Effect of Cross Frames on Behavior of Steel Girder Bridges

Atorod Azizinamini, *University of Nebraska–Lincoln*
Steve Kathol, *Schemmer Associates, Inc.*
Mike Beacham, *Nebraska Department of Roads*

Although cross frames are required before construction, their usefulness after construction has been questioned. A combination of experimental and analytical studies was conducted to investigate the performance of steel girder bridges that use different types and spacing of cross frames. The experimental investigation included construction and testing of a full-scale steel girder bridge in the laboratory. Unique characteristics of the bridge include a concrete slab designed on the basis of AASHTO's 1994 empirical design approach, which requires a minimal amount of reinforcement. Elastic and ultimate load tests were carried out, and punching shear tests were conducted after the ultimate load tests. Results of the research indicate that for bridges with zero skew, the influence of cross frames is minimal. Ultimate tests indicate that steel girder bridges have large reserve capacities. Very large punching shear capacity of the slab was also observed.

The 15th edition of the AASHTO manual requires intermediate cross frames in steel girder bridges with maximum spacing of 7.63 m. Although these cross frames may be needed for temporary loads, their effectiveness after construction has been a point of debate for short- and medium-span bridges.

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

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 simple span 21.35 m long and 7.93 m wide. Figure 1 shows a photograph of the completed bridge in the laboratory. The superstructure consisted of three welded plate girders built compositely with a 190.5-mm-thick reinforced concrete deck. The girders, each 1.37 m deep, were spaced 3.05 m on center, and the reinforced concrete deck had a 0.915-m overhang. The concrete barrier structure was an open concrete bridge rail, with 280- × 280-mm posts spaced 2.44 m on center. The construction sequence was identical to that of field practice.

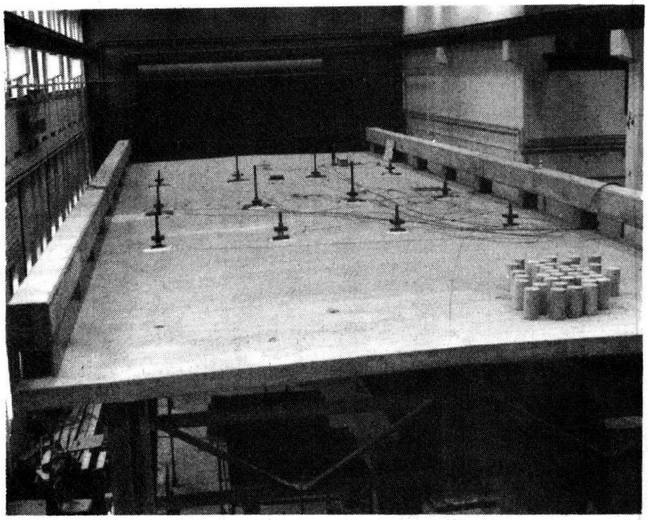

FIGURE 1 Completed bridge.

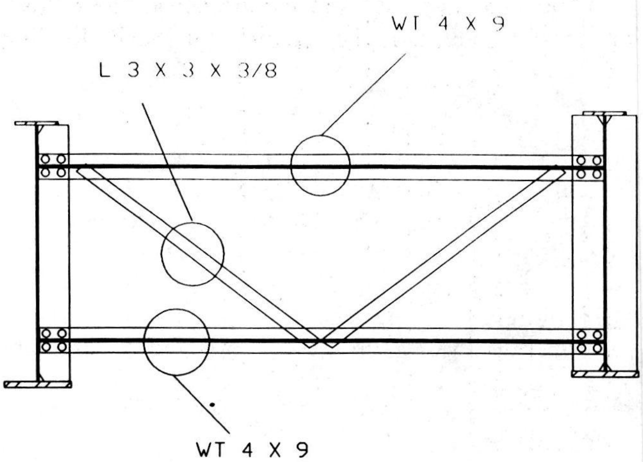

FIGURE 2 Member sizes used for K-type cross frames.

During construction, K-type cross frames at 3.42-m spacing, as shown in Figure 2, were used. This resulted in a total of seven cross frames in each lane.

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

The behavior of the bridge was monitored continuously for approximately 102 days following casting of the concrete deck before live load tests were conducted 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 gauges. In general, the maximum strain resulting from dead load and creep and shrinkage was very small; the resulting stress did not exceed 11.72 MPa in members of the cross frames monitored.

