Influence of Cross Frames on Load Resisting Capacity of Steel Girder Bridges

ATOROD AZIZINAMINI, STEVE KATHOL, and MICHAEL W. BEACHAM

INTRODUCTION

The fifteenth edition of the AASHTO manual requires intermediate cross frames in steel girder bridges with maximum spacing of 25 feet. Although these cross frames may be needed for temporary loads their effectiveness following construction has been a point of debate for short and medium span bridges.

Cross frames with different configurations have been utilized in bridge construction. Besides increasing construction costs, many problems in steel girder bridges could be attributed to the presence of cross frames. For instance many states have observed cracking in the girder web of bridges in the vicinity of the cross frame's connection to the beam, especially for details where stiffeners are not rigidly connected to top and bottom flanges.

Prior to casting the concrete deck, smaller spacing of cross frames results in smaller laterally unbraced lengths of steel girders and, consequently, could result in smaller sections. In some steel bridges it is therefore possible to have diaphragm spacing of less than 25 ft., the maximum spacing allowed by the AASHTO manual. The need for cross frames in these instances becomes an even more serious question, especially if their presence results in unsatisfactory performance of the bridge (such as developing cracking in girder web).

To address this issue, a combination of analytical and experimental investigations was conducted. A summary of the experimental investigation related to the use of cross frames is presented in this paper.

EXPERIMENTAL INVESTIGATION

Bridge Description

A series of tests was carried out on a full scale bridge constructed in the structural laboratory. The bridge was a 70-ft. long simple span with a total width of 26 ft. Figure 1 shows a photo of the completed bridge in the laboratory; Figure 2 shows the bridge cross section. The superstructure consisted of three welded plate girders built compositely with a 7 ½-in. thick reinforced concrete deck. The girders, each 54-in. deep, were spaced 10-ft. on center and the reinforced concrete deck had a 3-ft. overhang. The concrete barrier structure was an open concrete bridge rail, with 11×11-in. posts spaced 8-ft. on center. The construction sequence was identical to field practice.

During construction, *K* type cross frames at 11.2-ft. spacing as shown in Figure 3 were used. This resulted in a total of 7 cross frames in each lane as shown in Figure 4.

A series of strain gages and potentiometers was attached to the bridge to measure strains and bridge deflections at different locations.

Strains in Cross Frames During Curing

The behavior of the bridge was monitored continuously for a period of approximately 102 days following casting of the concrete deck before conducting live load tests to evaluate the effect of cross frames. During this period strains in the cross frames resulting from creep and shrinkage of the concrete deck were measured using vibrating wire strain gages. Cross frames located in the south lane along lines A2 and A4 (shown in Figure 4) were instrumented with three vibrating wire gages each. Figures 5 and 6 show locations of the gages for both cross frames and give the identification designation

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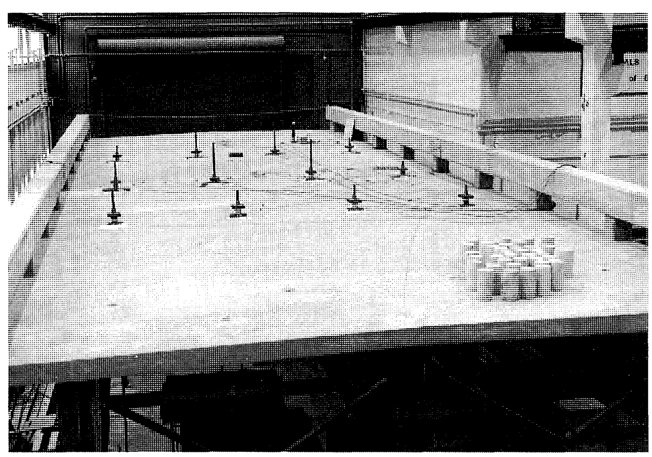

Fig. 1. Photo of completed bridge.

for each gage. Figure 7 shows the measured strains in cross frames for the first 90 days after casting the concrete deck. The resulting strains result from both dead load and creep and shrinkage of the concrete deck. The maximum strain occurred in the upper WT4×9 member in both *K* cross frames. The maximum strain resulting from dead loads only was approximately the same for both cross frames (approximately 60 micro strain (tension)). Assuming a modulus of elasticity of 29,000 ksi, the maximum dead load stress in the upper chord of each diaphragm is calculated to be 1.7 ksi (tensile). During the curing process the strain in the same members became compressive and after 90 days exhibited maximum compressive stress of approximately 0.9 ksi. This behavior indicates that during the curing process the influence of cross frames is minimal. Additionally, although small, cross frames restrain the concrete deck to shrink in the transverse direction during curing. This restraint should be expected to increase as member sizes for cross frames increase. Therefore, if transverse restraint is of any concern smaller cross frame sizes should help the situation.

