Effect of Trucks upon a Few Bridge Floors in Iowa in 1922 and in 1948

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SYNOPSIS

INVESTIGATIONS for impact were made during the summers of 1922 to 1925 when the trucks were entirely different from those now in use. The present discussion is concerned not with definite values, but with various factors such as (1) the relation of the force of a wheel blow upon pavement to that of a similar blow upon a more flexible bridge floor and (2) the relation of the stress which is developed in the stringers and floor beams by a dynamic blow and by a static load of the same weight as the force of the blow.

Initial static readings for deformation were taken primarily as a basis for dynamic readings. These readings also showed the comparative deformations of the various longitudinal stringers, or in other words, the distribution of load among the stringers. They also pointed definitely to the action of the reinforced-concrete floor slab in relieving the steel stringers from much of the stress which would have been developed if they alone had been carrying all the load.

A brief exploratory research was undertaken in 1948 as a means of observing whether the composite action between the concrete floor slab and the steel stringers had remained after 28 years of service.

The results indicate that in a 34-ft. I-beam span the composite action was still effective; and that in a panel of a truss bridge, although the bond was apparently broken, the deformations and resulting stresses were less than if the steel alone was supporting the load.

The results also indicate that in the 20's and in 1948 the load transferred to the most loaded stringer was (for these closely spaced stringers) somewhat in line with the provisions of the AASHO specification.

● THE SIGNIFICANT results of the effect of trucks upon impact and stresses in the floors of 12 bridges in central Iowa, with which the writer has been connected, have been published (*1, 2, 3, 4, 5*). Four of the published reports are on the researches conducted from 1922 to 1925 and include the effects of both static and dynamic loads. Another of the publications deals only with a few tests from static loading which were made in 1948 as an exploratory research for the purpose of roughly approximating the effects of a quarter century of traffic upon the composite action between the reinforced-concrete floor slab and the steel stringers and floor beams.

The researches of the early 1920's were directed at the problem of impact in highway bridges. The available loads were Liberty trucks which weighed about 3 1/2 tons and, although rated for 5 tons of load, were loaded with gravel to a total of about 15 tons with 12 tons on the rear axle. The maximum speed attainable was 15 mph. Although 12 bridges were included in the study, the greater part of the data, especially for concrete floor on steel stringers, was secured from an 18-ft.-9-in. panel of a 150-ft. steel truss span, and a 32-ft.-8-in. beamspan, with 20-ft. roadway and a 24-ft. steel beam span with 24-ft. roadway. The dimensions of all of the spans and the loads are given in the report on the work (*3*).

The impact results of this and other available work were reviewed by the Committee on Impact in Highway Bridges of the American Society of Civil Engineers, of which the author was the chairman. A report was presented at the annual meeting in 1929 (*4*). All available data were based upon loads which seldom produced a static unit stress as great as 10,000 psi., and most stresses were far below that figure. Many impacts were reported of several hundred percent, but they were for light loads, largely unsprung, with the greatest dynamic unit stresses around 16,000 psi. and with the truck wheels going over artificial obstructions up to 2 by 4 in. The tires were well-worn solid rubber.

The recommendations of the committee in regard to impact in floors (*4*) are:

1. That stresses due to static loads and to impact are important, as regards the safety of the structure, only when they approach design values

2 That the percentage of impact increment decreases as the loads increase, and therefore as the unit stresses increase

3. That the larger impacts observed in the tests were produced by obstructions such as would be accidental and infrequent under actual traffic conditions.

4. That the actual occurrence, on a bridge, of loads having a magnitude corresponding to those used in the design of modern structures is infrequent.

5 That the simultaneous occurrence on a bridge floor of a maximum truck load and an accidental obstruction capable of producing high impact will be such a rare coincidence that presumbably the factor of safety usually will provide safety for this condition.

This committee recommends, therefore, that for the design of highway bridge floors and floor-beam suspenders, the impact increment of stress be assumed as 25% of the live load stress It should be used only when the floors are sufficiently smooth to conform to good modern practice, and unusual conditions should be provided for in accordance with the judgment of the individual designer The committee believes that this report contains information, with necessary precision, for guidance in unusual conditions.

The "information . . . for guidance in unusual conditions" is given in appendices. It may be separated for convenience into five steps: (1) the force of a blow of a truck wheel upon a concrete pavement; (2) the force of a blow of a truck wheel upon a bridge floor; (3) the relation between force of a blow upon pavement and the force of a similar blow upon the floor of a bridge; (4) the relation between the blow upon a bridge floor and the stress in a stringer or floor beam; and (5) the relation between the stress from a blow and the stress which would be developed by a static load of the weight of the force of a dynamic blow.

