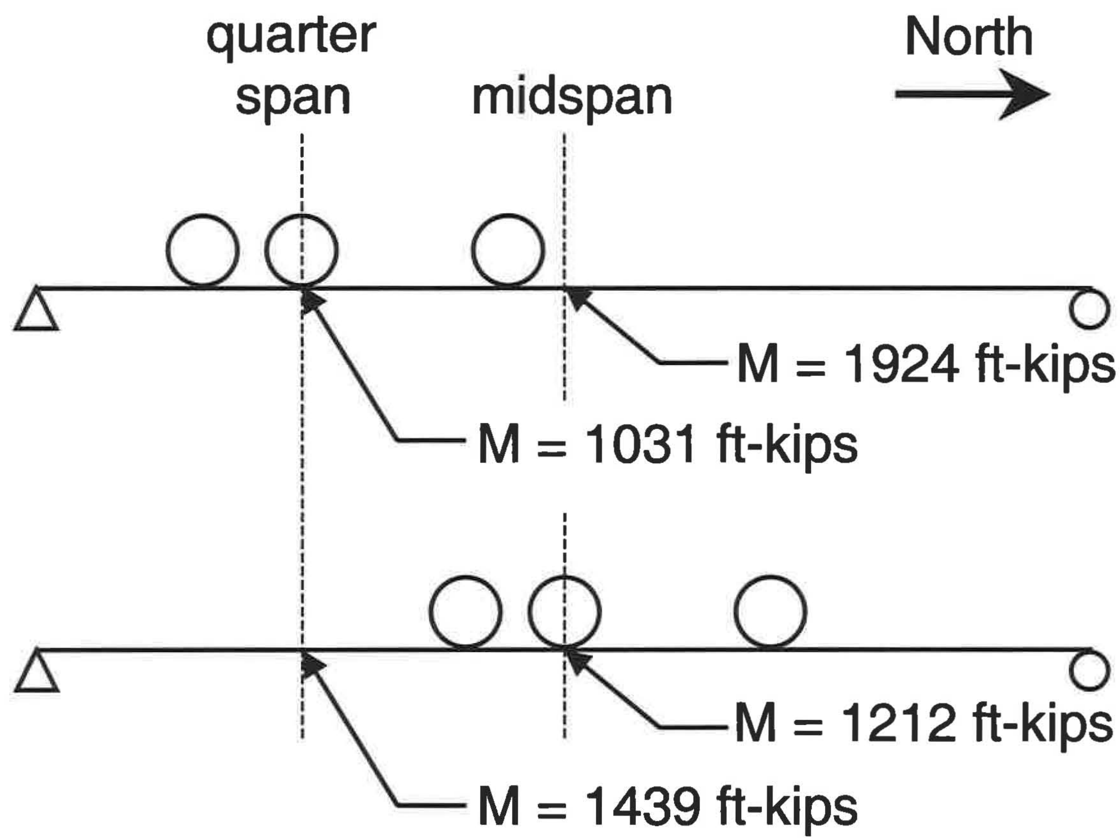

Figure 4.1. Bending Moments due to Test Truck

Table 4.1. Truck Load Distribution Factors for Exterior Girders from AASHTO *LRFD* Special Analysis

Load Position	AASHTO *LRFD* Distribution Factor	Tests
1	0.811, $N_L = 2$	0.563
2	0.716, $N_L = 2$	0.464
3	0.770, $N_L = 2$	0.522
1 + 5	0.804, $N_L = 3$	0.572
2 + 6	0.680, $N_L = 3$	0.577
3 + 5	0.726, $N_L = 3$	0.535

Table 4.2. Distribution Factors Calculated using Data from Bottom Flange Electrical Resistance Strain Gages

Load Position	Girder Number				
	1	2	3	4	5
M1	0.108	0.284	0.602	0.443	0.563
M2	0.160	0.315	0.591	0.470	0.464
M3	0.129	0.292	0.601	0.455	0.522
M1 + M5	0.501	0.577	0.708	0.642	0.572
M2 + M6	0.577	0.609	0.683	0.667	0.464
M3 + M5	0.519	0.584	0.708	0.654	0.535

Table 4.3 Comparison of Distribution Factors

Method	Interior Girder	Exterior Girder
AASHTO *LRFD*	0.681	0.712/0.811*
AASHTO *Std. Spec.*	0.795	0.705
Tests	0.708	0.577

* Special analysis

Table 4.4 (a) Strains on Composite Cross Sections at Midspan

Location	Test Pos. 1 + 5	Standard Spec.s		LRFD	
		Calc.	% Diff.	Calc.	% Diff.
a) Interior Girder, Trucks at Midspan					
Deck, Top (ER)	-60	-70	17	-60	0
Girder, Top (VW)	-20	-20	0	-20	0
Girder, Bot. (VW)	120	190	58	160	33
Girder, Bot. (ER)	130	190	46	160	23
Girder, Bot. (Ave.)	125	190	52	160	28
b) Exterior Girder, Trucks at Midspan					
Deck, Top (ER)	-80	-70	-13	-70	-13
Girder, Top (VW)	-30	-20	-33	-20	-33
Girder, Bot. (VW)	130	170	31	190	46
Girder, Bot. (ER)	130	170	31	190	46
Girder, Bot. (Ave.)	130	170	31	190	46
c) Interior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-30	-40	33	-40	33
Girder, Top (VW)	-10	-10	0	-10	0
Girder, Bot. (VW)	70	120	71	100	43
Girder, Bot. (ER)	80	120	50	100	25
Girder, Bot. (Ave.)	75	120	60	100	33
d) Exterior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-50	-40	-20	-50	0
Girder, Top (VW)	-20	-10	-50	-20	0
Girder, Bot. (VW)	90	110	22	120	33
Girder, Bot. (ER)	70	110	57	120	71
Girder, Bot. (Ave.)	80	110	38	120	50

Table 4.4 (b) Strains on Composite Section at Quarter Span

Location	Test Pos. 1 + 5	Standard Spec.s		LRFD	
		Calc.	% Diff.	Calc.	% Diff.
a) Interior Girder, Trucks at Midspan					
Deck, Top (ER)	-30	-40	33	-30	0
Girder, Top (VW)	-10	-10	0	-10	0
Girder, Bot. (VW)	50	80	60	70	40
b) Exterior Girder, Trucks at Midspan					
Deck, Top (ER)	-40	-40	0	-40	0
Girder, Top (VW)	-10	-10	0	-10	0
Girder, Bot. (VW)	60	70	17	80	33
c) Interior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-50	-50	0	-50	0
Girder, Top (VW)	-10	-20	100	-10	0
Girder, Bot. (VW)	80	110	38	90	13
d) Exterior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-50	-50	0	-60	20
Girder, Top (VW)	-10	-20	100	-20	100
Girder, Bot. (VW)	80	90	13	110	38

