CHAPTER 6
LOAD DISTRIBUTION RESULTS

6.1 Introduction

The purpose of this chapter is to evaluate the load distribution behavior amongst the girders in the Little Buffalo Creek Bridge. The effects of truck position (transverse and longitudinal) and truck speed on load distribution are discussed in the initial sections. In addition, distribution factors determined from the field data are compared to the distribution factors from the design calculations provided by VDOT and the distribution factors determined using AASHTO SSHB (1996) and AASHTO LRFD (1994).

6.2 Effects of Truck Position and Speed on Load Distribution

6.2.1 Effects of Longitudinal Truck Location on Load Distribution

Bakht and Moses (1988) have shown that the percentage of load distributed to a particular girder amongst a set of girders can vary as the live load crosses the bridge span. The effects of the longitudinal truck location on the load distribution amongst the girders were evaluated using the “time response” and “max response” load fractions approach discussed in Section 4.4.2 for the dynamic load cases (Load Cases 6, 7, 8, and 9). The field data from the stationary static, load cases (Load Cases 1 and 2) was used as well.

Figure 6.1 shows a typical continuous load fraction plot determined using the “time response” approach. This particular load fraction is for Girder 1 in Load Case 7, and is based on WIM bottom flange strain ratios. The continuous load fractions based on mid span deflections are similar. Figure 6.1 shows the load fraction varied considerably except for the time when the truck was completely on the bridge (all three axles on the bridge). The actual time that the truck was completely on the bridge varied depending on the truck speed. The extreme variations in the load fractions present before and after the truck was on the bridge were because the load fractions were determined using electronic signal noise rather than pertinent data. The center portion of each continuous load fraction was used to determine an average “time response” load fraction. The center portion corresponds to the time range when all three axles were completely on the bridge and includes the point of peak response. Table 6.1 shows the results of the average "time strains" and average "time deflections" load fractions for Load Cases 6 through 9. Table 6.2 shows the results of the "max strains" and "max deflections" load fractions for Load Cases 6 through 9, and Table 6.3 shows the load fractions calculated from the field data for Load Cases 1 and 2 utilizing mid-span bottom flange strains and deflections. Although Load Cases 1 and 2 had transverse truck locations that were not in the center of the painted lanes, the results are applicable for discussion. The highlighted values in each table correspond to the girder that had the highest measured response. It is noted that within the "time strains" load fractions the girder with the highest strain did not necessarily have the highest average value.