Live Load Test Setup

The application of live loads was achieved by using 12 hydraulic rams, shown in Figure 1, each capable of applying 400,000 lbs. Each ram simulated one wheel load. Six rams were placed on each lane of the bridge to simulate the wheel configuration of one AASHTO HS20 truck load. Figure 8 shows locations of all 12 rams simulating two trucks side by side on the bridge. The spacing of axles as shown in Figure 8 are 12 and 15 ft. instead of 14 and 14 ft. as specified by AASHTO. This was a result of laboratory constraints. Each load point consisted of a 1⅜-inch diameter DYWIDAG rod passed through the concrete deck and anchored in the basement of the structural laboratory. The hydraulic rams simulating rear axles were operated using a single pump, while the front axles were activated using a different pump. Using this hydraulic ram configuration it was possible to simulate having a) only one truck in the north or south lane (by activating only six rams in the desired lane), b) one truck straddling about the centerline (by activating the middle six rams) or c) one truck on each lane (by activating all rams). Steel plates were used to simulate the footprint of trucks at each load point. The size of the footprint was determined from the AASHTO manual. The size of the plates used in the rear and front axles were 20×8×2 in. and 10×4×2 in., respectively.

Test Descriptions and Results

Table 1 gives a list of 12 tests conducted. Each test consisted of applying three cycles at the indicated load level. Also shown in Table 1 are the type and spacing of cross frames used in each test. For instance during Test No. 1, *K* type cross

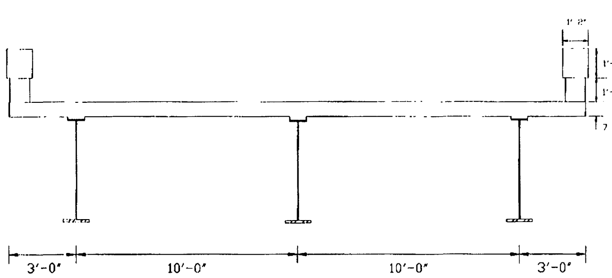

Fig. 2. Bridge cross section.

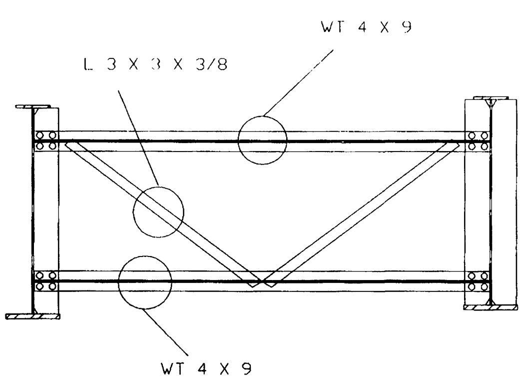

Fig. 3. Member sizes used for K type cross frames.

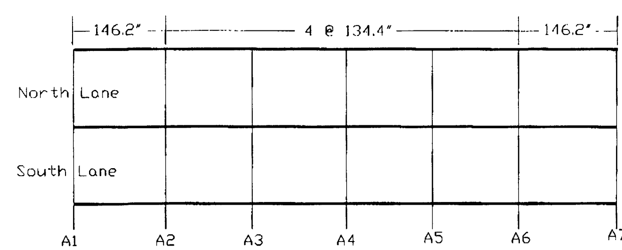

Fig. 4. Layout of cross frames during construction.

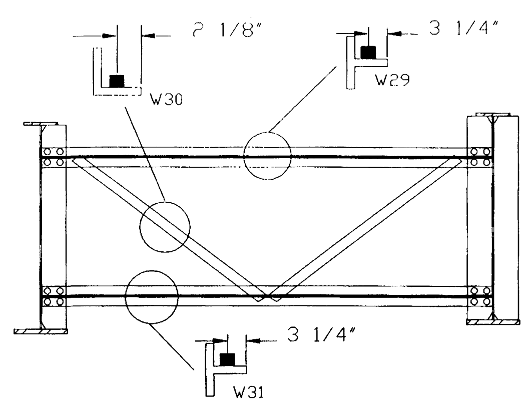

Fig. 5. Location of gages used in cross frame along A2 line in Figure 4.

frames at 11.2-ft. spacing were used. In this test all 12 rams were activated to simulate having two trucks side by side. Further, during Test No. 1, the bridge was loaded and completely unloaded three times at a load level corresponding to having trucks weighing 2.5 times the AASHTO HS20 truck load on each lane (180,000 lbs. total on each lane). The load level corresponding to 2.5 times AASHTO HS20 truck load was achieved by increasing load points on rear and front axles to 40,000 and 10,000 lbs, respectively.

During Tests 2 through 5, *K* type cross frames at 22.4-ft. spacing were utilized. All but the end cross frames were removed during Tests 6 through 9. Tests 10 through 12 utilized X type cross frames of the configuration shown in Figure 9 at 22.4-ft. spacing.