Table 4.5 (a) Stresses on Composite Cross Section at Midspan

Location	Test Pos. 1 + 5	**Standard Spec.s**		*LRFD*	
		Calc.	% Diff.	Calc.	% Diff.
a) Interior Girder, Trucks at Midspan					
Deck, Top (ER)	-399	-467	17	-400	0
Girder, Top (VW)	-115	-125	9	-107	-7
Girder, Bot. (Ave.)	718	1093	52	936	30
b) Exterior Girder, Trucks at Midspan					
Deck, Top (ER)	-532	-432	-19	-492	-8
Girder, Top (VW)	-172	-122	-29	-139	-19
Girder, Bot. (Ave.)	746	973	30	1110	49
c) Interior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-200	-294	47	-252	26
Girder, Top (VW)	-57	-97	38	-67	17
Girder, Bot. (Ave.)	431	689	60	590	37
d) Exterior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-333	-272	-18	-310	-7
Girder, Top (VW)	-115	-77	-33	-88	-23
Girder, Bot. (Ave.)	459	613	33	699	52

Table 4.5. (b) Stresses on Composite Cross Section at Quarter Span

Location	Test Pos. 1 + 5	Standard Spec.s		LRFD	
		Calc.	% Diff.	Calc.	% Diff.
a) Interior Girder, Trucks at Midspan					
Deck, Top (ER)	-200	-250	25	-214	7
Girder, Top (VW)	-57	-67	17	-57	-1
Girder, Bot. (VW)	287	433	51	371	29
b) Exterior Girder, Trucks at Midspan					
Deck, Top (ER)	-266	-231	-13	-264	-1
Girder, Top (VW)	-57	-66	15	-75	31
Girder, Bot. (VW)	344	384	11	438	27
c) Interior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-333	-349	5	-299	-10
Girder, Top (VW)	-57	-93	62	-80	39
Girder, Bot. (VW)	459	605	32	518	13
d) Exterior Girder, Trucks at Quarter Span					
Deck, Top (ER)	-333	-323	-3	-368	11
Girder, Top (VW)	-57	-91	59	-104	81
Girder, Bot. (VW)	459	537	17	612	33

Table 4.6 Calculated and Measured Deflections

Method	Distribution Factor	Trucks at Midspan: Midspan Defl., in.	Trucks at Midspan: Quarter Span Defl., in.	Trucks at Quarter Span: Midspan Defl., in	Trucks at Quarter Span: Quarter Span Defl., in.
Std. Spec.s	0.795	0.78	0.52	0.59	0.46
LRFD	0.681	0.67	0.45	0.50	0.40
NL/Nb	3/5	0.59	0.40	0.44	0.35
Tests (Pos. 1+5), max.	- -	0.50	0.35	0.37	0.30
Tests (Pos. 1+5), ave.	- -	0.42	- -	0.31	- -
NL/Nb	2/5	0.39	0.26	0.30	0.23
Tests (Pos. 4), max.	- -	0.34	0.22	0.25	0.20
Tests (Pos. 4), ave.	- -	0.29	- -	0.21	- -

- - Does not apply

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Strain gages were installed in Alabama's HPC Bridge during construction. After the bridge construction was completed, a series of static and dynamic live load tests were performed. Loading was provided by ALDOT's Load Test Trucks. Deflections, strains and stresses measured during the live load tests were compared with values calculated using AASHTO design methods. The comparisons justify the following conclusions.

A comparison of truck load distribution factors calculated from the test results shows that distribution factors from both AASHTO *LRFD* and AASHTO *Standard Specifications* are conservative.

Stresses at the bottom of the girders near midspan typically control the design for live load of prestressed concrete girder bridges. Girder stresses predicted using the simplified structural analyses of the AASHTO *LRFD* and AASHTO *Standard Specifications* were higher than the measured values. For interior girders, girder stresses predicted using AASHTO *LRFD* match better with the test results than those calculated using the AASHTO *Standard Specifications*. For exterior girders, AASHTO *LRFD* requires the load distribution factor to be calculated by two methods. In one method referred to here as the special analysis, the bridge cross section is assumed to deflect downward and rotate as a rigid body. This special analysis produced the most conservative estimate of the distribution factor for the exterior girders. Stresses at the bottom of the exterior girders calculated using the distribution factor from the special analysis were up to 52% higher than the measured stresses. The other LRFD method and the lever rule of the AASHTO *Standard Specifications* produced approximately the same distribution factor for the exterior girders. Overall, using the AASHTO LRFD distribution factors, except for the special analysis, provided calculated stresses that were larger than the measured stresses by less than 40%.

AASHTO LRFD suggests that deflections be calculated by assuming all girders deflect the same amount. This assumption resulted in the best match between calculated and measured deflections. The calculated deflections using this assumption were larger than the maximum measured deflections by 20% or less.

RECOMMENDATIONS

For the bridge investigated here, the AASHTO LRFD method of calculating the distribution factor for exterior girders by assuming the bridge cross section deflects down and rotates as a rigid body produced very conservative results. The exterior girders of this bridge would be designed under AASHTO LRFD for more load than the interior girders, although none of the live load test results indicated that the exterior girders resisted more load than the most heavily loaded interior girder. This special analysis for exterior girders may be inappropriate for prestressed concrete girder bridges. Further research is needed to determine when the special analysis for exterior girders is appropriate.

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APPENDIX A
DATA

Table A.1. Measured Deflections at Midspan

Load Position	Test	Girder Number 1	2	3	4	5
Q1	1	0.055	0.138	0.229	0.305	0.286
	2	0.054	0.136	0.226	0.304	0.292
	3	0.056	0.138	0.227	0.306	0.301
	Ave.	0.055	0.137	0.227	0.305	0.293
Q2	1	0.084	0.160	0.244	0.296	0.251
	2	0.086	0.157	0.235	0.292	0.260
	3	0.093	0.162	0.238	0.292	0.264
	Ave.	0.088	0.160	0.239	0.299	0.258
Q3	1	0.065	0.145	0.232	0.303	0.271
	2	0.070	0.147	0.231	0.299	0.278
	3	0.073	0.148	0.230	0.296	0.277
	Ave.	0.069	0.147	0.231	0.299	0.275
Q4	1	0.211	0.232	0.252	0.229	0.149
	2	0.213	0.229	0.244	0.226	0.156
	3	0.215	0.229	0.242	0.224	0.158
	Ave.	0.213	0.230	0.246	0.226	0.154
Q5	1	0.197	0.156	0.115	0.066	0.017
	2	0.196	0.155	0.115	***	0.018
	3	0.194	0.157	0.117	0.069	0.018
	Ave.	0.196	0.156	0.116	0.068	0.018
Q6	1	0.210	0.158	0.110	0.057	0.008
	2	0.206	0.159	0.113	0.061	0.011
	3	0.210	0.161	0.114	0.060	0.009
	Ave.	0.209	0.159	0.112	0.059	0.009
Q7	1	0.016	0.062	0.113	0.161	0.160
	2	0.018	0.063	0.115	0.160	0.160
	3	0.015	0.061	0.112	0.159	0.162
	Ave.	0.016	0.062	0.113	0.160	0.161
Q8	1	0.025	0.066	0.112	0.152	0.144
	2	0.023	0.065	0.113	0.155	0.151
	3	0.025	0.068	0.116	0.156	0.151
	Ave.	0.024	0.066	0.114	0.154	0.149
Q9	1	0.038	0.076	0.117	0.148	0.133
	2	0.037	0.074	0.118	0.151	0.142
	3	0.038	0.078	0.120	0.152	0.142
	Ave.	0.038	0.076	0.118	0.150	0.139
Q10	1	0.100	0.112	0.122	0.111	0.075
	2	0.101	0.112	0.123	0.114	0.080
	3	0.102	0.114	0.123	0.111	0.077
	Ave.	0.101	0.113	0.123	0.112	0.077