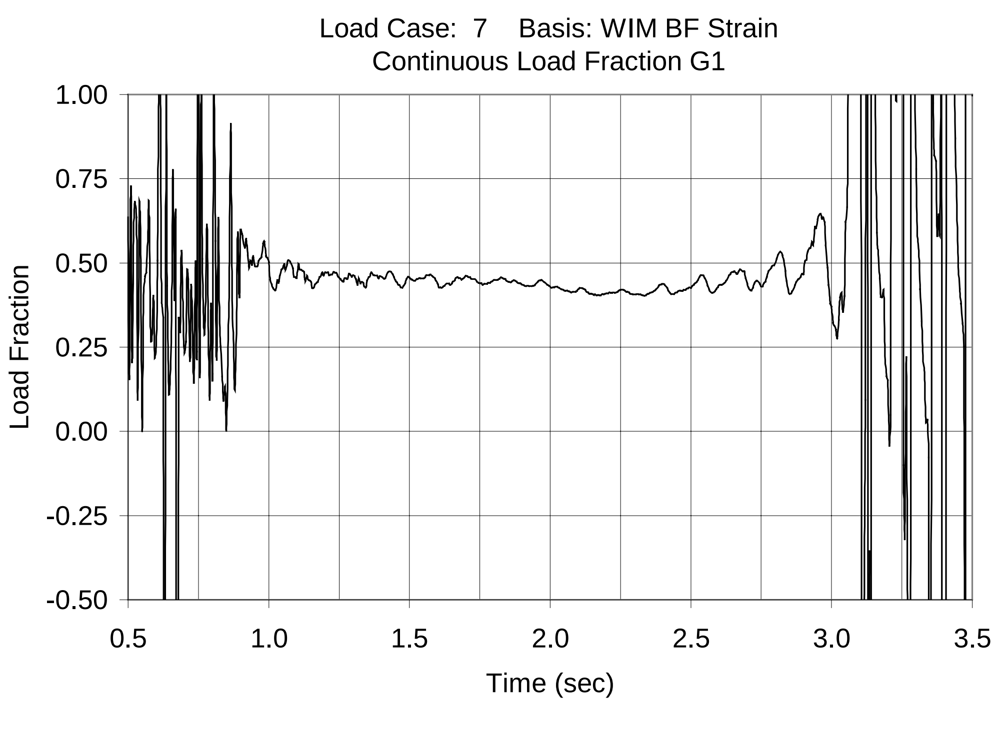

Figure 6.1 Typical Continuous Load Fraction

Table 6.1 Average Load Fractions from "Time Responses" Load Fractions

Basis	Load Case	Loaded Lane		Girder 1	Girder 2	Girder 3	Girder 4
Time Strain	6	right	Average	0.433	0.401	0.130	0.036
			Std. Dev.	0.015	0.023	0.011	0.014
	7	right	Average	0.435	0.398	0.131	0.037
			Std. Dev.	0.023	0.027	0.008	0.014
	8	middle	Average	0.184	0.342	0.269	0.205
			Std. Dev.	0.024	0.022	0.020	0.020
	9	middle	Average	0.200	0.348	0.260	0.192
			Std. Dev.	0.027	0.023	0.020	0.021
Time Defl.	6	right	Average	0.391	0.414	0.173	0.023
			Std. Dev.	0.012	0.007	0.008	0.010
	7	right	Average	0.389	0.417	0.173	0.022
			Std. Dev.	0.010	0.006	0.004	0.010
	8	middle	Average	0.143	0.340	0.326	0.191
			Std. Dev.	0.012	0.010	0.011	0.011
	9	middle	Average	0.158	0.351	0.351	0.176
			Std. Dev.	0.015	0.012	0.014	0.014

Table 6.2 "Max Responses" Load Fractions

Basis	Load Case	Loaded Lane	Girder 1	Girder 2	Girder 3	Girder 4
Max Strain	6	right	0.419	0.406	0.122	0.053
	7	right	0.407	0.430	0.122	0.041
	8	middle	0.197	0.329	0.259	0.214
	9	middle	0.196	0.345	0.256	0.202
Max Defl.	6	right	0.391	0.408	0.168	0.034
	7	right	0.387	0.414	0.171	0.029
	8	middle	0.155	0.330	0.316	0.199
	9	middle	0.161	0.339	0.310	0.190

Table 6.3 Load Fractions from Load Cases 1 and 2

Basis	Load Case	1st Rear Axle Loc.	Girder 1	Girder 2	Girder 3	Girder 4
Mid-span Strain	1	1/4 point	0.224	0.291	0.240	0.245
		1/2 point	0.204	0.327	0.246	0.222
		3/4 point	0.238	0.313	0.228	0.222
		Average	0.222	0.310	0.238	0.229
		Std. Dev.	0.017	0.018	0.010	0.013
	2	1/4 point	0.302	0.251	0.183	0.264
		1/2 point	0.307	0.261	0.180	0.252
		3/4 point	0.310	0.265	0.177	0.247
		Average	0.307	0.259	0.180	0.254
		Std. Dev.	0.004	0.007	0.003	0.009
Mid-span Defl.	1	1/4 point	0.182	0.304	0.283	0.231
		1/2 point	0.175	0.309	0.291	0.225
		3/4 point	0.186	0.318	0.288	0.208
		Average	0.181	0.311	0.287	0.221
		Std. Dev.	0.006	0.007	0.004	0.012
	2	1/4 point	0.261	0.264	0.225	0.251
		1/2 point	0.268	0.261	0.221	0.250
		3/4 point	0.266	0.271	0.224	0.240
		Average	0.265	0.265	0.223	0.247
		Std. Dev.	0.004	0.005	0.002	0.006

