LIVE-LOAD STRESS MEASUREMENTS ON FORT LOUDON BRIDGE FINAL REPORT[1]

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SYNOPSIS

EIGHT VEHICLES, ranging from a two-axle 27,830-lb. truck to a five-axle 150,000-lb. tank transporter were used in testing a low truss bridge of 110 ft. 8 in. span at Fort Loudon, Pennsylvania. During each passage of a test vehicle over the structure, 24 simultaneous strains were recorded with electromagnetic gages. In this report, figures are presented for typical truss members, showing static stresses, impact stresses and secondary stresses. Data are presented on vertical and horizontal truss deflections and stresses in the floor system are discussed. Vehicle speeds varied from 5 to 50 mph.

While these data apply only to the particular bridge tested, it is believed that the results will have some bearing on the design of highway bridges in general. Tests of this kind should lead to a better understanding of the behavior of bridges under moving live loads.

● DURING the summer of 1948, a comparative study was made in Pennsylvania to determine the fuel and time consumed by selected, heavy test-vehicles in traveling a relatively good and poor route over rough terrain with various gross vehicle weights. This was known as the Pennsylvania Vehicle Performance Study, and the routes selected were between Carlisle, New Stanton, and Greensburg, Pennsylvania.

With suitable trucks available, there was a desire to test a typical highway bridge under dynamic loading. This was made possible by the coöperation of the Association of American Railroads. The bridge-test project was sponsored jointly by the Pennsylvania Department of Highways and the Bureau of Public Roads. Considerable thought was given to the selection of the structure to be tested. It was desirable that it be a truss bridge, so that the action of the truss members as well as those of the stringers and floor beams under live load could be studied independently. Further, in order that the test vehicles might be available at all times, it was necessary that the structure be located within a convenient distance of the base of operations of the performance study at Carlisle. These considerations led to the selection of a low truss bridge on US 30 at Fort Loudon, Pennsylvania.

The Association of American Railroads conducted the tests during the fall of 1948, furnishing not only the necessary equipment but also supplying the services of an experienced field party.

[1] A preliminary report appears in Vol. 29 (1949), page 83, of the PROCEEDINGS.

GENERAL

The tested structure, which was built in 1924, has a span length of 110 ft. 8 in. between centers of bearing and a roadway width of 23 ft. 0 in. between curbs. It was designed for H20 loading, with a basic unit stress of 16,000 psi. The trusses are of the Warren type, with a panel length of 13 ft. 10 in. and a maximum depth of 14 ft. 0 in. between centers of chords at midspan. The 7-in. reinforced concrete floor slab is supported directly on 7 lines of 12-in. 40.8-lb. I-beam stringers, spaced 3 ft. 10 in. on centers. The bottom of the floor slab is flush with the top surface of the stringers, so that there is no encasement of the stringer flanges. Over the floor slab is a concrete wearing surface of 4½ in. maximum thickness. All floor beams are 24-in. 121-lb. Bethlehem girder beams. The bottom lateral system consists of double cross-braces in each panel. It lies in the plane of the bottom of the bottom chords and of the floor beams. The laterals are connected together at their intersections on the center line of roadway, but they are not connected to the stringers. All laterals are 3-in. by 3-in. by $\frac{5}{16}$-in. single angles. A view of the bridge, looking west, is shown in Figure 1 (a), and a typical section through the roadway appears in Figure 2.

ALIGNMENT AND GRADE OF ROADWAY

The grade and alignment in the vicinity of the bridge are shown in Figure 3. The roadway is of bituminous macadam, 20 ft. wide, and in fairly good condition. To the west of the structure the sight distance is short, because

of the grade and curvature. This, and the fact that the site is within a village, tended to limit the maximum speeds attained during the tests.

TESTING EQUIPMENT

The gages used were of the electromagnetic type, with a gage length of two inches. The principle by which strains are measured by the electromagnetic gage is based on the fact that a magnetic circuit, in which the lines of magnetic force must travel through both a magnetic material and an air gap, has the greater part of its reluctance in that part of its path which is through the air gap, even if the gap is very short. Therefore, a small change in the length of the air gap will have a relatively large effect on the total reluctance of the circuit and on the current carried in the winding of the magnet. For structural steel, with a modulus of elasticity of 30,000,00,0 psi. a change in length of only 0.001 in. between the gage points, which are 2 in. apart, represents a change in stress of 15,000 psi. for stresses below the proportional limit. The testing equipment and its operation are fully described in *Proceedings*, American Railway Engineering Association, Vol. 42, 1941, page 402. The gages are rugged, operate reliably under all weather conditions, and can readily be moved from one position to another. For these reasons, they are particularly adapted to the testing of bridges and other outdoor structures under dynamic loads.

The traces of 12 gages were recorded by one oscillograph on one film, and since in these tests, two oscillographs were used, 24 simultaneous strains generally were recorded for each passage of a test vehicle over the bridge. Each gage was calibrated before the start of the tests and again after all the tests had been completed. The results of the two calibrations were very close, and therefore, average values obtained for each gage were used in determining the stress factors. The stress factor of a gage is the amount of unit stress in steel for 1 in. of deflection of the light trace on the film for that particular gage. In general, the stress factors of the gages were in the neighborhood of 10,000 psi. per in. deflection of the light trace.

On the photographic films, or oscillograms, on which the traces of the gages are recorded,