*** Not Available

Table A.1. (continued)

Load Position	Test	Girder Number				
		1	2	3	4	5
M1	1	0.073	0.187	0.316	0.412	0.374
	2	0.078	0.186	0.309	0.404	0.381
	3	0.080	0.188	0.309	0.405	0.391
	Ave.	0.077	0.187	0.311	0.407	0.382
M2	1	0.105	0.215	0.338	0.401	0.327
	2	0.112	0.213	0.324	0.391	0.335
	3	0.115	0.214	0.324	0.393	0.346
	Ave.	0.111	0.214	0.329	0.395	0.336
M3	1	0.088	0.198	0.320	0.403	0.345
	2	0.092	0.197	0.316	0.401	0.360
	3	0.093	0.197	0.313	0.398	0.366
	Ave.	0.091	0.197	0.316	0.401	0.357
M4	1	0.271	0.316	0.353	0.310	0.192
	2	0.267	0.308	0.342	0.309	0.208
	3	0.281	0.314	0.340	0.304	0.205
	Ave.	0.273	0.313	0.345	0.308	0.202
M5	1	0.259	0.211	0.155	0.089	0.024
	2	0.258	0.209	0.153	***	0.024
	3	0.261	0.214	0.158	0.091	0.024
	Ave.	0.259	0.211	0.155	0.090	0.024
M6	1	0.277	0.215	0.149	0.077	0.012
	2	0.275	0.211	0.142	***	0.008
	3	0.281	0.220	0.153	0.079	0.011
	Ave.	0.278	0.215	0.148	0.078	0.010
M7	1	0.022	0.083	0.152	0.213	0.206
	2	0.023	0.084	0.155	0.215	0.213
	3	0.021	0.082	0.152	0.212	0.213
	Ave.	0.022	0.083	0.153	0.213	0.211
M8	1	0.032	0.089	0.153	0.205	0.190
	2	0.038	0.094	0.159	0.210	0.202
	3	0.033	0.090	0.155	0.206	0.199
	Ave.	0.034	0.091	0.156	0.207	0.197
M9	1	0.050	0.101	0.160	0.201	0.177
	2	0.049	0.10	0.161	0.203	0.188
	3	0.048	0.101	0.162	0.203	0.189
	Ave.	0.049	0.101	0.161	0.202	0.185
M10	1	0.129	0.152	0.172	0.148	0.095
	2	0.132	0.153	0.171	0.153	0.107
	3	0.132	0.166	0.175	0.154	0.105
	Ave.	0.131	0.157	0.173	0.152	0.102

***Not Available

Table A.2. Measured Deflections at Quarterspan

Load Position	Test	Girder Number		
		3	4	5
Q1	1	0.178	0.243	0.284
	2	0.179	0.245	0.290
	3	0.182	0.246	0.294
	Ave.	0.180	0.245	0.289
Q2	1	0.194	0.240	0.239
	2	0.191	0.237	0.248
	3	0.195	0.239	0.248
	Ave.	0.193	0.239	0.245
Q3	1	0.184	0.245	0.266
	2	0.180	0.244	0.271
	3	0.185	0.241	0.264
	Ave.	0.183	0.243	0.267
Q4	1	0.205	0.182	0.133
	2	0.199	0.181	0.138
	3	0.203	0.180	0.136
	Ave.	0.202	0.181	0.136
Q5	1	0.092	0.047	0.013
	2	0.091	0.046	0.013
	3	0.094	0.048	0.013
	Ave.	0.092	0.047	0.013
Q6	1	0.085	0.040	0.006
	2	0.089	0.043	0.008
	3	0.089	0.041	0.005
	Ave.	0.088	0.041	0.006
Q7	1	0.086	0.134	0.153
	2	0.089	0.135	0.152
	3	0.087	0.133	0.153
	Ave.	0.087	0.134	0.153
Q8	1	0.089	0.128	0.135
	2	0.091	0.131	0.140
	3	0.093	0.132	0.139
	Ave.	0.091	0.130	0.138
Q9	1	0.097	0.127	0.125
	2	0.097	0.128	0.128
	3	0.098	0.128	0.128
	Ave.	0.097	0.128	0.127
Q10	1	0.109	0.088	0.062
	2	0.111	0.090	0.065
	3	0.112	0.089	0.063
	Ave.	0.111	0.089	0.063

Table A.2. (continued)

Load Position	Test	Girder Number		
		3	4	5
M1	1	0.199	0.267	0.326
	2	0.199	0.264	0.33
	3	0.200	0.265	0.333
	Ave.	0.199	0.265	0.330
M2	1	0.212	0.257	0.280
	2	0.207	0.253	0.288
	3	0.210	0.255	0.292
	Ave.	0.210	0.255	0.287
M3	1	0.203	0.259	0.302
	2	0.198	0.259	0.310
	3	0.201	0.257	0.310
	Ave.	0.201	0.258	0.307
M4	1	0.222	0.200	0.162
	2	0.213	0.200	0.174
	3	0.217	0.196	0.167
	Ave.	0.217	0.199	0.168
M5	1	0.102	0.059	0.020
	2	0.100	0.057	0.020
	3	0.105	0.060	0.019
	Ave.	0.102	0.059	0.020
M6	1	0.098	0.051	0.010
	2	0.093	0.045	0.005
	3	0.102	0.052	0.008
	Ave.	0.098	0.049	0.008
M7	1	0.098	0.136	0.173
	2	0.100	0.139	0.177
	3	0.098	0.136	0.175
	Ave.	0.099	0.137	0.175
M8	1	0.099	0.131	0.159
	2	0.103	0.135	0.166
	3	0.100	0.132	0.163
	Ave.	0.101	0.133	0.163
M9	1	0.102	0.127	0.147
	2	0.104	0.131	0.154
	3	0.104	0.131	0.154
	Ave.	0.103	0.130	0.152
M10	1	0.108	0.094	0.077
	2	0.110	0.099	0.086
	3	0.111	0.099	0.084
	Ave.	0.110	0.097	0.082