In the following sections the effect of cross frame types and spacing on response of the bridge to applied live loads will be discussed in terms of a) maximum strain in the concrete slab, b) maximum strain in steel girders, c) girder separation d) maximum strains in the cross frames, e) maximum girder deflections, and f) load distribution factors.

a) Strain in Concrete Slab

Table 1 gives maximum transverse concrete strain in the top of the south lane slab measured at mid span of the bridge in the middle of the two girder lines. As indicated from this table, the level of strain is relatively small for all tests. It should also be noted that the applied live loads in all tests correspond to having one or two trucks weighing 180,000 lbs. each (2.5 times HS20 truck load). Data indicate that for comparable spacing, *K* type cross frames resulted in relatively smaller strains than *X* type cross frames.

b) Strain in Steel Girders

The effect of altering cross frame spacing and types on the behavior of steel girders will be discussed in terms of maximum tensile strain in the bottom flange of interior and exterior girders at mid and quarter span of the bridge. In all cases the applied live load corresponds to having one or two trucks on the bridge, each weighing 180,000 lbs. A summary of strains measured at midspan and quarter span of the bridge for different cross frame configurations and loading conditions are given in Table 2. The reported strains are all measured in the bottom flange at indicated locations. The following sections discuss the results for each loading condition.

Both Lanes Loaded

Table 2 gives maximum bottom flange strains at midspan and quarter point for the cases where both lanes are loaded. Comparison of test results for *K* cross frames at 11.2-ft and 22.4-ft. spacing indicate that the maximum tensile strain in bottom flange of center girder increased slightly as the spacing of cross frames increased. The difference in exterior girder strain is not sufficiently significant to make definite conclusions. As shown in Table 2, the maximum strains in both interior and exterior girders are not significantly affected by using *X* type cross frames instead of *K* type. The case in which all cross frames are removed resulted in higher tensile strain in the center girder and smaller strains in exterior girders when compared to cases where *X* or *K* type cross frames at 22.4-ft. spacing are used. The increase in maximum tensile strain in the bottom flange of the center girder at midspan resulting from removal of all cross frames is only 6.4 percent over the case where *K* type cross frames are used at 22.4-ft. spacing. This behavior indicates that load distribution factors are only slightly affected by the presence of intermediate cross frames.

Straddling the Center Lane

Table 2 also gives the maximum bottom flange strains observed for each cross frame configuration at midspan and quarter point with applied load simulating one truck weighing 180,000 lbs. straddling the bridge centerline. Results indicate

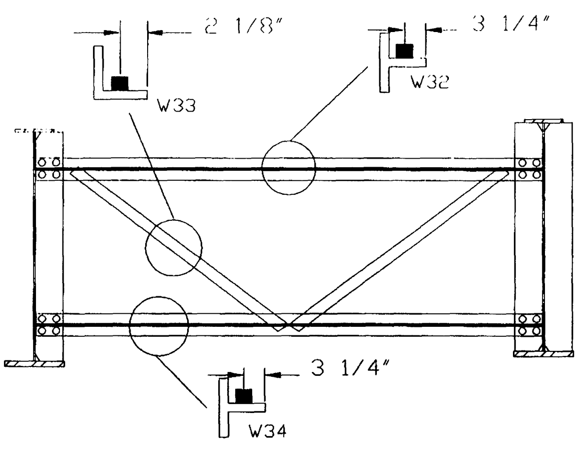

Fig. 6. Location of gages used in cross frame along A4 line in Figure 4.

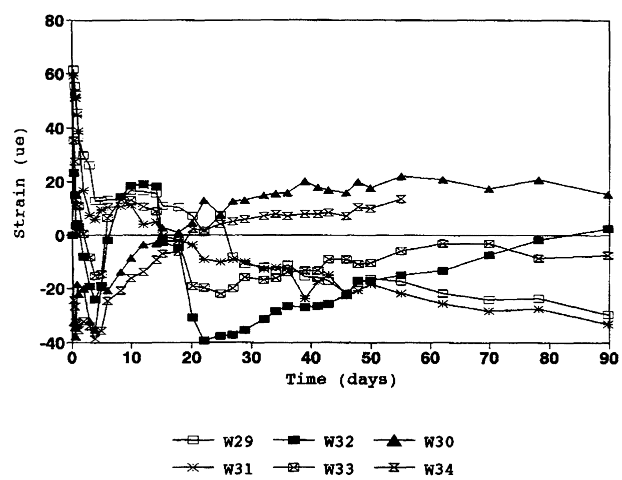

Fig. 7. Measured cross frame strains during curing process.

that X and K type cross frames result in the same behavior. From this table it could be observed that in the case of no cross frames, resulting strain in center girder is 14.5 percent higher than the case with K cross frames at 22.4-ft. spacing. In evaluating this result two points are important to consider. First, the difference is relatively small. Second, although the case of one truck straddling the centerline results in higher differences percentage wise, the magnitude of resulting strain in bottom flange is significantly smaller than the case where both lanes are loaded. This is important when considering the fact that the design situation will be governed when both lanes are loaded.