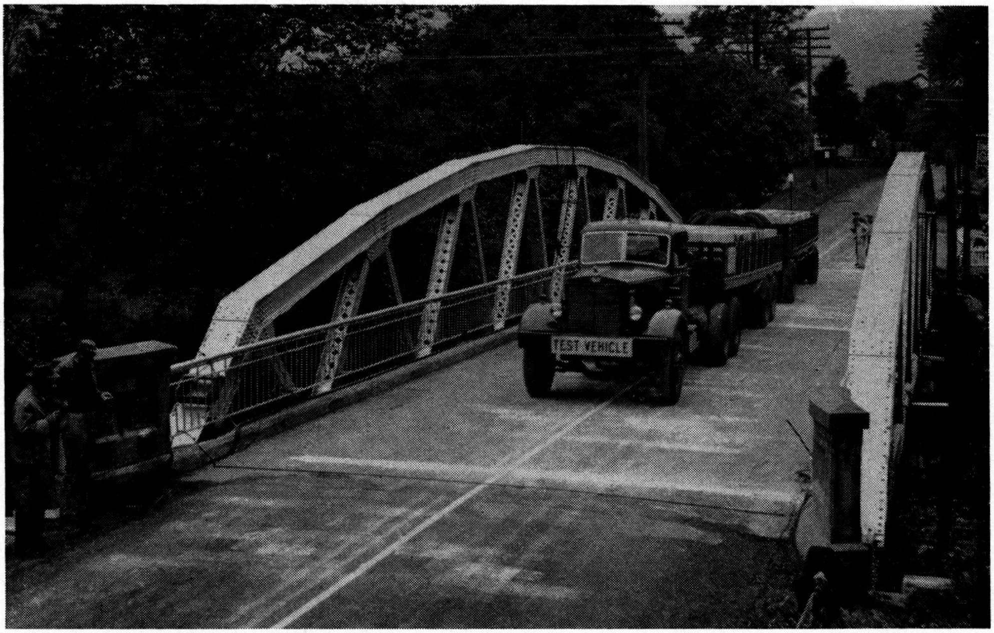


Figure 1(a). View of bridge; the test vehicle is Truck 7.

the abscissas, or horizontal distances, represent time, while the ordinates, or vertical distances above or below the base line of each trace represent unit stress. Time is measured from left to right, and intervals of 0.01 seconds are marked off over the entire length of the oscillogram. In the Fort Loudon tests, pneumatic-tube wheel trips, similar to those used in highway traffic studies, were placed across the roadway at each end of the bridge, directly over the centers of bearing. The distance between these tubes was therefore 110 ft. 8 in., the span length of the bridge. The tubes were electrically connected with the marker units in each oscillograph. By compressing the tubes, characteristic marks were recorded on the oscillograms. Thus, under traffic, each vehicle axle was recorded at the instant it entered and left the structure. From the oscillograms vehicle speeds may easily be determined by noting the elapsed time for any axle to travel the known distance between the wheel markers. With a vehicle speed known, the position of any axle at any time, such as the time of maximum stress in any member, may readily be ascertained. A typical oscillogram is shown in Figure 6. It is the record of a five-axle vehicle passing west bound over the structure at a speed of 7.6 mph.

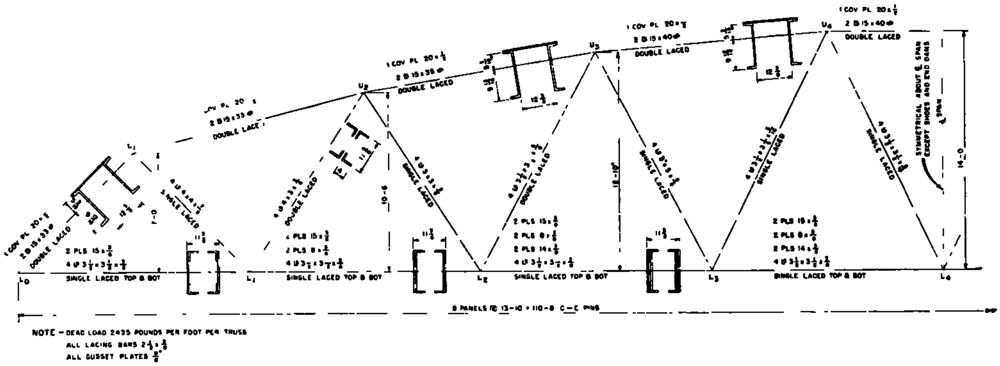


Figure 1(b). Make-up of truss members.

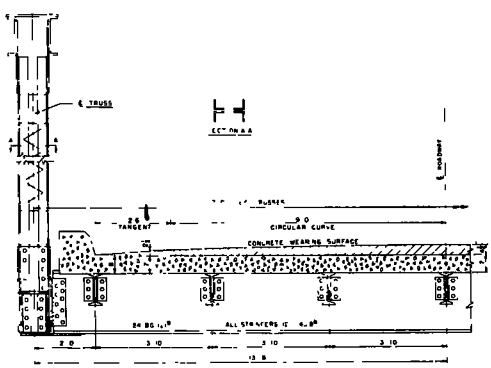


Figure 2. Typical cross-section.

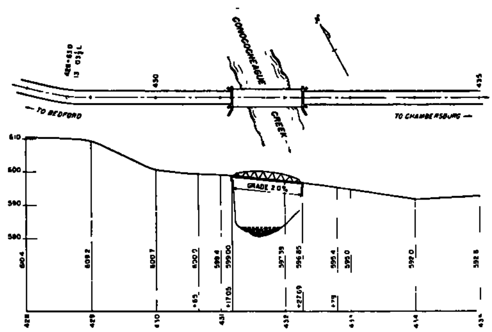


Figure 3. Roadway alignment and grade.

The locations of the gages on typical members of the structure are shown in Figures 4(a) and 4(b).

TEST VEHICLES

Eight test vehicles were used in the work, ranging in weight from a two-axle 27,830-lb. truck to a 150,000-lb. tank transporter. These test vehicles are diagrammed in Figure 7. The Tank Transporter A and the Tank B were supplied by the U. S. Army. The weight of the transporter was 150,000 lbs. when carrying the tank. The trucks, other than the transporter and the tank, were loaned