Table A.3. Bottom Flange Strains at Midspan - Electrical Resistance Gages

Load Position	Test	Girder Number				
		1	2	3	4	5
Q1	1	13	33	64	45	63
	2	14	34	59	45	65
	3	16	34	60	46	67
	Ave.	14	33	61	45	65
Q2	1	22	39	63	47	55
	2	24	39	57	45	59
	3	26	40	56	45	60
	Ave.	24	39	59	46	58
Q3	1	16	35	62	46	60
	2	19	36	58	45	63
	3	20	36	56	44	65
	Ave.	19	36	58	45	63
Q4	1	54	46	48	52	34
	2	56	47	46	47	37
	3	57	47	44	45	39
	Ave.	56	46	46	48	36
Q5	1	48	27	17	25	4
	2	48	27	18	24	4
	3	49	27	18	25	6
	Ave.	48	27	18	25	4
Q6	1	51	30	16	25	2
	2	50	29	16	25	4
	3	51	29	16	26	4
	Ave.	51	29	16	25	3
Q7	1	4	17	28	25	35
	2	4	17	27	25	35
	3	4	17	27	25	36
	Ave.	4	17	27	25	35
Q8	1	8	19	27	24	32
	2	6	18	26	24	34
	3	7	19	27	25	35
	Ave.	7	19	27	24	33
Q9	1	10	20	26	22	30
	2	12	22	27	25	35
	3	10	20	25	22	33
	Ave.	11	21	26	23	33
Q10	1	28	25	24	20	18
	2	30	26	25	21	22
	3	29	24	23	18	21
	Ave.	29	25	24	19	20

Table A. 3. (continued)

Load Position	Test	Girder Number				
		1	2	3	4	5
M1	1	17	49	111	79	97
	2	19	50	104	78	100
	3	20	50	103	78	102
	Ave.	19	50	106	78	99
M2	1	26	57	113	89	81
	2	30	57	105	84	85
	3	31	58	104	83	88
	Ave.	29	57	107	85	84
M3	1	21	51	111	83	88
	2	24	52	106	80	93
	3	25	52	103	79	97
	Ave.	23	52	107	81	93
M4	1	71	82	83	99	44
	2	73	82	81	95	50
	3	77	83	77	92	51
	Ave.	74	82	80	95	48
M5	1	73	57	24	40	5
	2	73	55	24	40	5
	3	73	55	24	40	7
	Ave.	73	56	24	40	6
M6	1	77	58	21	41	3
	2	79	57	22	41	3
	3	80	57	21	40	4
	Ave.	79	57	21	40	3
M7	1	6	23	56	39	51
	2	5	22	54	39	53
	3	5	23	54	39	54
	Ave.	5	23	55	39	52
M8	1	9	25	55	39	46
	2	11	27	54	42	52
	3	8	26	53	41	50
	Ave.	9	26	54	41	49
M9	1	13	28	54	40	42
	2	15	30	52	43	48
	3	12	28	51	40	47
	Ave.	13	29	53	41	46
M10	1	36	41	40	50	22
	2	38	43	40	48	28
	3	36	42	39	46	27
	Ave.	37	42	40	48	26

Table A.4. Bottom Flange Strains at Midspan - Vibrating Wire Gages

Load Position	Test	Girder Number		
		3	4	5
Q1	1	58	40	80
	2	54	39	81
	3	54	39	82
	Ave.	56	39	81
Q2	1	58	43	71
	2	52	38	74
	3	52	39	75
	Ave.	54	40	73
Q3	1	57	40	75
	2	53	39	78
	3	51	38	78
	Ave.	54	39	77
Q4	1	44	45	42
	2	42	41	45
	3	41	38	46
	Ave.	42	42	45
Q5	1	16	22	4
	2	17	22	5
	3	16	21	5
	Ave.	16	22	5
Q6	1	15	23	3
	2	15	22	3
	3	15	22	2
	Ave.	15	22	3
Q7	1	25	21	42
	2	24	22	43
	3	24	22	43
	Ave.	25	22	43
Q8	1	25	21	40
	2	24	21	42
	3	24	21	42
	Ave.	24	21	41
Q9	1	24	19	38
	2	23	20	40
	3	22	20	39
	Ave.	23	19	39
Q10	1	23	16	23
	2	21	15	24
	3	21	15	23
	Ave.	22	16	23

Table A.4. (continued)

Load Position	Test	Girder Number		
		3	4	5
M1	1	102	71	126
	2	95	69	126
	3	93	68	128
	Ave.	97	69	127
M2	1	103	81	106
	2	96	73	108
	3	94	71	111
	Ave.	98	75	108
M3	1	***	***	***
	2	97	70	117
	3	94	68	119
	Ave.	96	69	118
M4	1	76	87	56
	2	74	82	62
	3	71	79	61
	Ave.	74	83	60
M5	1	22	36	6
	2	22	36	5
	3	22	35	6
	Ave.	22	36	6
M6	1	20	36	3
	2	20	36	2
	3	20	35	3
	Ave.	20	35	3
M7	1	51	34	64
	2	50	35	67
	3	49	34	66
	Ave.	50	35	66
M8	1	51	35	58
	2	48	35	62
	3	48	53	61
	Ave.	49	41	61
M9	1	49	35	54
	2	46	36	57
	3	46	35	57
	Ave.	47	35	56
M10	1	36	43	29
	2	36	40	32
	3	35	40	31
	Ave.	36	41	31

*** Data not available.

Table A.5. Top Flange Strains at Midspan - Vibrating Wire Gages

Load Position	Test	Girder Number				
		1	2	3	4	5
Q1	1	-2	-9	-11	-10	-11
	2	-2	-8	-10	-9	-10
	3	-2	-8	-9	-10	-10
	Ave.	-2	-8	-10	-10	-11
Q2	1	-3	-8	-11	-9	-12
	2	-3	-7	-10	-9	-10
	3	-4	-8	-9	-9	-10
	Ave.	-3	-8	-10	-9	-11
Q3	1	-3	-8	-11	-9	-11
	2	-3	-8	-11	-9	-10
	3	-3	-8	-9	-9	-10
	Ave.	-3	-8	-10	-9	-11
Q4	1	-9	-10	-9	-6	-13
	2	-9	-9	-8	-6	-12
	3	-9	-9	-7	-6	-11
	Ave.	-9	-9	-8	-6	-12
Q5	1	-7	-5	-3	-1	-7
	2	-6	-5	-2	-1	-6
	3	-7	-5	-3	-1	-7
	Ave.	-7	-5	-3	-1	-7
Q6	1	-7	-4	-2	1	-6
	2	-7	-5	-2	0	-6
	3	-7	-5	-2	0	-6
	Ave.	-7	-5	-2	0	-6
Q7	1	0	-4	-5	-5	-5
	2	-1	-4	-5	-5	-5
	3	-1	-4	-5	-5	-5
	Ave.	-1	-4	-5	-5	-5
Q8	1	0	-3	-5	-4	-4
	2	0	-3	-4	-4	-4
	3	-1	-4	-4	-5	-4
	Ave.	0	-3	-4	-5	-4
Q9	1	-2	-4	-5	-5	-5
	2	-1	-4	-4	-5	-4
	3	-2	-4	-4	-5	-5
	Ave.	-2	-4	-5	-5	-4
Q10	1	-4	-3	-3	-3	-5
	2	-4	-4	-2	-3	-4
	3	-5	-4	-3	-3	-4
	Ave.	-4	-3	-3	-3	-4