These five steps have been fairly well developed for the trucks and the bridges which were available in and around Ames, Iowa, from 1922 to 1925. The tires of the heavy trucks were solid rubber and well-worn (*3*). A limited amount of supplementary data is available from work done at Iowa State College, in 1927 (*4*) and from investigations of the effects of blows on pavement by the Bureau of Public Roads (*6, 7, 8, 9, 10*).

For additional studies of dynamic action upon bridge floors the force of a blow upon pavement for any given truck (step 1) could be obtained from any reliable observations such as those by the Bureau of Public Roads or from other sources.

TABLE 1

DYNAMIC FORCE OF WHEEL BLOWS IN KIPS ON PAVEMENT AND ON THREE BRIDGES AT 12 MPH

Structure*	Obs	Liberty Loaded		Liberty Empty		Light Hwy Truck	
		Force	%	Force	%	Force	%
Pavement	1"	48 0	100	37 6	100	9 8	100
S M	1"	—	—	28 5	76	8 3	85
S A	1"	36 0	75	34 2	91	8 8	90
C T	1"	43 2	90	—	—	—	—
Pavement	2"	64 6	100	56 5	100	18 3	100
S A	2"	51 0	79	48 1	85	16 2	89
C T	2"	60 9	94	—	—	—	—
Static Wheel - Load		8 800		3 350		1 350	
Unspring Wheel-Load		2 200		2 200		1 000	
Percent Unsprung		25 0		66		74	
Dynamic Unit Stresses Developed in one Stringer 2 5 ft South of Center Line							
S M	1"	11 2		5 0		—	
S A	1"	6 8		5 8		1 6	
S M	2"	15 6		—		—	
S A	2"	9 1		7 8		2 8	
C T	2"	4 9		—		—	

* S M Skunk River Main Span
S A Skunk River Approach
C T Campus Test

The force of a wheel blow upon a bridge floor (step 2) might be secured directly from available sources, by field observations, or it could be computed within a reasonable tolerance by the use of Equation 1, Appendix C of the reference (*4*).

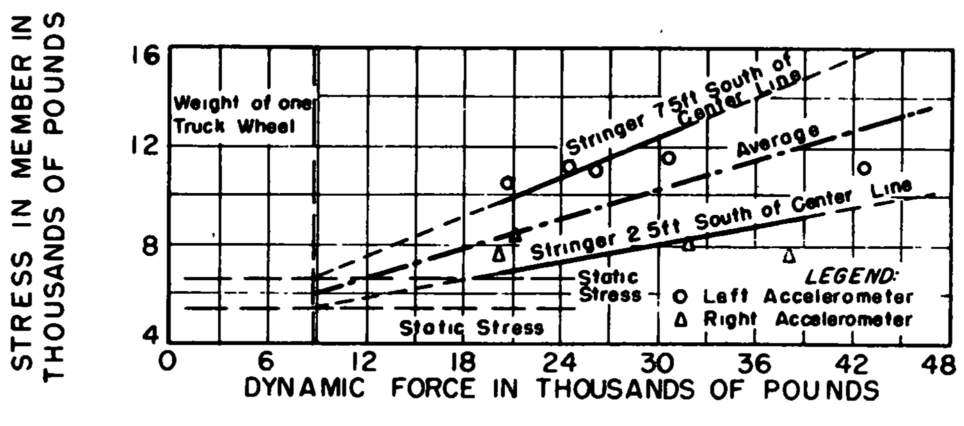

Figure 1. Relationship between dynamic force and simultaneous stresses in stringers 2 1/2 and 7 1/2 ft. south of centerline of Skunk River main span, Truck B; 1- by 2-in. obstruction.

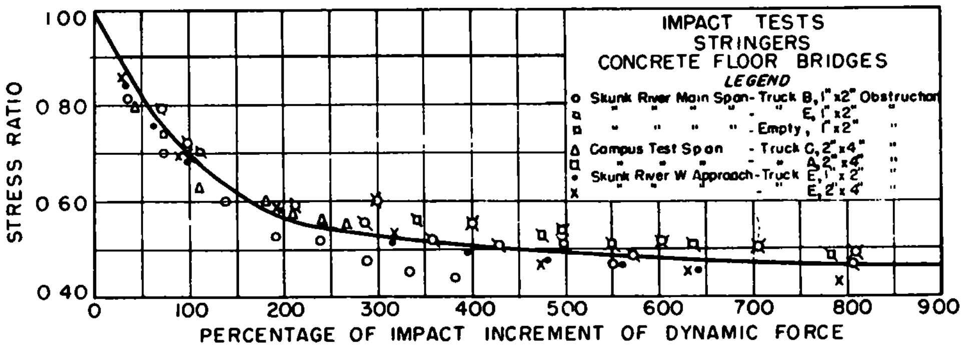

Figure 2. Relation of impact increment and stress ratio: Stringers of concrete-floor bridges.