Loading One Lane Only

Table 2 shows maximum bottom flange strains observed at midspan and quarter point with only the south lane loaded with a truck weighing 180,000 lbs. As expected, the influence of removing all cross frames is more pronounced for the exterior girder farthest from the loaded lane (north lane). As noted in this table, although the percentage difference in exterior girder (north lane) strain is larger compared to the case of having K cross frames at 22.4-ft. spacing (23 micro strain compared to 35 micro strain), the magnitude of resulting strains in exterior girders is much smaller compared to the case when both lanes are loaded (23 micro strain compared to 349 micro strain).

Figures 10 and 11 show the measured strain distribution over the depth of the bridge cross section at midspan for the center and one of the exterior girders, respectively, for the cases where both lanes were loaded with trucks weighing 180,000 lbs. each. The strain distributions correspond to maximum loading point. Each figure gives strain distribution for different cross frame configurations. The strains were obtained from surface gages attached to concrete surfaces, steel web and flanges in addition to concrete embedment gages placed in concrete prior to casting the slab. Both figures exhibit an approximately linear strain distribution. From Figures 10 and 11 all four diaphragm configurations produced approximately the same strain distribution. The neutral axis for the center girder is shown to be approximately 39 inches from the bottom flange. This compares quite well with the calculated neutral axis position of 37.2 inches from the bottom flange. The neutral axis location for the exterior girder is measured to be approximately 54 inches from the bottom flange, 38.5 percent higher than the calculated value. This discrepancy could be attributed mainly to the presence of concrete rail located approximately 2.5 ft. from the exterior girder. The presence of concrete rails increase the flexural stiffness of the exterior girder.

Table 3 gives a comparison of maximum bottom flange stresses in the center girder predicted by AASHTO procedures and measured experimental values. The stresses calculated based on AASHTO procedures are presented using modular ratios, n, of 8 and 24. All calculated stresses are based on HS-20 truck loads without including impact or load factors. Stresses given under the experimental column are based on live loading of one HS-20 truck in each lane (i.e. a 72,000 lb. truck in each lane). These stresses were calculated by multiplying the strain by the average modulus of elasticity obtained from material tests (26,200 ksi).

Table 3 indicates that the AASHTO procedure overestimates the maximum bottom flange stresses by approximately 100 percent. One of the reasons for this overestimation could be the conservative estimate of distribution factor given by the fifteenth edition of the AASHTO manual. Further, the measured maximum bottom flange stress in the case of no cross frames is 70 percent of the predicted values.

c) Girder Separation

The deflection of the slab under applied live load results in relative rotation between the two girders. This effect is shown schematically in Figure 12. Assuming that the top flanges of the girders are attached to the slab using mechanical connectors, Figure 12 shows the exaggerated deformation that the outside girder would experience if the slab were subjected to downward load. This behavior causes the change in horizontal distance between the bottom flanges of the two girders, referred to hereafter as "girder separation."

Table 4 gives the maximum girder separation between

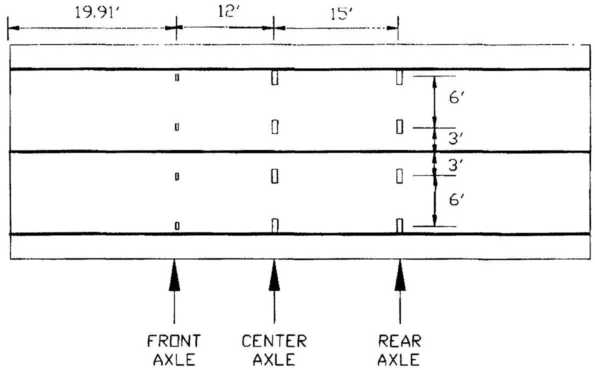

Fig. 8. Location of 12 hydraulic rams used for applying live load.

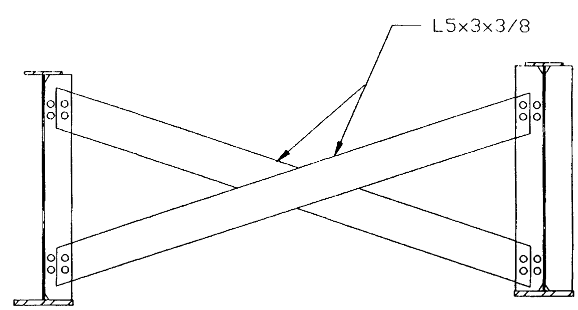

Fig. 9. Configuration of X type cross frames.