Table A. 5. (continued)

Load Position	Test	Girder Number				
		1	2	3	4	5
M1	1	-3	-13	-16	-14	-18
	2	0	-9	-12	-10	-13
	3	-3	-11	-13	-13	-15
	Ave.	-2	-11	-14	-12	-15
M2	1	-4	-13	-14	-14	-19
	2	-4	-12	-13	-13	-16
	3	-5	-12	-12	-13	-15
	Ave.	-4	-12	-13	-13	-17
M3	1	***	***	***	***	***
	2	-4	-12	-15	-13	-16
	3	-4	-12	-13	-13	-15
	Ave.	-4	-12	-14	-13	-15
M4	1	-12	-16	-12	-9	-19
	2	-12	-14	-12	-9	-18
	3	-12	-14	-9	-9	-18
	Ave.	-12	-15	-11	-9	-19
M5	1	-11	-9	-4	0	-10
	2	-10	-9	-3	0	-9
	3	-11	-10	-4	-1	-10
	Ave.	-11	-9	-4	0	-10
M6	1	-10	-8	-3	1	-9
	2	-10	-8	-1	2	-8
	3	-12	-10	-3	0	-10
	Ave.	-11	-9	-2	1	-9
M7	1	0	-5	-8	-7	-8
	2	0	-5	-7	-8	-8
	3	0	-6	-7	-7	-8
	Ave.	0	-5	-7	-8	-8
M8	1	0	-5	-8	-6	-7
	2	-1	-6	-7	-8	-7
	3	-1	-6	-6	-7	-7
	Ave.	-1	-6	-7	-7	-7
M9	1	-2	-6	-7	-7	-7
	2	-1	-6	-6	-7	-6
	3	-2	-6	-6	-7	-7
	Ave.	-2	-6	-6	-7	-7
M10	1	-5	-6	-5	-3	-9
	2	-5	-7	-4	-4	-7
	3	-6	-8	-4	-4	-8
	Ave.	-6	-7	-4	-4	-8

*** Data not available.

Table A.6. Bottom Flange Strains at Quarter Span - Vibrating Wire Gages

Load Position	Test	Girder Number		
		3	4	5
Q1	1	68	49	79
	2	69	49	78
	3	69	49	80
	Ave.	69	49	79
Q2	1	71	58	62
	2	70	57	63
	3	70	58	62
	Ave.	70	58	62
Q3	1	72	53	69
	2	72	52	71
	3	72	53	70
	Ave.	72	53	70
Q4	1	51	63	29
	2	51	63	29
	3	50	62	30
	Ave.	51	63	29
Q5	1	8	23	2
	2	8	23	3
	3	8	24	3
	Ave.	8	23	3
Q6	1	7	20	2
	2	7	20	2
	3	7	21	2
	Ave.	7	20	2
Q7	1	42	21	39
	2	42	21	39
	3	42	21	40
	Ave.	42	21	40
Q8	1	43	25	33
	2	43	25	34
	3	43	25	34
	Ave.	43	25	34
Q9	1	41	28	29
	2	41	28	29
	3	41	28	29
	Ave.	41	28	29
Q10	1	25	41	13
	2	24	41	13
	3	24	40	12
	Ave.	24	40	13

Table A.6. (continued)

Load Position	Test	Girder Number		
		3	4	5
M1	1	44	32	60
	2	43	31	60
	3	43	30	60
	Ave.	43	31	60
M2	1	42	34	53
	2	41	33	53
	3	41	33	54
	Ave.	41	33	53
M3	1	***	***	***
	2	42	32	56
	3	42	31	57
	Ave.	42	32	57
M4	1	33	34	30
	2	33	33	32
	3	32	33	32
	Ave.	33	34	32
M5	1	10	16	4
	2	9	16	4
	3	10	16	4
	Ave.	10	16	4
M6	1	9	16	3
	2	8	16	3
	3	9	16	3
	Ave.	8	16	3
M7	1	22	16	30
	2	22	16	31
	3	22	15	31
	Ave.	22	16	31
M8	1	21	17	29
	2	21	16	29
	3	22	16	29
	Ave.	21	16	29
M9	1	20	17	27
	2	21	17	28
	3	21	16	28
	Ave.	21	17	28
M10	1	17	17	16
	2	17	17	16
	3	17	16	16
	Ave.	17	17	16

*** Data not available.

Table A.7. Top Flange Strains at Quarter Span - Vibrating Wire Gages

Load Position	Test	Girder Number		
		3	4	5
Q1	1	-10	-11	-8
	2	-9	-11	-9
	3	-10	-12	-8
	Ave.	-9	-12	-9
Q2	1	-7	-11	-8
	2	-7	-11	-8
	3	-7	-12	-8
	Ave.	-7	-11	-8
Q3	1	-7	-10	-8
	2	-8	-11	-10
	3	-8	-12	-8
	Ave.	-8	-11	-9
Q4	1	-6	-12	-5
	2	-6	-12	-6
	3	-7	-13	-5
	Ave.	-6	-12	-5
Q5	1	-4	-6	0
	2	-3	-6	1
	3	-3	-7	0
	Ave.	-3	-6	0
Q6	1	-2	-5	1
	2	-3	-6	1
	3	-2	-6	1
	Ave.	-2	-5	1
Q7	1	-3	-6	-5
	2	-4	-7	-5
	3	-4	-6	-5
	Ave.	-4	-6	-5
Q8	1	-2	-5	-4
	2	-2	-6	-4
	3	-4	-6	-5
	Ave.	-3	-5	-5
Q9	1	-3	-6	-4
	2	-3	-6	-5
	3	-4	-6	-5
	Ave.	-3	-6	-5
Q10	1	-3	-5	-1
	2	-3	-5	-1
	3	-4	-6	-2
	Ave.	-4	-5	-1

Table A.7. (continued)