Live Load Test Setup

Live loads were applied by using 12 hydraulic rams, shown in Figure 1. 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 3 shows the locations of all 12 rams, which simulated two trucks side by side on the bridge. The spacing of axles, as shown in Figure 3, is 3.66 and 4.58 m instead of 4.27 and 4.27 m as specified by AASHTO. This spacing was a result of laboratory constraints. 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) on truck straddling 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

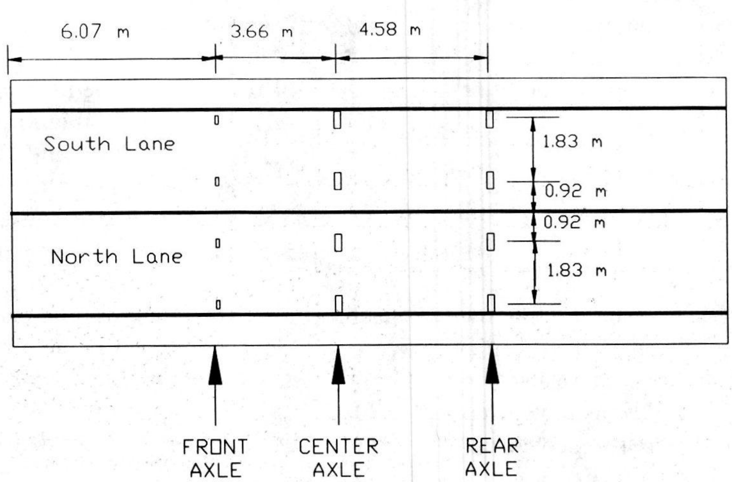

FIGURE 3 Location of 12 hydraulic rams used for applying live load.

size of the footprint was determined from the AASHTO manual. The plates used in the rear and front axles were 508 × 203 × 50.8 mm and 254 × 101.6 × 50.8 mm, respectively.

Test Descriptions and Results

Table 1 gives a list of 12 tests. 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 1, K-type cross frames at 3.42-m spacing were used. In this test all 12 rams were activated to stimulate two trucks side by side. Further, during Test 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 (800 kN) on each lane. This load level was achieved by increasing load points on rear and front axles to 178 and 44.48 kN, respectively.

During Tests 2 through 5, K-type cross frames at 6.83-m spacing were used. All but the end cross frames were removed during Tests 6 through 9. Tests 10 through 12 used X-type cross frames of the configuration shown in Figure 4 at 6.83-m spacing.

The effect of cross frame types and spacing on response of the bridge to applied live loads will be discussed in terms of maximum strain in steel girders, maximum girder deflections, and load distribution factors.

Strain in Steel Girders

The effect on the behavior of steel girders of altering cross frame spacing and type will be discussed in terms of maximum tensile strain in the bottom flange of interior and exterior girders at the mid- and quarter spans of the bridge. In all cases the applied live load corresponds to one or two trucks being on the bridge, each truck weighing 800 kN. A summary of strains measured at midspan and quarter span of the bridge for different cross frame configurations and loading conditions is 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.

TABLE 1 Descriptions of Live Load Tests

Test Number+	Cross frame Type	Cross frame Spacing, meters	Loading Condition#	Maximum Load *
1	K	3.42	Both Lanes	2.5 x HS-20
2	K	6.83	Both Lanes	2.5 x HS-20
3	K	6.83	South Lane	2.5 x HS-20
4	K	6.83	North Lane	2.5 x HS-20
5	K	6.83	Straddling	2.5 x HS-20
6	None	-----	Both Lanes	2.5 x HS-20
7	None	-----	South Lane	2.5 x HS-20
8	None	-----	North Lane	2.5 x HS-20
9	None	-----	Straddling	2.5 x HS-20
10	X	6.83	Both Lanes	2.5 x HS-20
11	X	6.83	North Lane	2.5 x HS-20
12	X	6.83	Straddling	2.5 x HS-20

+ Each test consisted of applying three complete cycles of loading. Each cycle is defined as increasing the loads in rear rams to 178 kN and front rams to 44.48 kN load levels.

Both Lanes = All 12 hydraulic rams were activated, simulating having tow truck side by side

South Lane = The six rams in south lane were activated, simulating having one truck in south lane. See Fig. 3 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 800 kN each. This weight corresponds 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

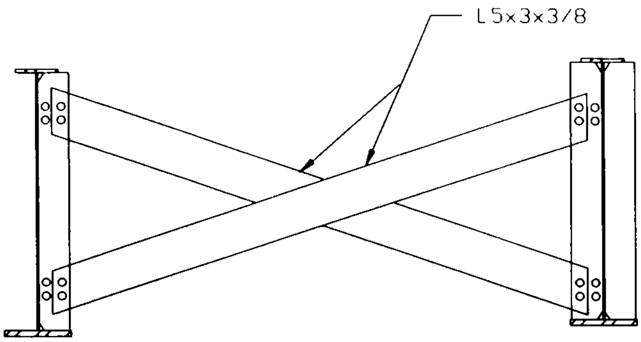

FIGURE 4 Configuration of X-type cross frames.