Test Number*	Cross Frame Type	Cross Frame Spacing, ft.	Loading Condition**	Maximum Load†	Maximum Transverse Concrete Strain in South Lane‡
1	*K*	11.2	Both Lanes	2.5×HS-20	–114
2	*K*	22.4	Both Lanes	2.5×HS-20	–138
3	*K*	22.4	South Lane	2.5×HS-20	–201
4	*K*	22.4	North Lane	2.5×HS-20	+38
5	*K*	22.4	Straddling	2.5×HS-20	–155
6	None	—	Both Lanes	2.5×HS-20	–218
7	None	—	South Lane	2.5×HS-20	–267
8	None	—	North Lane	2.5×HS-20	
9	None	—	Straddling	2.5×HS-20	–221
10	*X*	22.4	Both Lanes	2.5×HS-20	–182
11	*X*	22.4	North Lane	2.5×HS-20	+50
12	*X*	22.4	Straddling	2.5×HS-20	–163

Table 1.
Descriptions of live load tests conducted

*Each test consisted of applying three complete cycles of loading. Each cycle is defined as increasing the loads in rear rams to 40,000 lb. and front rams to 10,000 lb. load levels.
**Both Lanes = All 12 hydraulic rams were activated, simulated having two trucks side by side.
South Lane = The six rams in south lane were activated, simulating having one truck in south lane.
See Figure 4 for definition of south and north lanes.
North Lane = The six rams in north lane were activated simulating having one truck in north lane.
Straddling = The middle six rams were activated, simulating one truck straddling the centerline.
† During each test applied loads in front and rear wheels simulated having one or two trucks weighing 180,000 lbs. each. This weight correspond to 2.5 times the weight of AASHTO's HS-20 truck.
‡ The maximum concrete strains reported are transverse strains (perpendicular to traffic direction) measured in top surface of the slab at midspan in middle of the two girder lines. Compressive and tensile strains are represented by negative and positive signs respectively.

center and exterior girders for different cross frame types and spacings at mid span. Also included in this table are test results for the cases where all cross frames were removed except the end ones. The maximum observed separation is for the case of no cross frames and live loads being placed in the south lane only. Tests with *X* cross frames resulted in relatively higher separation compared to *K* cross frames. However, these deformations were elastic and the girders returned to their original positions after unloading, i.e., girder separations were zero after unloading.

d) Strains in Cross frames

Table 5 gives the resulting strains in the top, bottom, and one of the diagonal members of the *K* cross frames measured when the bridge was loaded with one or more trucks weighing 180,000 lbs. each. The "Girder Line" referred to in the table is described in Figure 4. Of particular interest is the fact that measured strains in all 3 members are tensile. This could be related to the tendency of the exterior girder to rotate under live loads as described in the previous section. Even at such a high live loading condition as used in these tests (2.5 times the AASHTO HS20), the girders returned to their original configuration after unloading, i.e., there was no permanent deformation. The consequence of restraining the girders from separating is to produce tensile forces in the top and bottom chord with the level of force in the top chord being higher. It is interesting to note that, in general, fatigue cracking problems observed in steel girder bridges utilizing similar cross frame configuration in positive moment regions are usually noticed at junction of top member to the girder web. Restraining the separation of girders could especially be a problem if the stiffeners attaching cross frames to the girder web are not completely connected to top and bottom flanges as is the case

Table 2. Maximum Bottom Flange Strain $\mu\varepsilon$							
		Maximum Strain at Midspan			Maximum Strain at Quarter Span		
Loading Condition	Cross Frame Type and Spacing, ft.	North Exterior Girder	Center Girder	South Exterior Girder	North Exterior Girder	Center Girder	South Exterior Girder
Both Lanes Loaded	*K* at 11.2	359	425	358	335	371	329
	K at 22.4	372	473	367	340	393	331
	X at 22.4	367	481	368	339	402	330
	None	349	505	351	320	424	317
Straddling Center Lane	*K* at 22.4	160	302	161	152	245	150
	X at 22.4	156	309	155	155	253	147
	None	139	353	138	133	294	130
South Lane Loaded	*K* at 22.4	35	237	334	32	199	297
	X at 22.4	29	237	331	28	204	299
	None	23	253	324	25	216	290

Table 3. Maximum Bottom Flange Stress Measured Experimentally and Predicted by AASHTO				
		Maximum Bottom Flange Stress		
Cross Frame Type	Cross Frame Spacing, ft.	Experimental (ksi)	AASHTO* (ksi)	AASHTO** (ksi)
K	11.2	4.45	8.14	7.57
K	22.4	4.69	8.14	7.57
X	22.4	5.04	8.14	7.57
None	—	5.29	8.14	7.57

*Using the load distribution factor calculated from Table 3.23.1 of AASHTO and N = 24 in calculation of composite properties.
**Using N = 8 in calculation of composite section properties.

for older details used. In this investigation stiffeners were attached to the top and bottom flanges.