Load Position	Test	Girder Number		
		3	4	5
M1	1	-8	-8	-7
	2	***	***	***
	3	-7	-8	-7
	Ave.	-8	-8	-7
M2	1	-6	-7	-6
	2	-6	-7	-6
	3	-7	-8	-6
	Ave.	-6	-7	-6
M3	1	-6	-6	-6
	2	-7	-8	-8
	3	-7	-9	-6
	Ave.	-7	-8	-7
M4	1	-5	-8	-3
	2	-5	-7	-4
	3	-6	-8	-4
	Ave.	-5	-8	-4
M5	1	-2	-4	0
	2	-2	-4	1
	3	-2	-5	0
	Ave.	-2	-4	0
M6	1	-1	-3	1
	2	0	-2	3
	3	-2	-4	1
	Ave.	-1	-3	2
M7	1	-3	-3	-4
	2	-3	-4	-3
	3	-4	-4	-4
	Ave.	-4	-4	-3
M8	1	-2	-2	-3
	2	-4	-4	-3
	3	-3	-3	-3
	Ave.	-3	-3	-3
M9	1	-3	-3	-2
	2	-3	-3	-2
	3	-4	-4	-3
	Ave.	-3	-3	-3
M10	1	-1	-2	0
	2	-1	-3	-1
	3	-3	-3	-2
	Ave.	-2	-3	-1

*** Data not available.

Table A.8. Strains in Deck

Gage	Load Position Q1				Load Position Q2			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-32	-31	-32	-32	-37	-37	-36	-37
1-TT	9	11	6	9	17	14	11	14
2-TT	4	7	-5	2	13	-5	-17	-3
2-BL	-37	-34	-32	-34	-32	-28	-27	-29
2-BT	14	13	11	13	28	23	21	24
3-TT	1	2	-1	1	2	1	-1	1
4-TT	-3	-2	-3	-3	-4	-7	-6	-6
4-TL	-58	-45	-44	-49	-53	-45	-41	-46
4-BT	14	14	15	15	14	13	14	14
5-TL	-50	-45	-46	-47	-42	-43	-40	-42
5-TT	22	23	24	23	19	16	17	17
6-TL	-19	-22	-21	-21	-23	-20	-19	-20
6-TT	-3	0	0	-1	-13	-5	-4	-7
7-TL	-25	-25	-24	-25	-22	-24	-20	-22
7-TT	2	0	-2	0	2	-3	-5	-2
7-BL	-30	-27	-26	-28	-34	-28	-27	-30
7-BT	1	-2	-2	-1	0	-5	-5	-4
8-TL	-30	-26	-25	-27	-32	-26	-23	-27
8-TT	-9	-7	-4	-6	-12	-11	-6	-10
9-TL	-36	-35	-35	-35	-34	-35	-33	-34
9-TT	3	1	1	2	0	-2	-1	-1
9-BL	-23	-20	-22	-22	-22	-19	-23	-21
9-BT	3	5	2	3	3	3	2	3
10-TL	-46	-46	-46	-46	-43	-44	-44	-44
10-TT	23	24	24	24	22	22	23	22

Table A.8. (continued)

Gage	Load Position Q3				Load Position Q4			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-34	-34	-33	-34	-38	-39	-36	-38
1-TT	9	8	7	8	15	12	12	13
2-TT	-7	-3	-10	-7	-12	-9	-9	-10
2-BL	-37	-34	-33	-34	-31	-33	-31	-31
2-BT	17	15	15	15	16	14	13	15
3-TT	0	0	3	1	1	2	-3	0
4-TT	-2	-2	-3	-2	2	1	4	2
4-TL	-52	-43	-41	-46	-32	-29	-26	-29
4-BT	14	12	10	12	5	2	4	4
5-TL	-47	-44	-43	-45	-21	-25	-15	-21
5-TT	21	19	18	19	8	6	11	8
6-TL	-21	-21	-19	-20	-26	-21	-18	-22
6-TT	-6	-3	-3	-4	-13	-13	-11	-13
7-TL	-22	-23	-20	-21	-18	-19	-16	-18
7-TT	2	-2	-2	-1	1	-3	-4	-2
7-BL	-31	-27	-25	-28	-30	-27	-24	-27
7-BT	-1	-4	-5	-3	0	-3	-6	-3
8-TL	-29	-25	-23	-26	-23	-23	-21	-22
8-TT	-12	-11	-7	-10	-7	-4	-2	-13
9-TL	-33	-33	-33	-33	-24	-25	-25	-25
9-TT	3	-1	0	1	2	1	0	1
9-BL	-25	-22	-21	-23	-15	-12	-18	-15
9-BT	5	3	1	3	5	8	-3	3
10-TL	-45	-45	-44	-45	-27	-30	-29	-28
10-TT	23	24	24	23	14	16	16	15

Table A.8. (continued)

Gage	Load Position Q5				Load Position Q6			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-17	-16	-16	-16	-16	-15	-14	-15
1-TT	5	4	5	5	10	4	5	6
2-TT	3	3	5	4	23	4	6	11
2-BL	-13	-12	-14	-13	-12	-13	-12	-12
2-BT	-1	-1	-2	-1	1	-2	-2	-1
3-TT	-1	-3	3	0	1	1	-1	0
4-TT	2	2	4	2	1	2	2	2
4-TL	-4	-4	-5	-5	-6	-4	-4	-5
4-BT	-1	0	-2	-1	0	-3	-1	-1
5-TL	1	0	-7	-2	3	-4	1	0
5-TT	3	1	-3	0	3	0	1	1
6-TL	-13	-13	-12	-12	-16	-13	-12	-13
6-TT	-1	-1	-1	-1	1	1	2	1
7-TL	-11	-11	-9	-11	-13	-12	-10	-11
7-TT	2	2	3	2	4	2	5	4
7-BL	-11	-11	-10	-11	-11	-10	-11	-11
7-BT	3	3	3	3	4	5	5	5
8-TL	-11	-11	-10	-10	-12	-8	-9	-10
8-TT	4	5	6	5	4	7	6	6
9-TL	-7	-7	-7	-7	-7	-6	-6	-6
9-TT	1	2	3	2	1	3	3	2
9-BL	-2	-7	-3	-4	-1	-4	-5	-3
9-BT	4	3	6	4	5	3	1	3
10-TL	-3	-3	-3	-3	-1	-1	-1	-1
10-TT	2	2	0	1	1	0	0	0

Table A.8. (continued)