The continuous load fractions, whether based on strain or deflection, showed load fractions that were essentially constant for the time when the truck was completely on the bridge. The standard deviations for the average “time strains” load fractions were all less than 0.03, and the standard deviations for the average “time deflections” load fractions were all less than 0.02. The slightly lower standard deviations from the deflection based values were partially due to the use of “moving average” deflection data. The low standard deviations showed that load distribution amongst girders varied minutely as the truck traversed the span. Comparisons between load fractions based on WIM bottom flange strains indicated that the difference between average load fractions and load fractions based on maximum girder strains were generally less than 0.04. The same type of comparison between load fractions based on mid span deflections indicates differences always less than 0.02. The absolute differences between load fractions determined from all four methods for Load Cases 6, 7, 8, and 9 were generally less than 0.06, with only one difference greater than 0.06 (maximum difference = 0.07). No one method consistently resulted in lower load fractions.

The load fraction results from Load Cases 1 and 2 verified that the load distribution amongst girders was relatively constant as the trucks crossed the span. The load distribution amongst the girders was nearly the same as the trucks' first rear axles were placed at the 1/4, 1/2, and the 3/4 point of the bridge span. The differences between the load fractions for a particular girder within each calculation method (i.e., mid span strain or deflection) were less than 0.04. The standard deviations for each girder were all below 0.02. Statistically the standard deviation from only three values does not have a large significance, but the values were calculated to compare with the standard deviations from the average "time responses" load fractions. Comparisons between the two methods for each load case showed load fractions differed by values less than 0.05. Again, no one method resulted in lower load fractions.

6.2.2 Effects of Transverse Truck Location on Load Distribution

Based on the results shown in Tables 6.1 through 6.3, the transverse truck location affected the load distribution amongst the girders. Load Cases 6 and 7 were load cases with Truck 2 crossing in the right lane of the two available eastbound lanes. Figure 3.13c shows the transverse truck location on the Little Buffalo Creek Bridge for Load Cases 6 and 7.

The load fractions calculated using any of the four methods for Load Cases 6 and 7 showed that Girders 1 and 2 had the highest load fractions. However, a direct comparison between Girders 1 and 2 for the average "time strains" load fractions indicated that Girder 1 carried a slightly higher fraction of the load on average. The same comparison using the average "time deflections" load fractions indicated that Girder 2 carried a slightly higher fraction of the load than Girder 1. The field data and load fractions showed that the majority of the load was carried by the girders closest to the applied load.

Load Cases 8 and 9 were load cases with Truck 2 straddling the painted centerline. As noted in Section 3.5, the painted centerline does not coincide with the geometric centerline of the bridge. The actual transverse location of Truck 2 is shown in Figure 3.13d. Load Cases 8 and 9 had transverse truck locations that were nearly symmetrical and thus the load distribution amongst the girders was more uniform than for Load Cases 6 and 7. Tables 6.1 and 6.2 show that the highest load fractions were present in Girders 2 and 3, which were the two interior girders closest to the load. Interestingly, the load fractions based on the available WIM strain data indicated that Girder 2 had load fractions at least 0.07 higher than Girder 3. The max difference between Girder 2 and Girder 3 was 0.09. Comparisons between load fractions for Girder 2 and Girder 3 based on deflection data indicated differences generally lower than 0.02, with a maximum difference of 0.03.

A theoretical system (i.e., a pure simply supported span with no bearing restraint or secondary stiffening effects) with the truck in the geometric center of the bridge would result in Girders 2 and 3 having the same load fraction. Because the truck is approximately shifted one foot to the right toward Girder 2, the load fractions were expected to be slightly different. The differences from the strain based load fractions of Girders 2 and 3 indicated percent differences generally over 20 percent, and as high as 30 percent. The percent differences between the deflection based load fractions for the same girders were generally less than five percent, with a maximum percent difference of ten percent. The increased percent differences for the strain based load fractions seemed to indicate that a minor shift in load location caused a larger shift in load distribution. The large percent differences in the strain based load fractions for Girders 2 and 3 of Load Cases 8 and 9 cannot be explained with the limited field data.

Load Cases 1 and 2 were stationary static, load cases involving side-by-side loading with Trucks 1 and 2 (Reference Section 3.3 for truck descriptions and Section 3.5

for load case descriptions). Table 6.3 shows that the load distribution amongst the girders differed for Load Cases 1 and 2, indicating that the transverse truck location does affect the load distribution behavior amongst the girders.