Both Lanes Loaded

Table 2 gives maximum bottom flange strains at midspan and quarter point for the cases in which both lanes are loaded. Comparison of test results for K cross frames at 3.42- and 6.83-m spacing indicates that the maximum tensile strain in the bottom flange of the center girder increased slightly as the spacing of cross frames increased. The difference in exterior girder strain is not significant enough to make definite conclusions. As indicated 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 with cases in which X- or K-type cross frames at 6.83-m 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 in which K-type cross frames are used at 6.83-m spacing. This behavior indicates that load distribution factors are affected only slightly 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 800-kN truck straddling the bridge centerline. Results indicate that X- and K-type cross frames result in the same behavior. From this table it can be observed that in the case of no cross frames, the resulting strain in the center girder is 14.5 percent higher than it is with K cross frames at 6.83-m spacing. In evaluating this result two points are important. First, the difference is relatively small. Second, although the case of one truck straddling the centerline results in higher percentage differences, the magnitude of the resulting strain in the bottom flange is much smaller than it is when both lanes are loaded. This is important when considering that the design situation will be governed when both lanes are loaded.

Loading One Lane Only

Table 2 presents maximum bottom flange strains observed at midspan and quarter point with only the south lane loaded with a truck weighing 800 kN. 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 Table 2, although the percentage difference in exterior girder (north lane)

TABLE 2 Maximum Bottom Flange Strains

Loading Condition	Cross frame Type and Spacing meter	Maximum Strain at Midspan ($\mu\epsilon$)			Maximum Strain at Quarter span ($\mu\epsilon$)		
		North Exterior Girder	Center Girder	South Exterior Girder	North Exterior Girder	Center Girder	South Exterior Girder
Both Lanes Loaded	K at 3.42	359	425	358	335	371	329
	K at 6.83	372	473	367	340	393	331
	X at 6.83	367	481	368	339	402	330
	None	349	505	351	320	424	317
Straddling Center Lane	K at 6.83	160	302	161	152	245	150
	X at 6.83	156	309	155	155	253	147
	None	139	353	138	133	294	130
South Lane Loaded	K at 6.83	35	237	334	32	199	297
	X at 6.83	29	237	331	28	204	299
	None	23	253	324	25	216	290

strain is larger than that with K-type cross frames at 6.83-m spacing (23 micro strain compared with 35 micro strain), the magnitude of resulting strains in exterior girders is much smaller than when both lanes are loaded (23 microstrain compared with 349 microstrain).

Girder Deflections

A summary of the effects on deflection of interior and exterior girders of altering cross frame spacing and type is given in Table 3. The deflections are reported at midspan and quarter point of the interior and one of the exterior girders. All deflections are the maximum values observed and correspond to loading that simulates one or two 800-kN trucks on the bridge in a position producing the maximum moment in the girders. For the loading condition designated as "one land loaded," deflections are reported for the exterior girder located in the side of the loaded lane. All deflections are corrected for the end displacements produced by the flexibility of the bearing pads.

From the results given in this table, the following conclusions can be drawn:

- In the case of K cross frames, increasing the spacing from 3.42 to 6.83 m has a negligible effect on girder deflection, for both exterior and interior girders.
- The deflection of the girders for tests with X- and K-type cross frames differs very slightly.
- In the case of no intermediate cross frames, the 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. When there are no intermediate cross frames, the percentage difference in deflection of girders compared with having X- or K-type cross frames at 6.83-m spacing is higher when only one lane is loaded or when the load straddles the centerline. However, the resulting deflections for both interior and exterior girders in these cases are approximately 50 percent of what they are when both lanes are loaded.

Load Distribution Factors

By using the experimental results, load distribution factors for different cross frame configurations were obtained. First, however, load distribution factor, as used in this paper, will be defined briefly.

A load distribution factor is needed because current design approaches use two-dimensional models to approximate the real behavior of a bridge. Therefore, the load distribution factor can be viewed as a correction or correlation factor relating specific responses (such as a maximum stress in the bottom flange of the 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 girders in a bridge, one should use an appropriate correlation factor (or load distribution factor) that was based on stress considerations.