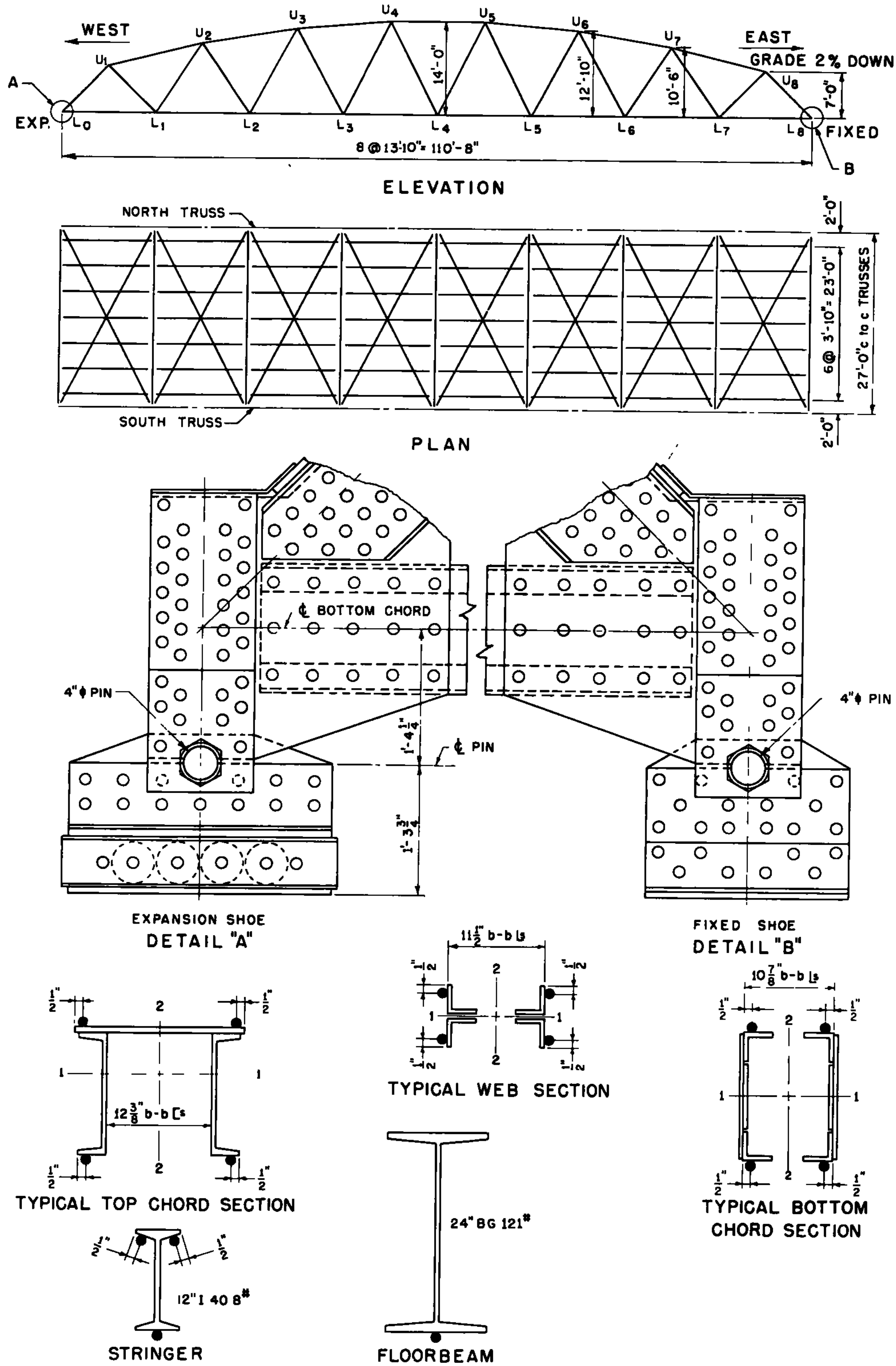


Figure 4(a). Bearing details and location of gages.

from the Vehicle Performance Study. They were loaded with precast concrete blocks weighing approximately 1,000 lbs. each. All axle loads were weighed before any test runs were made.

Truck 7A was used in every series of the entire test program, while each of the other vehicles was used to only a limited extent.

members. These cover groups of members in both trusses, near the ends, the quarter points and the center of the span. Series VII to IX inclusive covered secondary stresses in the truss members. In Series VI, the gages were placed on the flanges of the stringers and floor beams of the east center panel of the span in order to observe the behavior of the

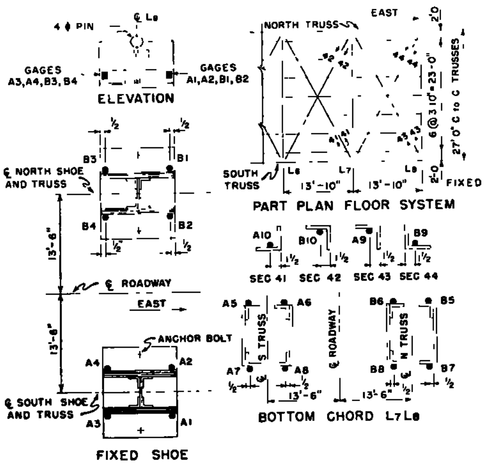


Figure 4(b). Location of gages.

Figure 5. Installing gages on truss member.

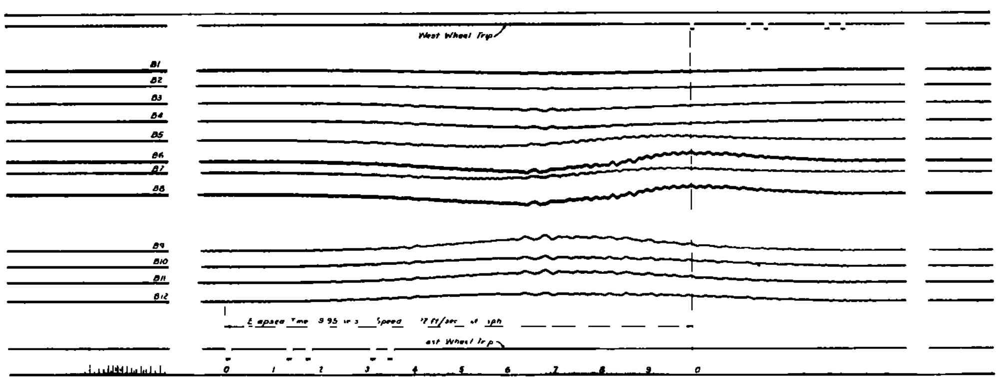


Figure 6. Typical Oscillogram.

The test program was planned to embrace: (1) static stresses; (2) impact stresses; (3) secondary stresses; (4) stresses in stringers and floor beams; (5) truss deflections, vertical and horizontal; and (6) longitudinal stresses.

In order to cover these subjects, eleven distinct series of tests were planned. Series I to V inclusive, and Series X were designed to study static stresses and impact in the truss

floor system under live load. An attempt was made in Series XI to measure longitudinal forces from acceleration and from braking in the bottom chords, the bottom lateral system and in the shoes at the fixed end of the bridge.