6.2.3 Effects of Truck Speed on Load Distribution

The effects of truck speed on load distribution were evaluated by comparing load fractions from load cases that had the same truck location, but different truck speed. Load Case 6 (47 mph) was compared to Load Case 7 (20 mph) and Load Case 8 (47 mph) was compared with Load Case 9 (25 mph). Figures 6.2 and 6.3 show graphs plotting the load distribution for the comparable load cases. The points on the graphs are from the "max strains" load fractions for each load case. The lines connecting the points link associated points within a load case and are intended to improve clarity. The connecting lines do not represent any type of strain continuity amongst girders. The graphs show that the load distribution amongst girders within comparable load cases do not vary with an increase in truck speed. Other graphs using average "time response" load fractions and "max deflections" load fractions show the same results.

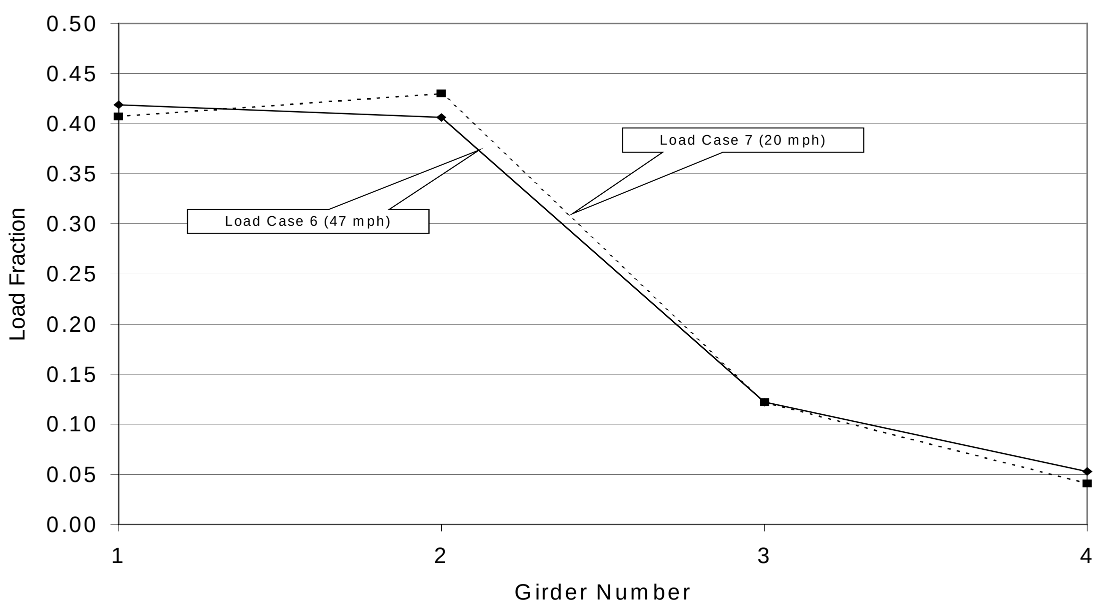

Figure 6.2 Load Distribution amongst Girders for Load Cases 6 and 7

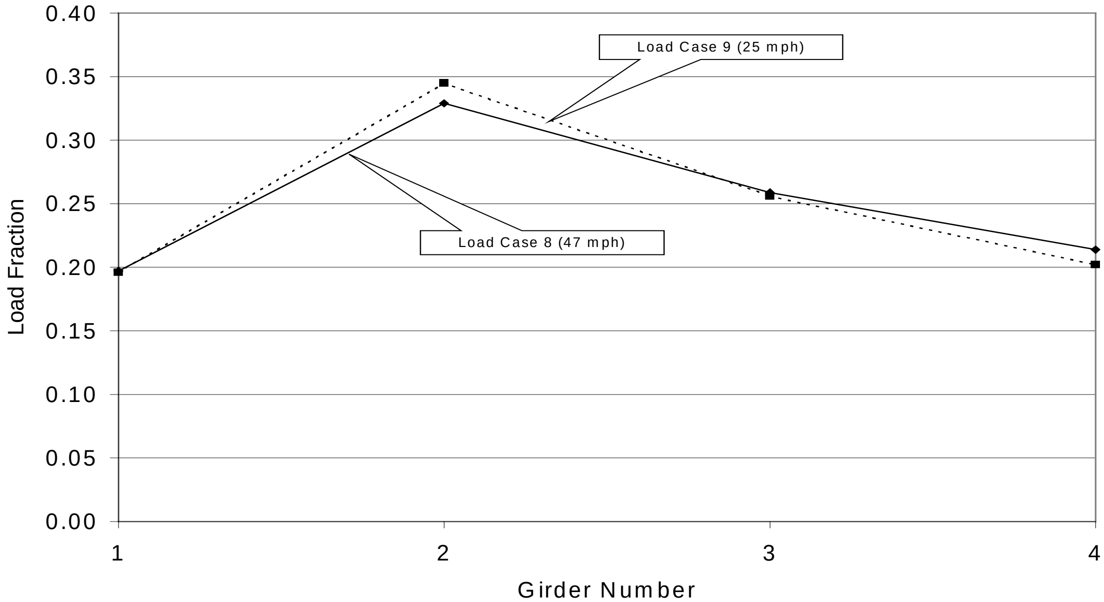

Figure 6.3 Load Distribution amongst Girders for Load Cases 8 and 9

6.3 Distribution Factor Results and Comparisons

6.3.1 Introduction

The distribution factors determined from Load Cases 1, 2, 6, 7, 8, and 9 for comparison to the distribution factors from the design calculations, AASHTO SSHB (1996), and AASHTO LRFD (1994) were determined using the "max response" approach. This approach was chosen because it was more convenient to determine a girder's maximum response from the field data collected by VTRC. The "max response" approach also allowed for superposition of one-lane loaded results to obtain two-lane loaded distribution factors. As discussed in the previous sections with load fractions, the "max response" and average "time response" approach yield similar results.

The maximum responses (mid span strain or deflection) from each girder in a particular truck crossing were used in Equation 4.1 to determine the distribution factors from the field data. As before, the mid-span deflection data used was "moving average of six" data.