Table 5 also gives the resulting strains for two different spacings (11.2 and 22.4-ft.) and four different loading conditions. Several observations could be made by studying the reported data. It should also be noted that all instrumented cross frames were located in the south lane.

a. Decreasing cross frame spacing decreases slightly the resulting strain in top and bottom chord.
b. The maximum tensile strain in diagonal members (357 micro strain) occurred when live loading simulated having one truck weighing 180,000 lbs. straddling the centerline.
c. The maximum top chord strain (55 micro strain) occurred when live loading simulated having one truck in south lane (lane in which instrumented cross frame was located) only. This coincided with the loading condition that produced maximum girder separation.

Table 6 shows the strains observed in *X* type cross frames. In all tests the spacing of cross frames was 22.4 ft. In general one diagonal member is in tension while the other in relatively small compression. This observation could be explained as follows. If the resulting strains in both members was just because of the relative vertical deflection of the girders, then it would be expected to have almost similar tension and compression strains in the diagonals. It can be shown that the relative vertical deflection of the girders combined with rotation of the girders results in reduction of the compressive strain in the diagonal member that is in compression and increase in tensile strain of the diagonal member that is in tension.

e) Girder Deflections

The effects of altering cross frame spacing and type on deflection of interior and exterior girders are summarized in Table 7. The deflections are reported at quarter and midspan of interior and one of the exterior girders. All deflections are the maximum values observed and correspond to loading simulating having one or two trucks each weighing 180,000 lb. on the bridge in a position producing the maximum moment in the girders. In the case of loading condition desig-

Table 4. Girder Separations				
Loading Condition	Cross Frame Type	Cross Frame Spacing, ft.	Maximum Separation of	
			South Exterior Girder (in.)	North Exterior Girder (in.)
Both Lanes Loaded	*K*	22.5	0.043	0.034
	X	22.5	0.069	0.061
	None	—	0.189	0.158
Staddling Center Lane	*K*	22.5	0.045	0.049
	X	22.5	0.076	0.079
	None	—	0.214	0.228
South Lane Loaded	*K*	22.5	0.053	0
	X	22.5	0.062	0
	None	—	0.246	–0.032*

*Negative value indicates a decrease in distance between girders.

nated as "one lane loaded," deflections are reported for the exterior girder located in the side of loaded lane. All deflections are corrected for the end displacements produced by flexibility of the bearing pads.

From the results shown in this table the following conclusions could be drawn:

a. in the case of *K* cross frames, increasing the spacing from 11.2 to 22.4-ft. has negligible effect in girder deflection, both for exterior and interior girders.
b. the deflection of the girders for tests with *X* and *K* type cross frames differ very slightly.
c. in the case of no intermediate cross frames, deflections of the exterior girders are decreased while the deflection of the interior girder is increased. However, again, this change in deflection is small. The main reason for this behavior is that the contribution of the slab in distributing load between girders is more pronounced than that of the cross frames. In the case of no intermediate cross frames the percent difference in deflection of girders compared to having *X* or *K* type cross frames at 22.4-ft. spacing is higher when only one lane is loaded or load straddle the centerline. However, it should be noted that the resulting deflections for both interior and exterior girders in these cases are approximately 50 percent of the case where both lanes are loaded.

f) Load Distribution Factors

Using the experimental results, load distribution factors for

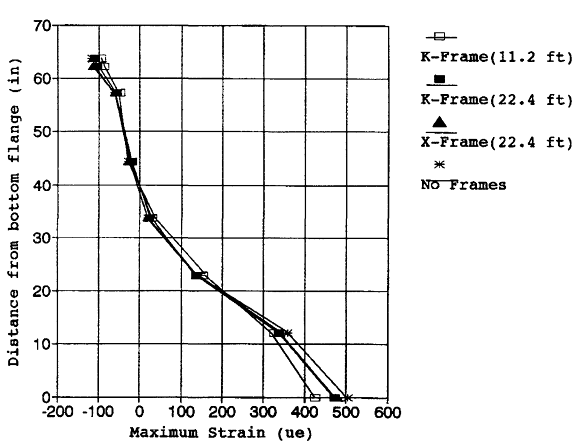

Fig. 10. Strain distrubution along the depth; interior girder.

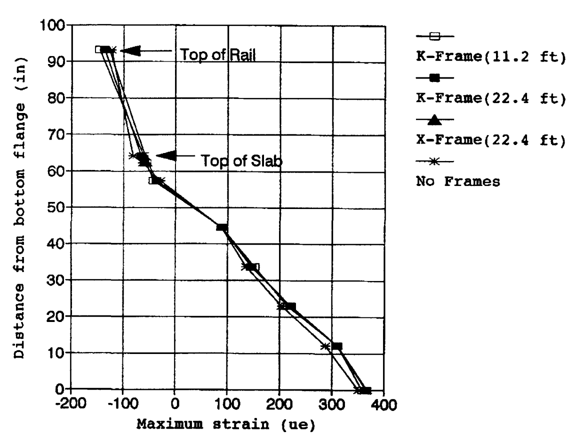

Fig. 11. Strain distribution along the depth; exterior girder.