TESTING PROCEDURE

The procedure for testing highway bridges is similar to that for railroad bridges. In each

series, the test vehicle makes one or more runs in each direction across the structure at a speed of 5 mph. At this speed the traces made by the gages on the oscillograms are smooth curves and the stresses recorded are practically static stresses. After the 5-mph. runs have been made, the speeds are increased by 5 mph. increments until the maximum speed has been attained. In this manner, critical speeds at which impact is maximum may be determined.

There is, however, one important difference between railroad and highway bridge-testing. On railraod structures, the live load is applied on rails, whose lateral position is fixed. If the track, or tracks, are not symmetrically located on the structure, the correction factor for the eccentricity is simply a constant. On highway bridges small accidental eccentricities are unavoidable. In these tests it was therefore necessary to measure and record the lateral position of the vehicle on the structure after each passage, in order that any eccenticity of load might be taken into consideration in interpreting the data.

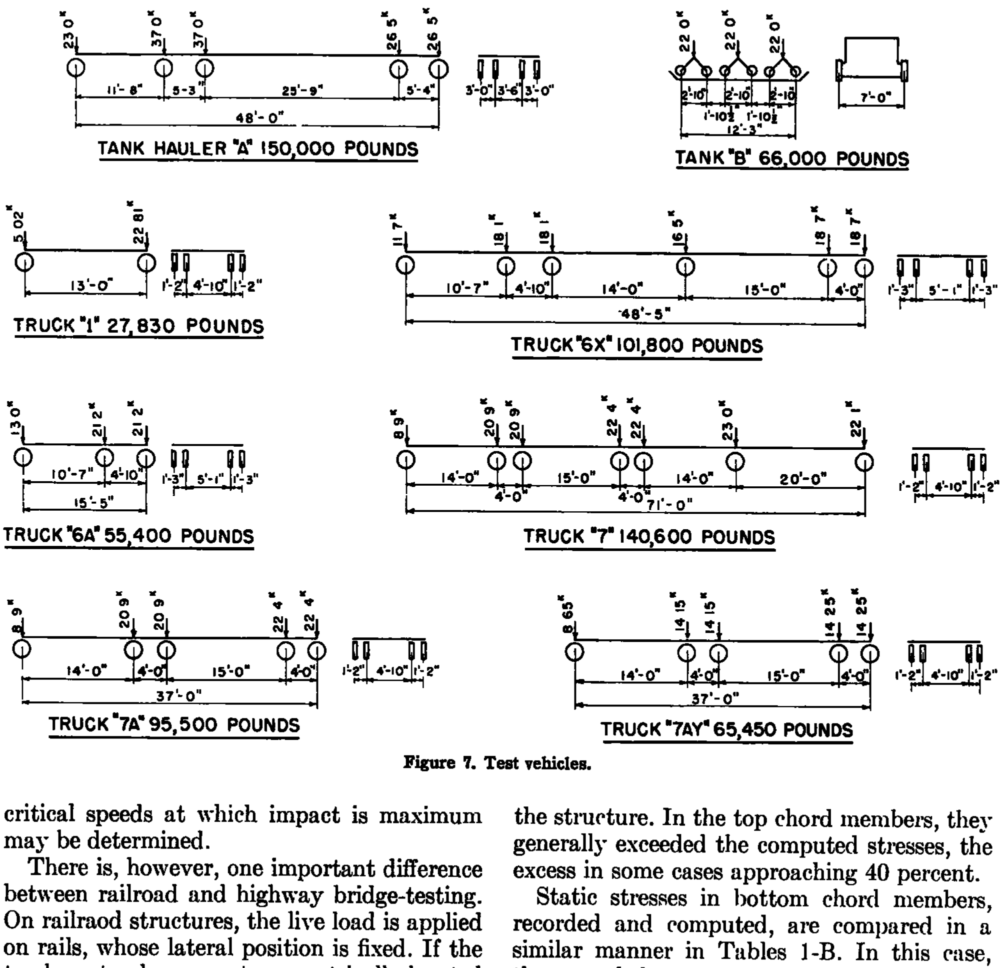


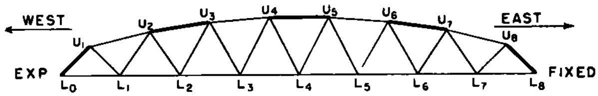
Figure 7. Test vehicles.

STATIC STRESSES

The recorded static stresses in the top chord members of the trusses are compared with the computed values in Table 1-A. These static stresses are the averages of all the 5-mph. runs made by any one vehicle over the structure. In the top chord members, they generally exceeded the computed stresses, the excess in some cases approaching 40 percent.

Static stresses in bottom chord members, recorded and computed, are compared in a similar manner in Tables 1-B. In this case, the recorded stresses were considerably less than the computed values. In members L_7L_8 of both trusses, the recorded stresses were compressive, while the computed values are tensile. Actually, the gages in these members recorded combinations of compression and bending about the horizontal axes. The stresses in the top fibers were practically zero,

TABLE 1-A
STATIC STRESSES IN TOP-CHORD MEMBERS

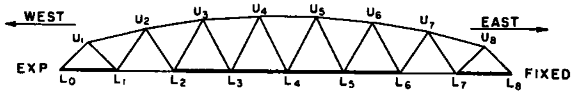


Comparison of Recorded and Computed Static Stresses
110'-8" LOW TRUSS SPAN
TOP CHORD MEMBERS