6.3.2 Distribution Factors from Field Data

6.3.2.1 One-Lane Loaded Results

The one-lane loaded distribution factors were determined from Load Cases 6, 7, 8, and 9. Load Cases 8 and 9 involved Truck 1 crossing approximately in the center of the bridge and not actually in a particular lane; however, the distribution factors for Load Cases 8 and 9 were calculated as if one-lane was loaded to determine if they resulted in critical distribution factors for an interior or exterior girder. For these load cases, the load fractions determined from Equation 4.1 have values of n = 1, thus the distribution factors, DF's, are the same as the load fractions. Table 6.4 contains the distribution factors determined from these load cases using the "max response" approach. As compared to Load Cases 6 and 7, Table 6.4 shows that Load Cases 8 and 9 do not result in critical distribution factors for an interior or an exterior girder. For the two heaviest loaded girders (Girders 1 and 2), the maximum strain distribution factors were slightly higher than the maximum deflections distribution factors.

Table 6.4 One-Lane Loaded Distribution Factors (DF's) from Field Data

Basis	Load Case	Loaded Lane	Distribution Factor, DF			
			Girder 1	Girder 2	Girder 3	Girder 4
Max Strain	6	right	0.42	0.41	0.12	0.05
	7	right	0.41	0.43	0.12	0.04
	8	middle	0.20	0.33	0.26	0.21
	9	middle	0.20	0.35	0.26	0.20
Max Defl.	6	right	0.39	0.41	0.17	0.03
	7	right	0.39	0.41	0.17	0.03
	8	middle	0.16	0.33	0.32	0.20
	9	middle	0.16	0.34	0.31	0.19

6.3.2.2 Two-Lanes Loaded Results

Two-lane loaded results were obtained directly from field data for Load Cases 1 and 2, which were stationary static tests with two side-by-side trucks. Equation 4.1 was used with the field data and a value of n = 2 (two trucks). As previously mentioned, the trucks in Load Cases 1 and 2 were not located in the center of the painted lanes; however, the distribution factors determined for these load cases were included in the discussion.