Table 5.
Maximum Observed Strain in *K* Type Cross Frames

Loading Condition		Two Lane	Two Lane	Straddling Center Lane	North Lane	South Lane
Spacing of Cross Frames		**11.2 ft.**	**22.4 ft.**	**22.4 ft.**	**22.4 ft.**	**22.4 ft.**
Girder Line A-2*	Top Chord	14				
	Bottom Chord	3				
	Diagonal	94				
Girder Line A-3*	Top Chord	27	32	33	3	31
	Bottom Chord	6	15	13	–1	16
	Diagonal	171	229	301	125	123
Girder Line A-4*	Top Chord	36				
	Bottom Chord	7				
	Diagonal	222				
Girder Line A-5*	Top Chord	33	40	37	–8	55
	Bottom Chord	12	26	22	3	26
	Diagonal	200	264	357	158	132
Girder Line A-6*	Top Chord	15				
	Bottom Chord	2				
	Diagonal	116				

*Refer to Figure 4 for girder line designation. Also note that cross braces along lines A-2, A-4, and A-6 were removed when cross frames were spaced at 22.4 ft. on center.

different cross frame configurations were obtained. However, before proceeding, a brief description of what this paper defines as load distribution factor will be provided.

The need for load distribution factor arises from the fact that current design approaches use two-dimensional models to approximate the real behavior of bridge. Therefore the load distribution factor could be viewed as a correction or correlation factor relating specific response (such as a maximum stress in bottom flange of girder) of a bridge component in a real structure to the same response of a simple model of that component. So, if one is interested in approximating the maximum tensile stress in the bottom flange of a girder in a bridge, then one should use an appropriate correlation factor (or load distribution factor) that was obtained based on stress consideration.

In current design approach an important limit-state criterion is the level of stresses in the steel girder portion of the bridge. Therefore in this study the distribution factor from experimental results were obtained based on stress considerations. The procedure that was used to calculate the distribution factors could be summarized as follows.

First a simply supported beam was loaded with three

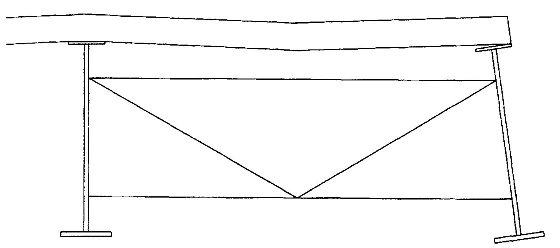

Fig. 12. Schematic representation of girder rotation.

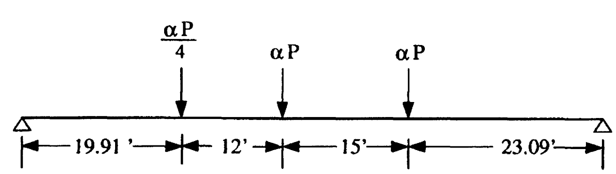

Fig. 13. Beam model and loading condition.

Table 6. Maximum Observed Strain in *X* Type Cross Frames					
Loading Condition		**Two Lane**	**Straddling Center Line**	**North Lane**
Spacing of Cross Frames		**22.5 ft.**	**22.5 ft.**	**22.5 ft.**
Girder Line A-3	Tension Member	244	374	152
	Compression Member	–4	–25	–28
Girder Line A-5	Tension Member	257	394	146
	Compression Member	4	–25	–37

Table 7. Maximum Girder Deflections						
			Maximum Deflection At			
			Quarter Point		**Mid Span**	
Loading Condition	**Cross Frame Type**	**Cross Frame Spacing, ft.**	**Interior Girder (in.)**	**Exterior Girder (in.)**	**Interior Girder (in.)**	**Exterior Girder (in.)**
Both Lanes Loaded	*K*	11.2	0.448	0.397	0.651	0.561
	K	22.4	0.477	0.401	0.687	0.567
	X	22.4	0.5	0.4	0.717	0.571
	None	—	0.516	0.385	0.749	0.549
Straddling Center Line	*K*	22.4	0.288	0.193	0.421	0.265
	X	22.4	0.302	0.182	0.445	0.255
	None	—	0.348	0.161	0.509	0.228
One Lane Loaded	*K*	22.4	0.234	0.363	0.343	0.513
	X	22.4	0.252	0.357	0.356	0.511
	None	—	0.256	0.349	0.371	0.498

concentrated loads with spacing corresponding to axle spacing used in the experimental investigation (approximately the same axle spacing as that of AASHTO HS20 truck). The three concentrated loads had relative magnitudes corresponding to HS20 truck wheel loads as indicated in Figure 13. The α factor in front of each concentrated load in Figure 13 is the distribution factor. From Figure 13 the maximum moment, M_a, as a function of α factor is then calculated. Next, from the experimental results, the maximum observed strain in bottom flange of the interior and exterior girders was obtained. The values were taken from tests corresponding to having two trucks side by side at a load level corresponding to AASHTO HS20 truck load (i.e. 4,000 and 16,000 lbs. under front and rear wheels, respectively). Using the average modulus of elasticity obtained from material tests, these strains were then converted to stress. Using these stresses, experimentally obtained moments were calculated using the following formula:

$$M_{exp} = f_t \, s_b$$

where:

M_{exp} = Maximum moment obtained from experimental results
S_b = Section modulus of the composite section with respect to bottom flange
f_t = Experimentally obtained tensile bottom flange stresses

To be compatible with assumptions used in AASHTO procedures for design of composite sections in calculating the S_b, the section modulus, the assumption was made that the effec-

Table 8. Load Distribution Factors							
Cross Frame Type	Cross Frame Spacing, ft.	Experimental		Current AASHTO		1994 AASHTO Provisions*	
		Interior	Exterior	Interior	Exterior	Interior	Exterior
K	11.2	1.21	1.03	1.82	1.82	1.62	1.62
K	22.4	1.34	1.05	1.82	1.82	1.62	1.62
X	22.4	1.38	1.06	1.82	1.82	1.62	1.62
None	—	1.42	1.01	1.82	1.82	1.62	1.62

*See Reference 1.

tive width of the slab is the same as that specified by the AASHTO manual. Finally, by equating the maximum moment obtained from experimental results, M_{exp}, to that obtained from Figure 13, M_a, the load distribution factor, α, was obtained. Table 8 provides a summary of distribution factors obtained for different cross frame configurations. Also given in this table are the distribution factors predicted by current AASHTO procedures and those given by AASHTO's new provisions,[1] "Guide Specifications for Distribution of Loads for Highway Bridges" (Ibsem formulas). The following observations could be made from studying the data reported in this table:

a. distribution factors obtained experimentally are smaller for exterior girders when compared to interior girders. This could be attributed mainly to strengthening effects of the railing system. The analytical models used in developing Ibsem formulas do not include the effect of railing systems.
b. the *X* and *K* type cross frames spaced at 22.4 ft. resulted in almost the same distribution factors.
c. in the case of no cross frames, the increase in distribution factor for interior girder and decrease in distribution factor for exterior girder was very small when compared to the case of *X* or *K* type cross frame spaced at 22.4 ft. However, the distribution factors for the case of having no cross frame obtained experimentally is still smaller than what is predicted by AASHTO or Ibsem formula.

CONCLUSIONS

From the results obtained from this investigation, the following conclusions could be made.

For steel bridges with small skew, although cross frames may be needed during construction, their presence has little influence on behavior of steel bridges after construction. Results indicate that after construction, cross frames not only are unnecessary but are, to a degree, harmful, as they try to prevent the small tendency of the girders to separate during elastic ranges, consequently transferring restraining forces to girder webs which has been shown to cause cracking. After construction, the stiffness of the slab is sufficient to distribute the live load to adjacent girders. It could be argued that cross frames are needed to provide redundancy in the bridge, i.e., cross frames could be used to provide alternate load paths in the event of failure in such elements as the concrete deck. In this scenario, i.e., failure of one of the main load carrying members, it is unlikely that cross frames could provide such a function and bridge failure would be imminent anyway. This is especially important given the fact that most problems in steel bridges are caused by the presence of cross frames to begin with. Results of this research indicate that if it is desired to leave cross frames in place, utilizing simpler forms of cross frames such as the *X* type provides as good behavior as the more expensive *K* or other types. Another application of this conclusion could be in retrofiting old steel girder bridges. In cases where cracking in elements connecting cross frames to the girder or girder webs is observed, a viable solution could be removal of cross frames altogether, avoiding costly repair expenses.

ACKNOWLEDGMENT

This project is sponsored by the Nebraska Department of Roads (NDOR), to which the authors are very appreciative. Valuable input by Gale Barnhill, Structural Engineer, Lyman Freemon, Bridge Engineer, and Mo Jamshidi, Assistant Bridge Engineer at the Bridge Division of NDOR, is greatly acknowledged. During the course of the investigation technical input was provided by Robert Nickerson for which the authors are thankful. The assistance of Jim Luedke, Yerapalli Shekar, and Bruce Keeler in conducting the experimental and analytical studies is acknowledged. Additional support was provided by the Center for Infrastructure Research (CIR) at the University of Nebraska-Lincoln.

REFERENCES

1. "Guide Specification for Distribution of Loads for Highway Bridges," AASHTO, March 1994.