STATIC STRESSES IN THOUSANDS OF POUNDS PER SQUARE INCH

Series	Member	Vehicle	Runs	Recorded Stress-Av.	Computed Stress	Recorded/Computed
I	L_0U_1	A	1-West	-2.29	-2.37	0.966
			1-East	-2.33	-2.31	1.009
		B	4-West	-1.32	-1.28	1.031
			4-East	-1.26	-1.28	0.984
		1	4-West	-0.58	-0.51	1.137
			4-East	-0.55	-0.56	0.982
		6A	3-West	-1.16	-1.03	1.126
			3-East	-1.11	-1.08	1.028
		6X	3-West	-1.74	-1.55	1.123
			3-East	-1.69	-1.63	1.037
		7	2-West	-2.32	-2.06	1.126
			2-East	-2.14	-1.89	1.132
		7A	3-West	-1.81	-1.58	1.146
			3-East	-1.89	-1.70	1.112
		7AY	3-West	-1.45	-1.05	1.381
			3-East	-1.41	-1.15	1.226
II	U_2U_3	7A	3-West	-2.73	-2.33	1.172
			3-East	-2.80	-2.37	1.182
III	U_4U_5	A	2-West	-3.14	-3.33	0.943
			1-East	-2.98	-3.33	0.895
		B	3-West	-1.87	-1.82	1.027
			3-East	-1.96	-1.82	1.077
		1	4-West	-0.92	-0.79	1.165
			4-East	-0.94	-0.79	1.190
		6A	3-West	-1.74	-1.52	1.145
			3-East	-1.70	-1.52	1.118
		6X	2-West	-2.62	-2.17	1.210
			2-East	-2.52	-2.17	1.161
		7	2-West	-3.12	-2.91	1.072
			2-East	-3.42	-2.91	1.175
		7A	2-West	-2.75	-2.29	1.201
			2-East	-2.60	-2.29	1.135
		7AY	3-West	-2.14	-1.57	1.363
			3-East	-2.06	-1.57	1.312
X	U_4U_5	7A	6-West	-2.99	-2.29	1.306
			6-East	-2.86	-2.29	1.249
IV	U_6U_7	7A	3-West	-2.63	-2.37	1.110
			3-East	-2.58	-2.33	1.107
V	U_8L_8	7A	3-West	-1.76.	-1.70	1.035
			3-East	-1.71	-1.58	1.082

TABLE 1-B
STATIC STRESSES IN BOTTOM-CHORD MEMBERS



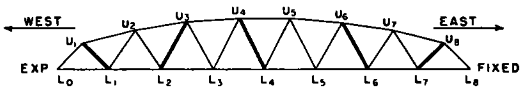
Comparison of Recorded and Computed Static Stresses
110'-8" LOW TRUSS SPAN
BOTTOM CHORD MEMBERS

STATIC STRESSES IN THOUSANDS OF POUNDS PER SQUARE INCH

Series	Member	Vehicle	Runs	Recorded Stress-Av	Computed Stress	Recorded / Computed
I	L_0L_1	A	1-West	+1.44	+2 34	0.616
			1-East	+1 14	+2 28	0 500
		B	4-West	+0.74	+1 26	0 587
			4-East	+0.58	+1 26	0 460
		1	4-West	+0 24	+0 51	0.471
			4-East	+0.16	+0.56	0.286
		6A	3-West	+0 77	+1.02	0.755
			3-East	+0.65	+1.07	0 607
		6X	3-West	+1 07	+1.54	0 695
			3-East	+1 03	+1 62	0.636
		7	2-West	+1 34	+2 04	0 657
			2-East	+1 19	+1 87	0 636
		7A	3-West	+1.12	+1 56	0 718
			3-East	+1 18	+1 68	0 702
		7AY	3-West	+0 85	+1 04	0 817
			3-East	+0.83	+1 13	0.734
II	L_2L_3	7A	3-West	+1 72	+1 91	0 900
			3-East	+1.80	+1 96	0.918
X	L_3L_4	7A	6-West	+1.88	+2 06	0.913
			6-East	+1 86	+2 07	0 899
III	L_4L_5	A	2-West	+1.91	+2.88	0 664
			1-East	+1.86	+2 93	0 635
		B	3-West	+1.16	+1 56	0.744
			3-East	+1.13	+1.56	0.724
		1	4-West	+0.46	+0 65	0 708
			4-East	+0.47	+0.66	0 712
		6A	3-West	+0 98	+1 29	0 760
			3-East	+1 02	+1 30	0.785
		6X	2-West	+1.55	+1 94	0 799
			2-East	+1.46	+1.98	0.738
		7	2-West	+1.84	+2 56	0.719
			2-East	+1 94	+2 59	0 749
		7A	2-West	+1.66	+2 07	0.802
			2-East	+1 62	+2 06	0.786
		7AY	3-West	+1 18	+1.40	0.843
			3-East	+1.12	+1 39	0 806
X	L_4L_5	7A	6-West	+1.76	+2.07	0 850
			6-East	+1.67	+2.06	0 811
IV	L_5L_6	7A	3-West	+1 33	+1.96	0 679
			3-East	+1.27	+1.91	0.665
V	L_7L_8	7A	3-West	-0 56	+1 68	-0 345
			3-East	-0.61	+1.56	-0 391

TABLE 1 C

STATIC STRESSES IN WEB MEMBERS



Comparison of Recorded and Computed Static Stresses 110'-8" LOW TRUSS SPAN WEB MEMBERS