In addition, two-lane loaded results were obtained from Equation 4.1 by superposition of the results from the one-lane loaded load cases, Load Cases 6 and 7. The results from Load Cases 8 and 9 were not applicable to this process. Past researchers, Stallings and Yoo (1993) and Kim and Nowak (1997) have used this approach. Stallings and Yoo (1993) verified the distribution factors from superposition of one-lane loaded results to those from actual two-lane loaded tests. Because actual tests were not performed by VTRC with both lanes loaded simultaneously and because the painted lanes are not centered on the bridge, a few assumptions were made. First, it was assumed that the superimposed truck in the left lane for side-by-side loading was symmetrical to the truck in the right lane. This assumption results in the left lane truck not being centered in the painted left lane. Second, it was assumed that the girders of the bridge would behave in a symmetrical fashion if the left lane were loaded in the same location as the right lane. This assumption is based on the fact that for a reasonably stiff bridge deck, the load distribution amongst girders is a global phenomenon not affected by localized behavior of the deck. In essence, the maximum responses of symmetrical girders from a particular load case were added to obtain the response at each girder. A value of $n = 2$ was used in Equation 4.1.

Table 6.5 shows the distribution factors determined for the two-lane loaded scenario based on maximum mid-span responses (i.e., maximum mid-span strain and deflection). Direct comparisons could only be made for distribution factors obtained from superposition of Load Cases 6 and 7 because the transverse truck locations were the same. For these two load cases, the maximum strain distribution factor was the highest for the exterior girders (DF = 0.47), while the maximum deflection distribution factor was the highest for the interior girders (DF = 0.58).

Table 6.5 Two-Lanes Loaded Distribution Factors (DF's) from Field Data

Basis	Load Case	Loaded Lane	Distribution Factor, DF			
			Girder 1	Girder 2	Girder 3	Girder 4
Max Strain	1	both*	0.41	0.65	0.49	0.44
	2	both*	0.61	0.52	0.36	0.50
	6	both**	0.47	0.53	0.53	0.47
	7	both**	0.45	0.55	0.55	0.45
Max Defl.	1	both*	0.35	0.62	0.58	0.45
	2	both*	0.54	0.52	0.44	0.50
	6	both**	0.42	0.58	0.58	0.42
	7	both**	0.42	0.58	0.58	0.42

*Slightly Anti-symmetrical Loading
**Using Superposition

6.3.3 Comparisons to the Design Calculations and the AASHTO Specifications' Distribution Factors

6.3.3.1 Introduction

The distribution factors from the design calculations and the AASHTO specifications (AASHTO SSHB 1996 and AASHTO LRFD 1994) were determined for interior and exterior girders involving one and two-lanes loaded (Reference Section 4.4 and Appendix B). In order to make comparisons with the distribution factors determined from the field data, multiple presence factors were either not included or removed from the distribution factors from the AASHTO specifications. As previously mentioned, the aluminum deck was assumed to behave similar to a concrete deck, which allowed the distribution factors to be calculated from the AASHTO specifications simplified procedures. The largest distribution factors, either strain or deflection based, determined from the field data within the common loaded cases, Load Cases 6 and 7, were used for each type of loading (i.e., one or two lanes loaded) to allow for a worst case comparison to the design calculations and specification values. Load Cases 1 and 2 were treated separate because of their differences in transverse truck location. Table 6.6 and 6.7 contain the distribution factors discussed in the following sections.

Table 6.6 Distribution Factors (DF's) for Typical Transverse Truck Locations

Number of Loaded Lanes	Girder Location	Field Data	Design Calcs.	SSHB (1996)	LRFD (1994)
1	Exterior	0.42	0.69	0.67	0.57
	Interior	0.43	0.63	0.61	0.39
2*	Exterior	0.47	0.90	0.77	1.14
	Interior	0.58	0.90	0.77	0.64

*Factors determined from superposition

Table 6.7 Distribution Factors (DF's) for Non-typical Transverse Truck Locations

Number of Loaded Lanes	Girder Location	Field Data	Design Calcs.	SSHB (1996)	LRFD (1994)
2	Exterior	0.61	0.90	0.77	1.14
	Interior	0.65	0.90	0.77	0.64