Gage	Load Position Q7				Load Position Q8			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-14	-14	-14	-14	-17	-17	-17	-17
1-TT	8	6	6	6	9	7	7	8
2-TT	3	-1	-3	0	-4	-6	-12	-8
2-BL	-19	-18	-17	-18	-12	-13	-15	-13
2-BT	7	6	6	6	16	13	13	14
3-TT	0	0	0	0	0	1	5	2
4-TT	-15	-16	-14	-15	-7	-12	-11	-10
4-TL	-35	-27	-28	-30	-29	-27	-26	-27
4-BT	22	24	21	22	14	17	12	14
5-TL	-25	-21	-25	-24	-22	-21	-27	-23
5-TT	13	14	10	12	11	12	3	9
6-TL	-12	-13	-12	-12	-11	-13	-11	-12
6-TT	2	1	2	2	0	0	-2	-1
7-TL	-12	-13	-11	-12	-11	-13	-11	-12
7-TT	0	-2	-2	-1	0	-2	-3	-2
7-BL	-14	-14	-13	-14	-15	-15	-13	-14
7-BT	-1	-3	-3	-2	-3	-3	-6	-4
8-TL	-13	-31	-10	-18	-13	-31	-11	-18
8-TT	-8	-9	-7	-8	-11	-10	-11	-11
9-TL	-18	-18	-17	-17	-17	-19	-17	-18
9-TT	0	-1	-1	-1	-1	-3	-3	-2
9-BL	-12	-12	-13	-12	-11	-12	-9	-11
9-BT	0	0	0	0	3	-1	2	1
10-TL	-26	-26	-25	-26	-25	-28	-25	-26
10-TT	13	13	15	13	13	13	15	14

Table A.8. (continued)

Gage	Load Position Q9				Load Position Q10			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-17	-17	-17	-17	-24	-23	-23	-23
1-TT	7	5	6	6	9	6	7	7
2-TT	-17	-19	-18	-18	-12	-12	-8	-11
2-BL	-10	-8	-9	-9	-12	-13	-16	-14
2-BT	18	18	18	18	13	10	11	12
3-TT	0	1	1	0	-1	2	1	0
4-TT	-4	-1	-4	-3	5	5	4	5
4-TL	-25	-22	-22	-23	-16	-13	-12	-13
4-BT	9	7	8	8	-1	0	0	0
5-TL	-17	-15	-18	-17	-9	-5	-10	-8
5-TT	9	9	7	8	4	5	2	4
6-TL	-10	-9	-10	-9	-8	-7	-7	-7
6-TT	-3	-1	-3	-3	-13	-9	-12	-11
7-TL	-9	-9	-8	-9	-8	-8	-7	-7
7-TT	-2	-4	-4	-3	-2	-4	-4	-3
7-BL	-14	-12	-12	-12	-13	-11	-11	-11
7-BT	-4	-4	-6	-4	-6	-5	-9	-7
8-TL	-11	-11	-9	-10	-12	-12	-11	-12
8-TT	-9	-8	-7	-8	-1	-2	0	-1
9-TL	-16	-16	-16	-16	-15	-14	-14	-15
9-TT	-2	-4	-2	-3	-1	-3	0	-1
9-BL	-11	-11	-9	-10	-10	-7	-8	-8
9-BT	0	2	3	2	1	4	2	2
10-TL	-23	-24	-24	-24	-16	-16	-16	-16
10-TT	13	14	14	14	10	10	10	10

Table A.8. (continued)

Gage	Load Position M1				Load Position M2			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-21	-21	-21	-21	-21	-22	-22	-22
1-TT	11	12	4	9	12	9	3	8
2-TT	27	19	4	17	39	15	0	18
2-BL	27	-26	-27	-27	-26	-26	-26	-26
2-BT	11	7	5	8	17	10	8	11
3-TT	2	3	4	3	3	2	0	2
4-TT	2	1	3	2	0	-1	1	0
4-TL	-39	-32	-30	-34	-36	-30	-28	-31
4-BT	5	6	5	5	7	8	7	7
5-TL	-37	-35	-38	-36	-33	-31	-34	-33
5-TT	14	12	10	12	12	11	10	11
6-TL	-44	-40	-38	-40	-47	-40	-38	-42
6-TT	-5	2	3	0	-10	-2	-1	-4
7-TL	-43	-43	-40	-42	-39	-35	-33	-35
7-TT	1	-2	-5	-2	1	-4	-9	-4
7-BL	-50	-44	-42	-45	-52	-44	-41	-46
7-BT	1	-5	-4	-3	-10	-15	-15	-13
8-TL	-58	-50	-48	-52	-62	-54	-49	-55
8-TT	1	2	6	3	-8	-8	-3	-6
9-TL	-59	-57	-56	-57	-53	-53	-51	-53
9-TT	2	0	0	1	0	-3	-2	-2
9-BL	-37	-33	-31	-34	-35	-33	-32	-33
9-BT	4	7	6	5	3	5	4	4
10-TL	-73	-73	-73	-73	-63	-64	-65	-64
10-TT	39	40	41	40	33	34	35	34

Table A.8. (continued)

Gage	Load Position M3				Load Position M4			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-20	-21	-19	-20	-21	-23	-20	-21
1-TT	15	9	4	9	4	5	1	3
2-TT	27	10	0	12	3	4	-3	1
2-BL	-25	-26	-24	-25	-23	-26	-23	-24
2-BT	13	8	6	9	12	8	7	9
3-TT	2	-1	-1	0	2	2	-2	1
4-TT	2	-1	0	1	2	0	2	1
4-TL	-35	-31	-28	-31	-26	-25	-20	-24
4-BT	9	6	6	7	5	4	5	5
5-TL	-35	-33	-29	-32	-20	-24	-15	-19
5-TT	13	15	14	14	7	4	11	7
6-TL	-42	-39	-37	-39	-53	-48	-44	-49
6-TT	-4	-3	-2	-3	0	0	-1	0
7-TL	-39	-39	-35	-38	-35	-35	-31	-34
7-TT	1	-4	-5	-3	0	-5	-6	-4
7-BL	-50	-46	-42	-46	-48	-45	-41	-45
7-BT	-3	-6	-7	-6	-3	-8	-7	-6
8-TL	-58	-53	-48	-53	-44	-43	-2	-30
8-TT	-3	-5	-2	-3	-6	-4	-2	-4
9-TL	-53	-54	-53	-53	-37	-39	-38	-38
9-TT	5	1	0	2	4	2	1	2
9-BL	-36	-35	-38	-36	-23	-27	-25	-25
9-BT	8	4	-1	3	6	5	-1	4
10-TL	-66	-68	-68	-68	-36	-40	-38	-38
10-TT	34	37	37	36	18	22	21	20

Table A.8. (continued)