STATIC STRESSES IN THOUSANDS OF POUNDS PER SQUARE INCH

Series	Member	Vehicle	Runs	Recorded Stress - Av	Computed Stress	Recorded / Computed
I	$U_1 L_1$	A	1-West	+3 29	+3 20	1 028
			1-East	+3 00	+3 12	0.969
		B	4-West	+1 98	+1 .73	1 144
			4-East	+1 98	+1 73	1 144
		1	4-West	+0 82	+0 70	1 172
			4-East	+0 84	+0 76	1 105
		6A	3-West	+1 70	+1 39	1.223
			3-East	+1 75	+1 46	1 190
		6X	3-West	+2 49	+2 10	1 186
			3-East	+2 53	+2 20	1 150
		7	2-West	+3 04	+2 79	1 090
			2-East	+2 95	+2 56	1 152
		7A	3-West	+2 45	+2 13	1 150
			3-East	+2 83	+2 29	1 236
		7AY	3-West	+1 77	+1 42	1 246
			3-East	+1 90	+1 .55	1.226
II	$L_2 U_3$	7A	3-West	-1 75	-1 59	1 101
			3-East	-1 93	-1 94	0 995
	$L_2 U_3$ (Rev)	7A	3-West	+1 85	+1 90	0 974
			3-East	+1 53	+1 62	0 944
III	$U_4 L_4$	A	2-West	+2 76	+2 92	0 945
			1-East	+2 45	+2 75	0 891
	$U_4 L_4$ (Rev)	A	2-West	-1 32	-1 61	0 820
			1-East	-1 53	-1 82	0 841
	$U_4 L_4$	B	3-West	+2 29	+1 96	1 168
			3-East	+2 20	+1 96	1 122
	$U_4 L_4$ (Rev)	B	3-West	-1 45	-1 43	1 014
			3-East	-1 38	-1 43	0 965
	$U_4 L_4$	1	3-West	+0 91	+0 82	1 110
			3-East	+1 03	+0 89	1 154
	$U_4 L_4$ (Rev)	1	4-West	-0 66	-0 66	1 000
			4-East	-0 56	-0 52	1 077
	$U_4 L_4$	6A	3-West	+1 72	+1 51	1 139
			3-East	+1 89	+1 68	1 125
	$U_4 L_4$ (Rev)	6A	3-West	-1 11	-1 23	0.902
			3-East	-1 07	-1 08	0 991
	$U_4 L_4$	6X	2-West	+2 10	+1 .84	1 142
			2-East	+2.05	+2 05	1 000
	$U_4 L_4$ (Rev)	6X	2-West	-1 39	-1 27	1 094
			2-East	-1 25.	-1 06	1.179
	$U_4 L_4$	7	2-West	+2 50	+2 33	1 073
			2-East	+2 16	+1 89	1 143
	$U_4 L_4$ (Rev)	7	2-West	-1 00	-1 04	0 962
			2-East	-1 40	-1 35	1 037
	$U_4 L_4$	7A	2-West	+2 35	+2 06	1 141
			2-East	+2 63	+2 40	1 096
	$U_4 L_4$ (Rev)	7A	2-West	-1 68	-1 65	1 018
			2-East	-1 32	-1 28	1 031
	$U_4 L_4$	7AY	3-West	+1 .60	+1 30	1.231
			3-East	+1 73	+1 60	1.081
	$U_4 L_4$ (Rev)	7AY	3-West	-1 17	-1 08	1 083
			3-East	-1 00	-0 75	1 333
IV	$U_6 L_6$	7A	3-West	-2 08	-1 94	1 072
			3-East	-1 95	-1 59	1 226
	$U_6 L_6$ (Rev)	7A	3-West	+1 36	+1 62	0 840
			3-East	+1 61	+1 90	0 847
V	$L_7 U_8$	7A	3-West	+2 60	+2 29	1 135
			3-East	+2.46	+2 13	1 155

so that the compressive stresses in the bottom fibers were about twice the average stresses in these members.

In Table 1-C, comparisons are made of the recorded and the computed static stresses in the web members of the trusses. In general, the recorded stresses were greater than the computed values. Reversals of stress occurred in several members during the passage of a vehicle over the span. It is of interest to note that the ratios of the recorded to the computed static stresses for the reversals did not differ greatly in value from those for the direct stresses.

The variations between the recorded and computed static stresses appear to be due to at least two causes. Without doubt, the stringers and the floor slab carried some direct stress, and in doing so, relieved the bottom chords of a portion of their load. However, since the elevation of the floor system is above that of the bottom chords, the effective truss depth was decreased, resulting in increased stresses in the top chords. The bending stresses which were recorded in members L_7L_8 suggest that there was a considerable amount of friction about the 4-in. round pins immediately below the panel points L_8. The location of these pins is shown in Figure 4(a).

TOTAL IMPACT

The additional force produced by a moving load over that produced by the same force in a state of rest is called impact. The total impact in a structure or in a particular member is the result of several individual effects, among them the effects of roll, of speed and of the pavement roughness. These do not necessarily reach their maximum values at the same vehicle speed, and the total impact therefore becomes a maximum when the instantaneous sum is a maximum. Some of the individual effects may be evaluated separately, but in this report only the roll effect and the speed effect will receive separate treatment.

In the studies of impact, the gages on the various members were placed in sections midway between the panel points. Total impact in the chords of the trusses are shown in Figure 10. For these members there appears to be a critical speed of about 30 mph. at which the total impact is the greatest. The maximum values recorded were about 40 percent of the recorded static values. In a similar manner, the total impacts in the web members are shown in Figure 11. There appear to be critical speeds of about 27 mph. and 42 mph. at which the total impacts reach their maximum values. These were about 50 percent of the recorded static stresses.

ROLL EFFECT

The roll effect results from the oscillation of the spring-borne weight of a vehicle about

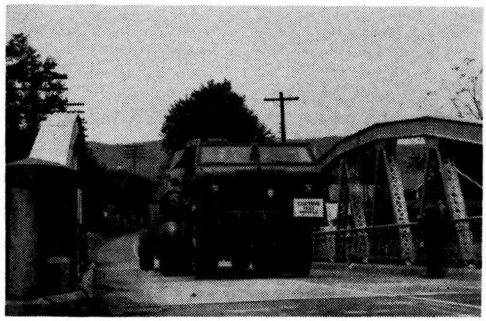


Figure 8. The tank transporter, Vehicle A.

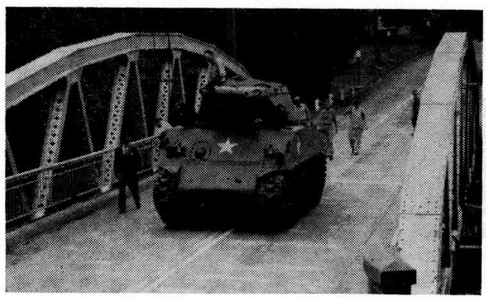


Figure 9. The tank, Vehicle B.

a longitudinal horizontal axis, as the vehicle passes over the structure. The roll of the spring-borne weight increases the wheel loads on one side of the vehicle and decreases them correspondingly on the other side. Roll effects are shown in Figures 12 and 13, for the chord members and the web members respectively. The maximum roll effects recorded in the truss members were about 16 percent of the static stresses. Since the transverse distance between the centers of the trusses is 27.0 ft. and the lengths of the axles between the centers

of the wheel treads about 6.0 ft., the roll of the vehicles themselves actually must have been 27.0 ÷ 6.0, or 4.5, times 16 percent, or 72 percent. The wheel loads, therefore, would have been increased or decreased about 72 percent of their static values under the effect of roll.