6.3.3.2 Comparison of One-Lane Loaded Distribution Factors

The upper portion of Table 6.6 contains the distribution factors for one-lane loaded comparisons. The exterior girder distribution factor determined from the field data is well below the other three values. The exterior girder distribution factor for one-lane loaded from the design calculations provided by VDOT is the highest. The "lever method" was used in the design calculations with the outer wheel-line of load only one-foot from the face of the concrete parapet (Reference Appendix B, p. 153-155) The design calculations' distribution factor is 64 percent greater than the distribution factor determined from the field data. The AASHTO SSHB (1996) value of 0.67 was determined from Equation 4.3, which was the controlling limit for the exterior girder distribution factor (Reference Appendix B, p. 156-158). The AASHTO LRFD (1994) value is the lowest and closest to the field data factor, and is 36 percent greater than the distribution factor determined from the field data. The AASHTO LRFD (1994) factor was determined from Equation 4.8, which was the limiting value set forth by the specification (Reference Appendix B, p. 159-161).

The interior girder, distribution factor from the field data is well below the design calculations' factor and the AASHTO SSHB (1996) factor. Again, the design calculations' distribution factor was determined from the "lever method," and the resulting value is 47 percent larger than the field data factor (Reference Appendix B, p. 153-155). The AASHTO SSHB (1996) distribution factor is 40 percent greater than the field data factor and was determined by dividing Equation 4.2 (with D = 7) by two to convert from a wheel-line distribution factor to a factor applicable to an entire truck. (Reference Appendix B, p. 156-158). Interestingly, the AASHTO LRFD (1994) interior girder, distribution factor of 0.39 is nine-percent less than the distribution factor determined from the field data. The AASHTO LRFD (1994) factor was determined using Equations 4.4 and 4.6 (Reference Appendix B, p. 159-161). It is noted that the multiple presence factor of 1.2 was removed to obtain a value for comparison to the field data factor. In actual design, the distribution factor would have been 0.47, which is nine-percent larger than the field data distribution factor.

6.3.3.3 Comparison of Two-Lanes Loaded Distribution Factors

The distribution factor comparisons for two-lanes loaded were divided into two groups. The maximum values determined from the superposition of common load cases, Load Cases 6 and 7, were compared and serve to represent typical side-by-side loading. These values are presented in the lower portion of Table 6.6. The maximum values from Load Cases 1 and 2, which are not so typical side-by-side loadings because of their respective transverse truck locations, are presented in Table 6.7.

The exterior girder distribution factors from the design calculations and AASHTO SSHB (1996) are the same as the interior girders because of the limitation that an exterior girder shall not have less carrying capacity than an interior girder. Although AASHTO SSHB (1996) specifies the use of the "lever method" for the exterior girder distribution factor, the limiting value was that determined for an interior girder. The factor was determined by dividing Equation 4.2 (with D=5.5) by two to convert from a wheel-line distribution factor to factor applicable to an entire truck (Reference Appendix B, p. 156-158). The AASHTO LRFD (1994) exterior girder distribution factor was determined using Equation 4.8, which was a specified limitation because of the existence of diaphragms (Reference Appendix B, p. 159-161). The AASHTO LRFD (1994) interior girder distribution factor for two-lanes loaded was determined using Equations 4.5 and 4.6.

The maximum, exterior girder distribution factor from the field data for typical side-by-side loading is much lower than the factor from the design calculations and both AASHTO specifications. The design calculations' distribution factor is over 90 percent larger than the factor determined from the field data, and the AASHTO SSHB (1996) exterior girder distribution factor is 64 percent greater than the field data factor. The largest exterior girder distribution factor is that specified by AASHTO LRFD (1994), which is over 140 percent larger than the distribution factor determined using the field data. If the limitation set forth by AASTHO LRFD (1994) in Equation 4.7 is neglected, the exterior girder distribution factor would have been the same as the interior girder distribution factor of 0.64 (multiple presence factor removed), which would still be 36 percent larger than the field data distribution factor.

As shown in Table 6.6, the maximum, interior girder distribution factor for typical side-by-side loading from the field data is lower than the design calculation's factor and both AASHTO specifications' factors. The design calculations' distribution factor for an interior girder is 55 percent larger than the field data distribution factor, while the AASHTO SSHB (1996) distribution factor is only 33 percent larger than the field data distribution factor. The closest agreement lies in the AASHTO LRFD (1994) interior girder, distribution factor, which is only ten percent larger than the maximum distribution factor determined from field data.