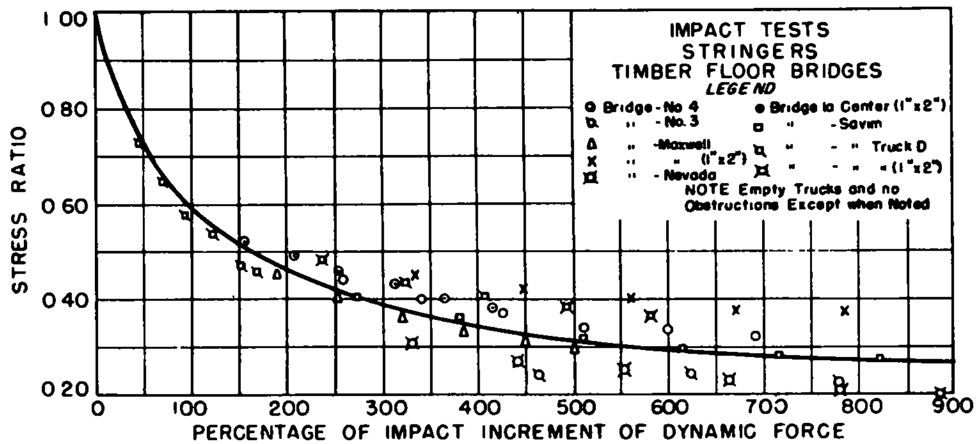

Figure 3. Relation of impact increment and stress ratio: Stringers of timber-floor bridge.

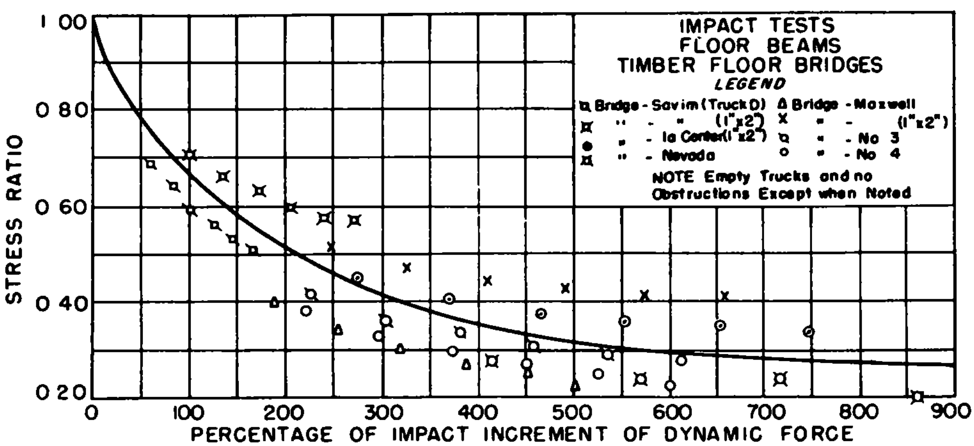

Figure 4. Relation of impact increment and stress ratio: Floor beams of timber-floor bridges.

The relation between a blow upon a pavement and upon a bridge floor (step 3) may be approximated by means described in Appendix D from which Table 1 has been prepared. The value of the data in the table for present trucks and tires is probably very small, but it is given to indicate a trend and to illustrate the possibilities of this type of analysis.

The relationship between dynamic force on bridge floors and simultaneous stresses (step 4) was given for the early trucks and structures by many diagrams (of which Fig. 1 is an example). From these diagrams a table was prepared to show the ratio between the *impact increment of dynamic force to the impact increment of simultaneous stresses* in the stringers and floor beams. These ratios were mostly above 2 1/2 in the stringers of three floors where concrete slabs were supported by steel stringers and more than four in the stringers and floor beams of six light truss bridges with timber floors on steel stringers.

Relationships between stress from a blow and the stress which would be developed by a static load which equals the force of a blow (step 5) were established as illustrated in Figures 2 to 4 by the use of the term *stress ratio,* which was defined as the ratio of the actual dynamic stress which was produced in a member to the stress that would have developed if a static load, equal in magnitude to the dynamic forces, had been applied to the place where the dynamic load was applied.