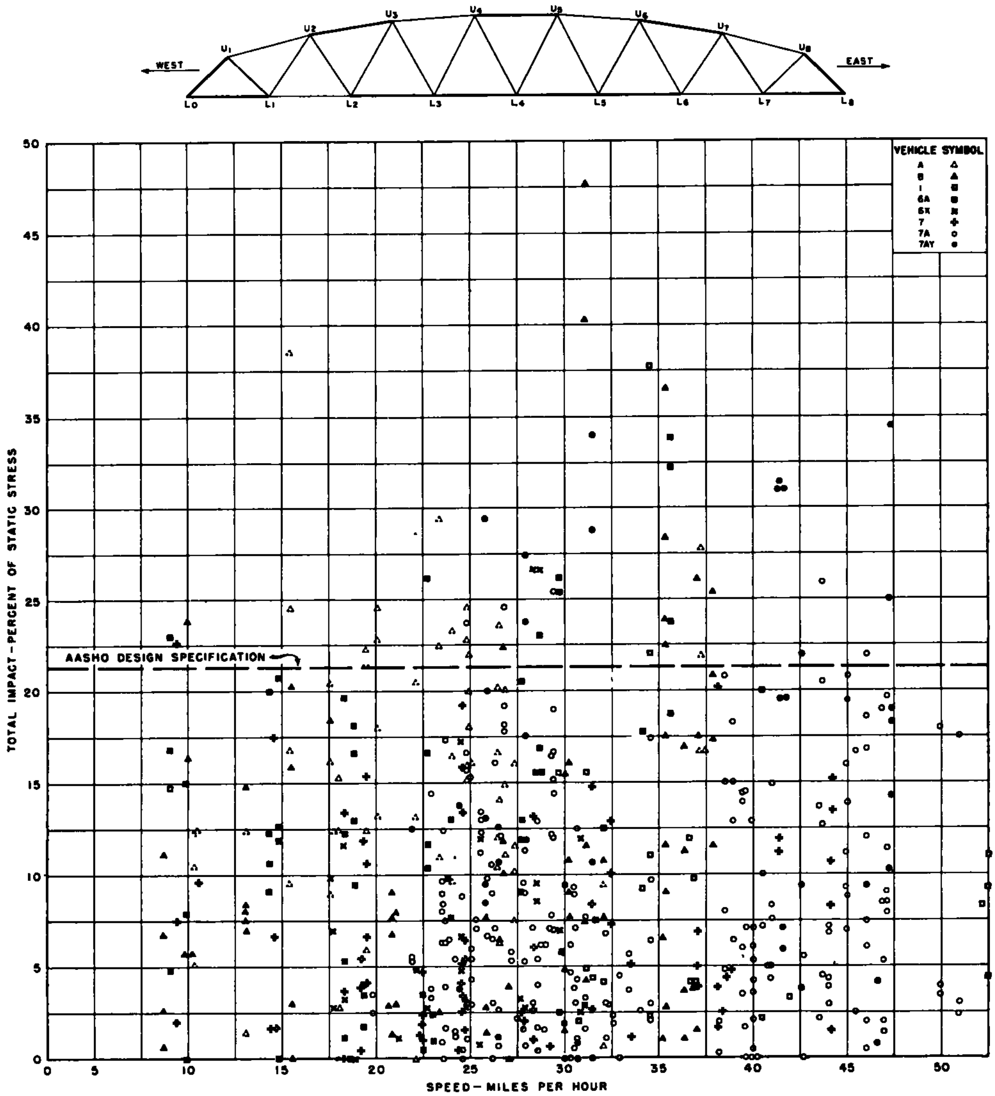


Figure 10. Total impact in chord members.

SPEED EFFECT

The speed effect is probably the result of the centrifugal force developed by the truck in following the path of the deflected span and of the increased weight on the span resulting from the downward acceleration of the spring-borne weight of the truck. Speed effects are shown in Figure 14 for the chord members of the trusses and in Figure 15 for the web members. They appear to have reached their maximum values at a speed of

31 mph. in the chords and of about 40 mph. in the web members.