Load Cases 1 and 2 were grouped together as loads that were not typical side-by-side truck occurrences. Although the transverse locations are not typical, they are possible. The maximum distribution factors determined from Load Cases 1 and 2 for each type of girder were determined for comparison to the design calculation's factors and both AASHTO specifications' factors. Table 6.7 presents the distribution factors. The maximum distribution factor for an interior girder resulted from Load Case 1 and the maximum distribution factor for an exterior girder resulted from Load Case 2. Both maximum factors were from the "maximum strain" approach.

The maximum, exterior girder distribution factor from the field data for these non-typical load cases was still considerably lower than the design calculation's factor and the AASHTO specifications' factors. The closest distribution factor to the field data value is the AASHTO SSHB (1996) distribution factor of 0.77, which is 26 percent larger than the field data factor.

The maximum, interior girder, distribution factor from the field data was lower than both the design calculation's factor and the AASHTO SSHB (1996) factor. However, the AASHTO LRFD (1994) interior girder, distribution factor was approximately two-percent less than the distribution factor determined from the field data.

6.3.4 Limitations of Load Distribution Evaluation

The evaluations presented above are limited because the field tests were not repeated for each particular load case. The field data provided by VTRC often contained values that did not seem logical for a particular load case. The measured strains and deflection values for certain girders within a load case did not match what would be expected from the transverse location of the trucks. Girders that were farther from the applied load often showed data that had considerably higher strains and deflections than those girders directly beneath the applied load. Percent differences in girder strain and deflection data for near symmetrical loading were often above 30 percent, which serves to limit the superposition approach used to evaluate two-lanes loaded behavior.

The assumption of the aluminum deck behaving similar to concrete deck provided for a means to calculate distribution factors to be compared to the field data values. The aluminum deck is said to behave similar to a concrete deck in System II bending (Matteo

1997), but this may not necessarily allow for the use of the empirical girder distribution factors in the AASHTO specifications.

In addition, the extent of composite action affects the load distribution behavior of a bridge. As discussed in Chapter 5, the composite action evaluation showed a need for laboratory research on component sections to evaluate the effectiveness of the grouted connection in providing full composite action. If this type of connection does not provide for near full composite action then the load distribution behavior of the Little Buffalo Creek Bridge may be affected accordingly.

6.3.5 Summary and Need for Research

With the noted limitations, the field data from VTRC showed that the longitudinal location of the trucks on the Little Buffalo Creek Bridge did not adversely affect the load distribution amongst the girders; however, the transverse location of the trucks greatly affects the load distribution amongst the girders. Moreover, the truck speed did not affect the load distribution amongst the girders.

Distribution factors determined from the field data for one and two-lanes loaded were all extremely less than the values from the design calculations, which indicates that compared to the test results the design calculations distribution factors were conservative. The field data distribution factors for one and two-lanes loaded were also below the AASHTO SSHB (1996) distribution factors for both interior and exterior girders, indicating that the empirical equations from AASHTO SSHB (1996) are probably conservative for this particular bridge configuration. The field data distribution factors for one and two-lanes loaded were below the AASHTO LRFD (1994) distribution factors for the exterior girders. For an interior girder with one-lane loaded and multiple presence neglected, the distribution factor from AASHTO LRFD (1994) is lower than the factor determined from the field data. Typical two-lanes loaded distribution factors determined from the available field data indicate that the AASHTO LRFD (1994) distribution factors are larger; however, if the non-typical side-by-side truck locations are considered the

AASHTO LRFD (1994) distribution factors are below the values determined from the field data.

In order to obtain reliable distribution factors for one and two-lanes loaded to be used in evaluation and rating of the Little Buffalo Creek Bridge, there needs to be additional testing conducted with test trucks and possibly actual truck traffic. As done by Kim and Nowak (1997), the testing under actual truck traffic would provide a means of statistically evaluating the load distribution amongst the girders and the effects of trucks of different configurations, speed, weight, and transverse location.