"The stress ratio diagram indicates a relation between the impact increment of dynamic force and the stress ratio for a variety of obstructions, loads and spans. The available information suggests that each relation is perfectly general in its field" (4). This statement was made in 1925 and was based upon researches conducted with vehicles with solid rubber tires. It needs checking for pneumatic-tired vehicles.

While the preceding five steps may give the basis for a rough indication of possible effect of present day equipment and structures, they are presented also as a basis for a question rather than as a definite guide. The question is: Would further researches under present or future conditions be justified, and would the results be useful in computing the impact upon and the stress in a bridge floor from any new trucks or special loads for which the force of a blow upon pavement would be established? Another question: Could useful information and statements be brought out such as that illustrated in the closing paragraph of Appendix B of Reference 4 ". . . impact is perceptibly greater when dual, rather than single tires are used and decidedly less when the rear load is carried on two axles (four wheels) rather than on the usual arrangement of one axle"?

Practically all of the available information on the behavior of bridge floors has been obtained in situations where the load was inadequate to develop stresses which even approached design values. All reported impacts, therefore, are too high to reflect the situation when overstressing is being approached, which, by the way, is the situation which reveals the true capacity of the floor to resist the unusual load without injury. Isn't it likely that the stresses, which might be developed by an occasional blow caused by an unusual obstruction, would be much less than usually suspected from observations upon the rigidity of an understressed floor?

No attempt has been made in this paper to bring together the effects of the force of a wheel blow and those of the speed of the truck. The variables appear to be too great to put into a mathematical statement; very few experimental results are available to show the relation between the force of a blow and speed of truck *on bridges.* The available reports on pavements, mostly by the U. S. Bureau of Public Roads, indicate the greatest force of the blow to be for a speed less than the maximum. A critical speed of around 15 mph. was indicated by much of the early equipment, while the latest information available suggests a heavier blow at speeds of 40 mph. than for greater speeds.

Static Effects

In the early studies the static stress in stringers and floor beams was determined as a reference point for impact. As the results were tabulated and plotted, they seemed to point to an interesting and perhaps valuable by-product in the form of the effect of the floor slab in distributing the load among the stringers. Figure 5 is one of seven similar plates which show the effect of one and of two trucks on each of four spans.

TABLE 2

TOTAL STRESS IN STRINGERS
ALL STRESSES ARE IN KIPS

Span	No of Trucks	Average of Observed Stresses	Computed stress	
			Stringers only	T-BM Action
West Panel	1	37.9	54.1	42.3
West Panel	2	75.2	104.0	82.7
West Approach	1	20.6	40.4	25.2
West Approach	2	42.9	78.2	48.8
Squaw Creek	2	44.2	113.5	44.5
Campus Test	1	8.5	20.5	13.0
Campus Test	2	17.5	41.0	26.0

The bottom figures at the left in Figure 5 give the sum of the observed stresses in the individual stringers in each of five positions of one truck. The bottom figures at the right indicate the sum. of the stresses as computed on the assumption that the stringers