In the current design approach, an important limit state criterion is the level of stresses in the steel girder

TABLE 3 Maximum Girder Deflections

Loading Condition	Cross frame Type	Cross frame Spacing meter	Maximum Deflection			
			Quarter Point		Mid Span	
			Interior Girder (mm.)	Exterior Girder (mm.)	Interior Girder (mm.)	Exterior Girder (mm.)
Both Lanes Loaded	K	3.42	11.38	10.08	16.54	14.25
	K	6.83	12.12	10.19	17.45	14.4
	X	6.83	12.7	10.16	18.21	14.5
	None	-----	13.1	9.78	19	13.95
Straddling Center Line	K	6.83	7.32	4.9	10.69	6.73
	X	6.83	7.67	4.62	11.3	6.48
	None	-----	8.84	4.09	12.93	5.79
One Lane Loaded	K	6.83	5.94	9.22	8.71	13.03
	X	6.83	6.4	9.07	9.04	12.98
	None	-----	6.5	8.86	9.42	12.65

portion of the bridge. Therefore, in this study the distribution factor from experimental results was obtained using stress considerations. The procedure used to calculate the distribution factors can be summarized as follows.

First, a simply supported beam was loaded with three concentrated loads with spacing corresponding to axle spacing used in the experimental investigation (approximately the same axle spacing as that of an AASHTO HS20 truck). The three concentrated loads had relative magnitudes corresponding to HS20 truck wheel loads as indicated in Figure 5. The α-factor in front of each concentrated load in Figure 5 is the distribution factor. From Figure 5, the maximum moment, M_a, as a function of the α-factor is then calculated. Next, from the experimental results, the maximum observed strain in the bottom flange of the interior and exterior girders was obtained. The values were taken from tests simulating two trucks side by side at a load level corresponding to AASHTO HS20 truck loads (i.e., 17.8 and 71.2 kN under the front and rear wheels, respectively). Using the average modulus of elasticity obtained from material tests, these strains were 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 composite section with respect to bottom flange, and
- f_t = experimentally obtained tensile bottom flange stresses.

To ensure compatibility with assumptions used in AASHTO procedures for the design of composite sections, in calculating the S_b, the section modulus, it was assumed that the effective 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 5 M_a, the load distribution factor, α, was obtained. Table 4 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, *Guide Specifications for Distribution of Loads for Highway Bridges* (Ibsem formulas) (*1*).

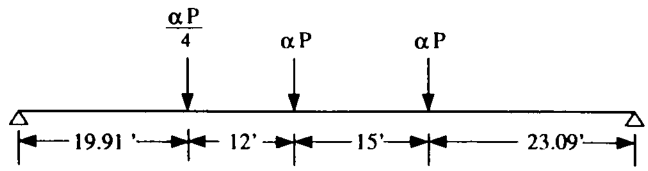

FIGURE 5 Girder model and loading condition.

The following observations can be inferred from the data reported in this table:

- Distribution factors obtained experimentally are smaller for exterior girders than for interior girders. This result can be attributed mainly to the strengthening effects of the railing system. The analytical model used in developing Ibsem formulas does not include the effect of railing systems.
- The X- and K-type cross frames spaced at 6.83 m resulted in almost the same distribution factors.
- In the case of no cross frames, the increase in distribution factor for interior girder and decrease in distribution factor for exterior girder were very small when compared with the case of X- or K-type cross frames spaced at 6.83 m. However, the distribution factors obtained experimentally are still smaller than what the AASHTO or Ibsem formulas predict.

ULTIMATE LOAD TEST

After the elastic load test just described, the bridge was loaded to collapse. During the ultimate load all 12 hydraulic rams were activated, to simulate having two HS20 trucks side by side. During the ultimate load test

TABLE 4 Load Distribution Factors

Cross frame Type	Cross frame Spacing, meter	Experimental		Current AASHTO		1994 AASHTO Provisions (*1*)	
		Interior	Exterior	Interior	Exterior	Interior	Exterior
K	3.42	1.21	1.03	1.82	1.82	1.62	1.62
K	6.83	1.34	1.05	1.82	1.82	1.62	1.62
X	6.83	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

all cross frames were removed except those at the end. Figure 6 gives the resulting deflection at midspan of the middle girder versus the total applied load. The applied loads are given in terms of the number of HS20 truck loads weighing 320 kN each. A safety concern arose during the ultimate load test when the applied total load was approximately equal to 14 times the HS20 truck load. Consequently, the bridge was unloaded to correct the problem. Upon reloading, the load deflection curve followed the unloading path as indicated in Figure 6. This indicates that even loading and unloading the bridge at such a high load level did not alter the composite action between the girders and the concrete slab. The bridge failed when the total applied load approached the equivalent of 16 AASHTO HS20 truck loads. Failure was by punching shear at one of the loading points. At the time of failure, the maximum strain in the bottom flange of the interior girder at midspan was 10,900 microstrain. The permanent deflections of interior and exterior girders after complete unloading after failure were approximately 36 and 102 mm, respectively.