Gage	Load Position M5				Load Position M6			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-11	-10	-11	-11	-12	-10	-10	-11
1-TT	4	0	1	2	7	1	2	4
2-TT	7	-3	1	2	23	-1	2	8
2-BL	-10	-9	-11	-10	-10	-9	-11	-10
2-BT	3	2	-1	2	4	3	0	3
3-TT	0	-5	0	-2	1	-5	-1	-2
4-TT	2	2	2	2	2	3	3	3
4-TL	-6	-5	-5	-6	-7	-6	-5	-6
4-BT	0	2	0	1	-1	2	1	1
5-TL	-1	2	-4	-1	-3	0	-2	-2
5-TT	2	5	-1	2	1	4	0	2
6-TL	-24	-23	-21	-23	-25	-23	-21	-23
6-TT	3	2	3	3	4	3	5	4
7-TL	-19	-18	-16	-18	-20	-18	-16	-18
7-TT	4	4	4	4	6	7	6	6
7-BL	-16	-15	-16	-16	-16	32	-16	0
7-BT	4	4	5	4	6	6	-23	-4
8-TL	-16	-15	-14	-15	-16	-15	-12	-14
8-TT	5	6	8	6	6	6	8	7
9-TL	-11	-10	-10	-10	-10	-8	-8	-9
9-TT	2	3	3	3	1	2	4	3
9-BL	-4	-10	-7	-7	-2	-7	-8	-5
9-BT	5	0	4	3	5	0	-2	1
10-TL	-3	-3	-3	-3	-1	0	0	-1
10-TT	2	3	1	2	1	2	0	1

Table A.B. (continued)

Gage	Load Position M7				Load Position M8			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-11	-11	-11	-11	-11	-10	-11	-11
1-TT	7	5	3	5	8	1	3	4
2-TT	12	8	2	7	12	-1	1	4
2-BL	-14	-14	-13	-14	-13	-13	-14	-13
2-BT	5	4	3	4	7	3	4	5
3-TT	1	0	3	1	0	0	3	1
4-TT	1	-1	-1	-1	-1	0	-3	-1
4-TL	-19	-17	-16	-17	-19	-15	-16	-17
4-BT	4	6	2	4	5	3	3	4
5-TL	-20	-17	-23	-20	-19	-15	-21	-18
5-TT	6	7	3	6	-7	6	0	4
6-TL	-20	-22	-20	-21	-20	-19	-20	-20
6-TT	7	6	6	6	5	6	4	5
7-TL	-21	-22	-19	-21	-20	-18	-18	-19
7-TT	-1	-3	-4	-3	-3	-6	-5	-5
7-BL	-24	-23	-22	-23	-24	-20	-22	-22
7-BT	-2	-4	-5	-4	-7	-8	-12	-9
8-TL	-27	-45	-23	-32	-28	-24	-23	-25
8-TT	-7	-7	-6	-7	-8	-4	-7	-6
9-TL	-26	-28	-26	-26	-27	-26	-27	-27
9-TT	-8	-10	-9	-9	-3	-6	-7	-5
9-BL	-16	-20	-13	-16	-16	-16	-16	-16
9-BT	-2	-4	2	-1	1	1	0	1
10-TL	-39	-41	-38	-39	-36	-36	-37	-36
10-TT	21	21	23	22	20	22	23	22

Table A.8. (continued)

Gage	Load Position M9				Load Position M10			
	Test				Test			
	1	2	3	Ave.	1	2	3	Ave.
1-TL	-10	-10	-11	-10	-10	-11	-12	-11
1-TT	4	1	2	2	-1	-3	0	-1
2-TT	1	-3	-1	-1	-2	-5	-3	-3
2-BL	-13	-13	-13	-13	-12	-12	-14	-13
2-BT	6	4	5	5	7	4	5	5
3-TT	0	2	0	1	-2	2	4	1
4-TT	1	0	1	0	2	1	3	2
4-TL	-18	-15	-15	-16	-14	-12	-12	-13
4-BT	5	3	5	4	3	2	3	3
5-TL	-15	-11	-16	-14	-10	-7	-15	-11
5-TT	7	8	7	7	4	4	-1	2
6-TL	-19	-18	-19	-19	-24	-19	-21	-21
6-TT	2	3	2	3	-10	-8	-11	-10
7-TL	-17	-16	-12	-15	-15	-16	-14	-15
7-TT	-6	-7	-9	-8	-4	-8	-9	-7
7-BL	-21	-20	-18	-19	-23	-19	-18	-20
7-BT	-13	-13	-16	-14	-9	-11	-14	-11
8-TL	-26	-24	-23	-24	-21	-22	-21	-21
8-TT	-7	-7	-4	-6	4	4	6	4
9-TL	-27	-27	-27	-27	-21	-21	-21	-21
9-TT	-2	-5	-3	-4	0	-2	0	-1
9-BL	-18	-15	-16	-16	-14	-11	-9	-11
9-BT	0	5	1	2	1	6	4	4
10-TL	-33	-35	-35	-34	-19	-21	-21	-20
10-TT	19	21	21	20	12	12	13	12

Table A.9. Peak Response from Dynamic Tests

Sensor	Southbound Trucks Test 1	Southbound Trucks Test 2	Southbound Trucks Ave.	Northbound Trucks Test 1	Northbound Trucks Test 2	Northbound Trucks Ave.
(a) Strains from Electrical Resistance Gages (microstrain)						
Girder 1	87	86	87	92	90	91
Girder 2	92	90	91	93	94	93
Girder 3	92	89	90	83	85	84
Girder 4	102	98	100	97	100	98
Girder 5	68	65	67	55	56	56
1-TL	-38	-37	-38	-36	-38	-37
2-BL	-36	-36	-36	-33	-34	-34
4-TL	-29	-29	-29	-26	-27	-26
5-TL	-26	-25	-25	-22	-22	-22
6-TL	-49	-46	-47	-48	-49	-48
7-TL	-36	-34	-35	-32	-32	-32
7-BL	-46	-45	-45	-43	-45	-44
8-TL	-42	-40	-41	-39	-40	-39
9-TL	-43	-42	-42	-38	-38	-38
9-BL	-29	-27	-28	-25	-25	-25
10-TL	-46	-45	-45	-38	-39	-39
1-TT	13	14	13	13	13	13
2-TT	-16	-15	-15	-13	-12	-12
2-BT	18	19	19	15	16	16
3-TT	0	0	0	0	0	
4-TT	-2	2	0	2	2	2
4-BT	8	8	8	7	5	6
5-TT	10	10	10	9	9	9
6-TT	-15	-13	-14	-11	-13	-12
7-TT	-7	-7	-7	-6	-6	-6
7-BT	-14	-15	-14	-11	-12	-11
8-TT	-7	-7	-7	-5	-5	-5
9-TT	-2	-2	-2	-2	-2	-2
9-BT	3	4	3	2	3	3
10-TT	28	27	28	23	24	24
(b) Deflections at Midspan (in.)						
Girder 1	0.323	0.323	0.323	0.335	0.334	0.335
Girder 2	0.361	0.358	0.360	0.354	0.356	0.355
Girder 3	0.397	0.390	0.393	0.366	0.373	0.369
Girder 4	0.376	0.366	0.371	0.327	0.335	0.331
Girder 5	0.275	0.267	0.271	0.225	0.232	0.228
(c) Deflections at Quarter Span (in.)						
Girder 3	0.262	0.255	0.258	0.239	0.241	0.240
Girder 4	0.248	0.240	0.244	0.213	0.217	0.215
Girder 5	0.230	0.223	0.226	0.184	0.191	0.188