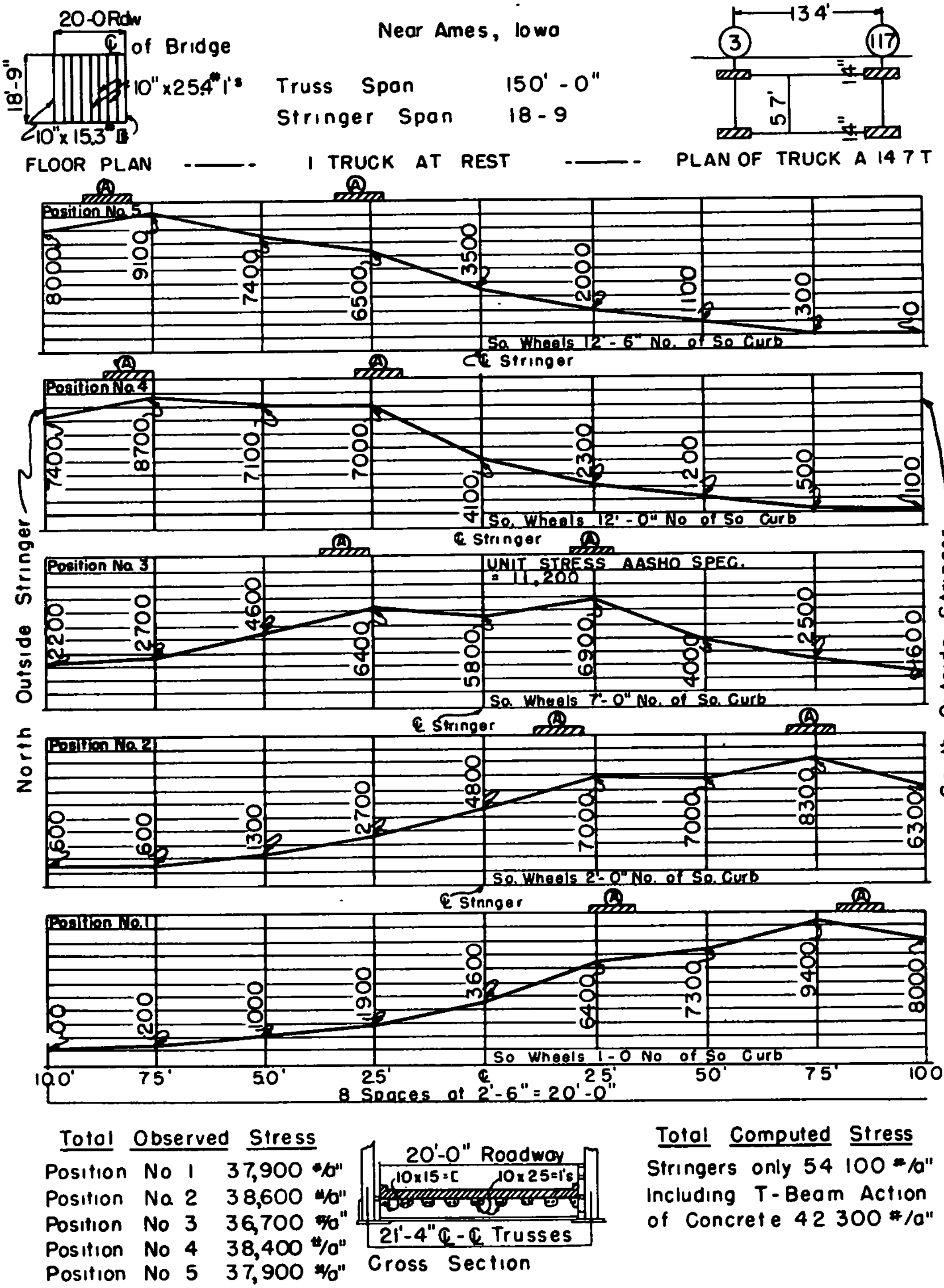

Figure 5. West panel of Skunk River Bridge: Distribution of static stress in stringers with one truck.

alone supported the entire load. They also show the sum of the stresses as computed on the basis of full composite action between the steel stringers and the reinforced floor slab.

Observed stresses from the seven plates under discussion were plotted by using the greatest stress in each stringer for any given span and load, regardless of the position of the load. A general tendency may be noticed for the greatest stressed stringer to be near the sides of the bridge for one truck and near the center when two trucks are on the span. This tendency is aggravated on the Skunk River Bridge where the outside stringers are lighter than the inside ones and cannot absorb as much load for a given deflection.

In each case the total observed stress is well below the stress which was computed under the assumption that the stringers alone supported the load. They are also somewhat below the computed stresses when full composite or T-beam action was assumed. This may indicate not only that fairly full composite action existed but also that additional restraint was supplied in some manner, possibly by an uncertain amount of fixity of the slab at the ends and from the stringer connections to the floor beams.

9-in. The bridge with original floor is considered typical of those of 30 years ago.

The Skunk River Bridge is on US 30, which was and still is an important transcontinental artery carrying heavy traffic, and the bridge has had an increasing volume of traffic over it in the 25 years between the tests. Therefore, it is of interest to see what effect age and traffic have had on the structure.

Although personnel, equipment, and budget for a comprehensive study were not available, a brief exploratory investigation was made in June 1948 by field measurements on three mornings between 4 and 8 o'clock. It was early recognized that high precision was impracticable, but sufficient checks were planned to catch mistakes.

The loads used in the 1948 study consisted of a heavy-duty truck trailer weighing 17,150 lb. carrying a 34,700 lb. track-laying tractor which could be moved on the truck to obtain various wheel concentrations. A skeleton outline of the truck is shown in Figure 6. The wheel concentrations are given in Table 3.

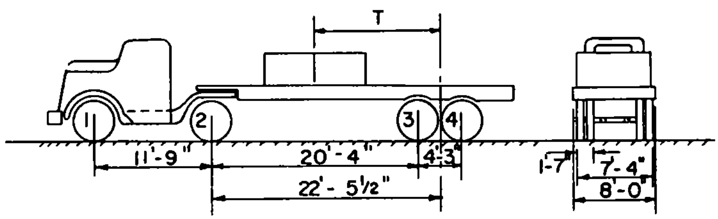

Figure 6. Skeleton outline of live load.