It is intersting that the bridge was designed for an AASHTO HS20 truck load (320 kN) and that the ultimate live load capacity of the bridge according to load factor design approach stated in the 15th edition of the AASHTO manual is approximately 2.7 times the HS20 truck load (in each lane). On the other hand, the bridge failed at a load corresponding to eight times the HS20 truck load in each lane, indicating a large reserve load-carrying capacity. This large reserve capacity could be attributed to (a) actual material properties that are higher than assumed smaller values in design, (b) a conservative approach to calculating distribution factors, (c) simplified two-dimensional analysis approaches to calculate induced moments in bridge girders, and (d) conservative effective width values used in the design process.

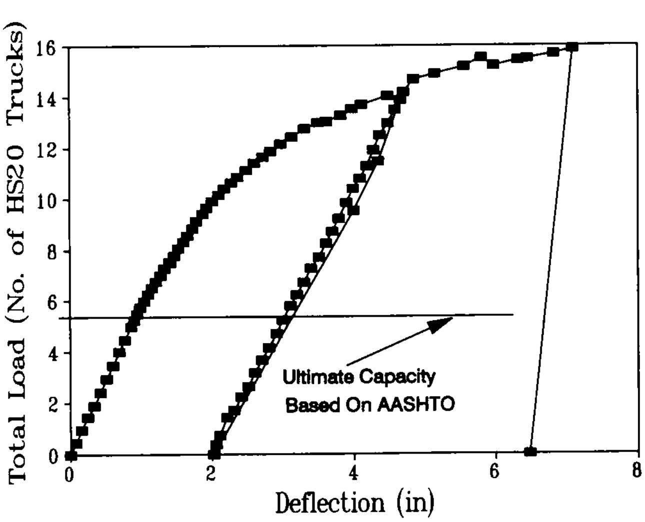

FIGURE 6 Load deflection response of interior girder at midspan during ultimate load test.

Punching Shear Tests

After the ultimate load tests, a series of punching shear tests was conducted on the bridge. These tests consisted of applying a single concentrated load at various locations over the concrete slab. Because of the ultimate load tests, the concrete slab bridge had experienced extensive cracking. Four punching shear tests were conducted, and despite the extensive cracking over the slab, the punching shear capacities obtained were between 543 and 694 kN: the capacity of 543 kN was obtained in the area where extensive cracking from the ultimate load test took place, and that of 694 kN was obtained in the area where relatively no cracking was present before the punching shear tests. These results indicate that the empirical design rule procedure as stated in the AASHTO 1994 load resistance factor design (LRFD) manual results in large punching shear capacity, even in the absence of cross frames and relatively large girder spacing.

Conclusions

From the results of this investigation, the following conclusions can be made.

For a steel bridge with small skew, cross frames may be needed during construction, but their presence has little influence on the behavior of the bridge after construction. After construction, the stiffness of the slab is sufficient to distribute the live load to adjacent girders. It can be argued that cross frames are needed to provide redundancy in the bridge—that is, cross frames could be used to provide alternative load paths if such elements as the concrete deck were to fail. In this scenario—failure of one of the main load-carrying members—it is unlikely that cross frames could provide such a function and the bridge would fail anyway. This is especially important given the fact that many 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, the use of simpler forms of cross frames such as the X type provides behavior as good as that of the more expensive K or other types. Another application of this conclusion could be in retrofitting old steel girder bridges. Where cracking in elements connecting cross frames to the girder or girder webs is observed, the cross frames could be removed altogether to avoid costly repair expenses.

Results of ultimate load tests conducted on the bridge without any intermediate cross frames indicate that simplified and conservative assumptions made during design have given steel girder bridges a very large reserve capacity. If a bridge is in good condition, this reserve capacity could be used in rating older bridges designed for smaller truck loads than they are required to carry.

Results of punching shear tests indicate that the empirical design rule stated in the AASHTO 1994 LRFD manual results in a very adequate slab design procedure.

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REFERENCE

1. *Guide Specification for Distribution of Loads for Highway Bridges.* AASHTO, Washington, D.C., March 1994.