While the results were apparently accepted as representing the situation on each of the relatively new structures, much doubt has been expressed concerning the continuation of composite action after years of service, and therefore upon the advisability of recognizing such action in bridge specifications.

The 1948 Investigations

When one of the three structures which were the subjects of investigation in the 1920's, the Skunk River Bridge, was declared inadequate for modern traffic, largely because of clearances, and it was announced that the bridge was to be moved to another location on a secondary road, the possibility of securing limited information concerning the action after 25 years of further service was recognized.

The bridge consists of a 150-ft. through-riveted-steel span and a beam approach at each end with 20-ft. roadway There are nine lines of stringers. The stringers on the main span consist of seven 10-in. 25 4-lb. I-beams flanked on each side by one 10-in. 15.3-lb. channel. On the west approach span nine 15-in. I-beams at 42.9 lb. are used, for which the clear span is 32 ft. 8 in., and the distance center to center of end bearings is 34 ft.

The original reinforced concrete floor was 6 in. thick on steel stringers. In 1929 the floor thickness was increased by a 3-in. reinforced-concrete layer to

The length of the truck was such that only the wheels of the two rear axles were on a span. The position longitudinally was approximately that for maximum moment Transversely, the loads were in three different positions which were designated W, X, and Y. For Position W the north wheels were against the north curb For Position X the load was approximately on the center line, while for Position Y the south wheels were against the south curb. Each position thus had a definite notation (for example, III-X). Two or more separate placements were made for each load and position, for example, IV-W-1 and IV-W-2. Readings were taken both upon the west panel of the truss span and upon the west approach span as was done in 1922-25.

TABLE 3

AXLE LOADS FOR VARIOUS LIVE LOADS

Load	Load No	Distance T (Fig 6)	Axle-Weight in Pounds				
			1	2	3	4	Total
Truck Only	—	—	5,200	4,600	4,300	3,080	17,150
Truck	I	16 0	5,200	29,200	8,700	8,700	51,800
and	III	0	5,200	4,600	21,000	21,000	51,800
Tractor	IV	6 7	5,200	14,600	16,000	16,000	51,800
Tractor	—	—	—	—	—	—	34,700

Instruments. Strains were measured with two different types of electrical instruments attached to SR-4 strain gages Type A-1, and with direct reading extensometers. A Baldwin-Southwark portable strain indicator with switching unit for 22 connections and a Baldwin-Southwark-type of 6-channel oscillograph were connected with individual SR-4 gages. Five extensometers of 20-in. gage length with Last Word dials were connected to the beam flanges. Electric current was supplied for the electrical instruments by means of a motor-generator set on a truck.

Deflections were taken by means of eight Federal 0.001-in. dials working between the bottom flanges of the beams and a reference beam which was supported from the ground and independent of the staging for the operators.

Field Work. Individual readings for strain were taken at the centerline of each stringer on each side of the top of the bottom flange and, for a few stringers, on the bottom of the top flange.

The strain indicator, with attachments for 22 points at a time, provided the only means for heading all of the stringers. The six channels of the oscillograph and the five extensometers, distributed among the stringers and one floor beam served as checks upon the behavior of the strain indicator. The eight available deflection instruments were used in reading deflections for all loads

Computations The strains were translated into unit stresses by considering the modulus of elasticity as 29 million psi. Unit stresses were also computed from deflections under two extreme conditions, that the steel beam alone carried the entire load, and that the steel beam and concrete floor had full composite action. The correct stresses would naturally lie between these two extremes.

Results

Observed Stresses in Stringers. Individual stresses in the bottom flanges of the stringers were plotted on six plates A separate plate was used for each position for each load, but no attempt was made to distinguish each individual application of the load. All available results were plotted, but those from the strain indicator (the only instruments with connections to all stringers) were given the greatest weight in locating the points on the curves. The results from the other instruments are considered primarily as checks and as a means for appraising the general reliability of the work as a whole. Dotted lines connect simultaneous readings on opposite flanges of the same stringer. Two of these figures have been reproduced as Figures 7 and 8. The data on each curve are from a truck position near the curb on the north side of each span (Position W). The maximum ordinate and other data for each of the three positions on each of the two spans are given in Table 4. The stringer designations, A, B, etc., refer to the stringers, consecutively from the north side of the bridge.

The stresses which were computed from deflections under the assumption that the steel alone carries all the load are less than the ones which were computed from the strains, while the stresses which were based upon full composite action are greater than those from the strains. For the approach span the differences are not great, and the two sets of stresses from deflection may be considered as the limits of a band within which the actual stresses should lie (Fig 7). For the west panel the deflection stresses for steel alone are reasonably close to those from strains, while those computed under the assumption of full composite action (Fig. 8) are very much greater; in fact some of them fall beyond the limits of the sheet.

TABLE 4

RATIO OF LOAD CARRIED BY MAXIMUM STRESSED STRINGER TO THE TOTAL LOAD

Span	Load	Unit Stress in Bottom Flange of Stringers for 1-Truck Kips per Square Inch					
		1948			1925		
		Max	Total	Max /Total	Max	Total	Max /Total
West Panel	W	6 0	33 5	0 18	9 1	37 9	0 24
	X	6 2	36 4	0 17	6 9	36 7	0 19
	Y	6 5	34 7	0 19	9 4	37 9	0 25
West Appr	W	3 9	20 3	0 19	5 2	20 6	0 25
	X	3 2	21 0	0 15	3 4	20 5	0 17
	Y	3 5	18 8	0 19	5 0	21 4	0 23
	Spec n	—	—	0 21	—	—	0 21

Load to Maximum Stressed Stringer. In Table 4 is shown the ratio of the load which is transferred to the maximum stressed stringer to the total load. The ratio is based upon the unit stress in the one stringer as compared with the sum of the unit stresses in all the stringers. The stresses are taken from the curves (Figs. 7 and 8 and four similar ones). Similar results are also given in Table 4 for the 1925 studies on the same spans. For 1948, these ratios are 0.18 and 0.19 with the load on the side and 0 15 and 0.17 with the load on the centerline, while the value computed from the specifications is 0.21. In 1925 results (*3*, p. 51) show from 0 23 to 0.25 for the load on side and 0.17 and 0 19 for the load on the centerline. The ratio from the specification is the same as above or 0.21.

In 1925 the load to the maximum stressed stringer was greater than that allowed by the specification, while in 1948 it was less. The differences may be attributed to at least two causes, the use of a different

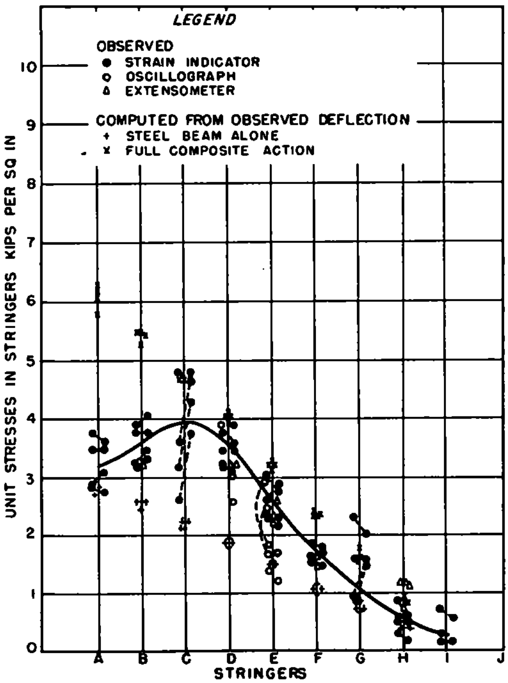

Figure 7. West-approach span, Load IV, Position W.

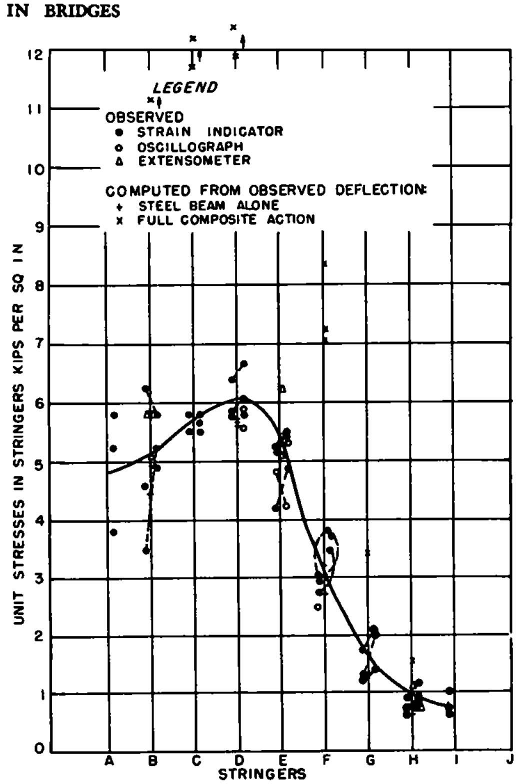

Figure 8. West panel, Load III, Position W.

type of truck, and the thickness of the concrete floor, which was 6 in. in 1925 and 9 in. in 1948.

The stringer spacing on the Skunk River Bridge, 2 ft. 6 in., is much less than for present practice. The present AASHO distribution of load is changed very slightly from the original which (as far as the author knows) first appeared in the 1923 specifications of the Iowa Highway Commission.

Computed Stress in Stringers. The computed stresses were based upon a distribution of load among the stringers according to the AASHO 1949 specifications. For the 1948 truck with two rear axles and four wheels on each axle, this distribution can be but a rough approximation at best.

TABLE 5

COMPARISON BETWEEN OBSERVED AND COMPUTED STRESSES

Type of Fiber Stress	Stringer in Approach Span	Stringer in West Panel, Main Span		
		Simple beam	Fixed ends	Probable end restraint
Computed				
Bottom flange, Steel only	9,800	15,600	6,100	11,500
Full composite action	4,900	5,400	2,100	4,000
Top fiber concrete, Full composite action	350	400	160	290
Horiz shear between steel and concrete, full composite action	90			300
Observed				
Bottom flange	3,500		6,200	
Top flange	±0		4,900	

A comparison between observed and computed unit stresses in stringers is made in Table 5. In the approach span the computed stress for steel alone is so much greater than the observed stress as to suggest definitely that the steel as a simple span did not carry the load. A very slight fixity exists, of course, from the fact that the supports are other than knife-edged. The greater part of the added resistance appears to come from composite action, which is the interaction between the steel beam and the concrete floor. (No mechanical bond was provided, but the concrete, 1 in. below the top of the steel, extended a little under the top flange.) The values for compression in concrete and horizontal shear between steel and concrete are within reasonable limits (even when dead load stresses are included) and do not exclude the possibility of full composite action. The fact that the computed stress in steel, even for full composite action, is so much greater than the observed stress (4,900 against 3,500) might be explained by the probability that the effective width of concrete, with the 9-in. thickness, was greater than the 30-in. (c-c stringers) which was used. The very small (0 ±) stress in the top flange suggests that the gravity axis was at about the top of the beam and that all the steel was in tension.

The computations for stresses in the stringers of the west panel were made under three assumptions: (1) simple beam, (2) beam with fixed ends, and (3) beam with end resistance which was computed from the reinforcement in the concrete floor and from the standard girder to floor beam connections. The first two represent extreme conditions which might be approached but could not be reached. For the third condition the steel was considered available up to the elastic limit (assumed conservatively as 30,000 psi.) with no help from the concrete. This might be a fairly reasonable value and will be used in the discussion.

In the west panel, although the stress in steel alone for simple span is higher than the observed stress, that for full composite action is less, the shear is too great, and the observed stress in the top flange approaches that in the bottom one. Therefore, any composite action must be small and may be mostly friction with the bond pretty well destroyed. It seems then that the composite action is small and that, therefore, the end restraint of the reinforced concrete floor contributes a partial continuity which is effective in reducing the stresses or, in other words, in increasing the capacity of the floor.

Floor Beam. The few observations which were made on one floor beam, when compared with computed values, suggest good composite action. The fact that the concrete floor is poured over the stringer flanges and the stringers are riveted to the floor beam apparently provides joint action between steel and concrete, which is independent of bond.

Deductions and Resulting Questions

This work has met with the same limitations which existed in many previous investigations—that of being unable to secure, or move over the highways, a load of sufficient weight to develop even full unit stresses in modern structures.

Within the limits of the available live load the present results point rather definitely to the fact that the stress in the steel stringers in each span was decidedly less than would have been developed in the steel action alone as simple beams. Although no mechanical bond was provided, the reinforced concrete floor evidently contributed in some manner to the value of the combined concrete floor and the steel stringers.

In the approach span the evidence points to definite (if not full) composite action, even though the span had been in use for twenty-eight years under heavy traffic. In the west panel the measured strains and deflections and the cracking of the concrete around the flanges of the stringers indicate but very little direct action between the concrete and the steel. Yet the low stresses in the stringers suggest that they are getting help from some source. The resisting moment from the steel reinforcement in the floor slab and from the stringer to floor beam connections could provide about the necessary help.

Although these deductions may reflect correctly the small unit stresses which were developed by the available live load, no information is at hand for extending the results to fully loaded structures.

Though no general conclusion should be drawn from an exploratory research as brief as the one under consideration, the present work might serve as a basis for possible further research.

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* In cooperation with the U S Bureau of Public Roads and the Iowa State Highway Commission