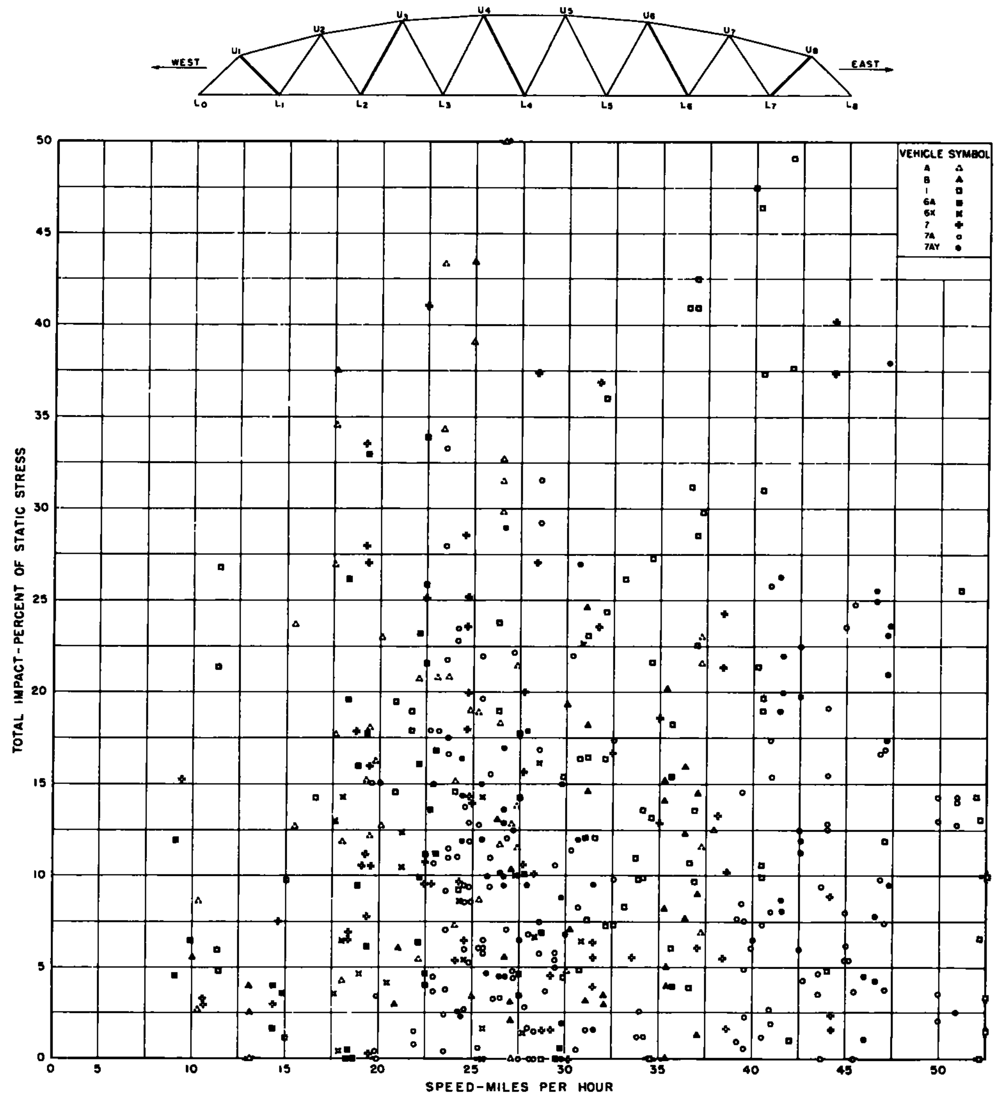


Figure 11. Total impact in web members.

SECONDARY STRESSES

Data on secondary stresses in the truss members are presented in Figure 16. These stresses were measured only in the south, or downstream truss. Truck 7A was used exclusively for this purpose.

Typical top chord, bottom chord and web members were selected for the study. The gages were installed in sections at both ends of the members, immediately adjacent to the gusset plates. In each section, four gages were used, one at each corner of the member, as shown in Figure 4(a). With the gages arranged in this manner, bending stresses were recorded about both the horizontal and vertical axes of the members. While the bending

stresses about the vertical axes were not great, they did contribute materially to the magnitudes of the total secondary stresses about static stresses. This friction appears to be responsible also for the high secondary stresses in members L_7L_8 and U_8L_8.

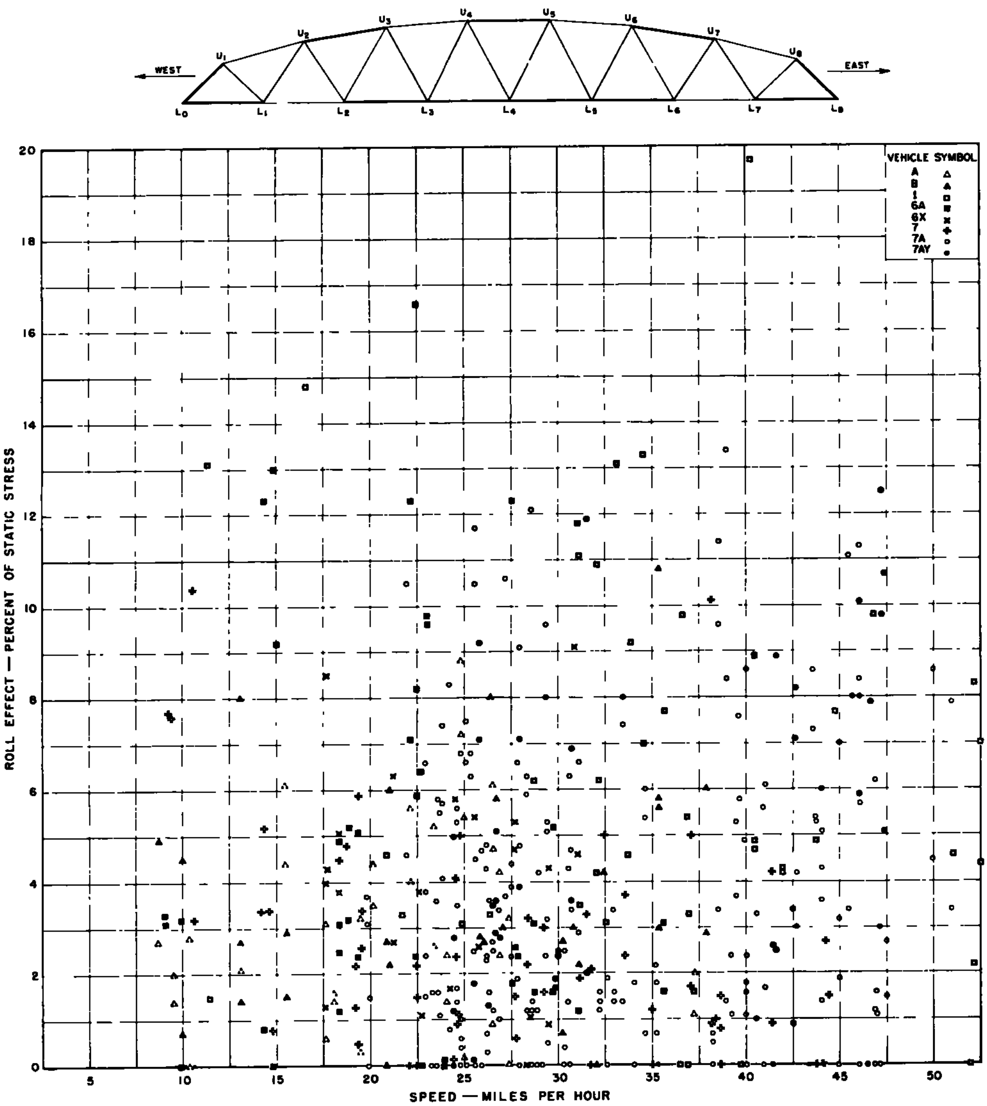


Figure 12. Roll effect in chord members.

both axes. In general, the recorded secondary stresses were moderate, except in the members which are adjacent to the panel point L_8. The excessive friction about the 4-in. pin in the fixed shoe just below the joint L_8 has already been mentioned in connection with

TRUSS DEFLECTIONS

Deflectometers were installed on both trusses at the center of the span. Vertical deflections were recorded at the lower chord panel points L_4, while horizontal deflections were read in the top chords at a section midway between panel points U_4 and U_5.

Vertical deflections in both trusses were measured under the action of Trucks 6A, 7 and 7A at speeds ranging from 5 mph. to 50 mph. The speed does not appear to influence siderating all the runs made by each of the trucks, the average eccentricities were very small and they will be neglected in this discussion.

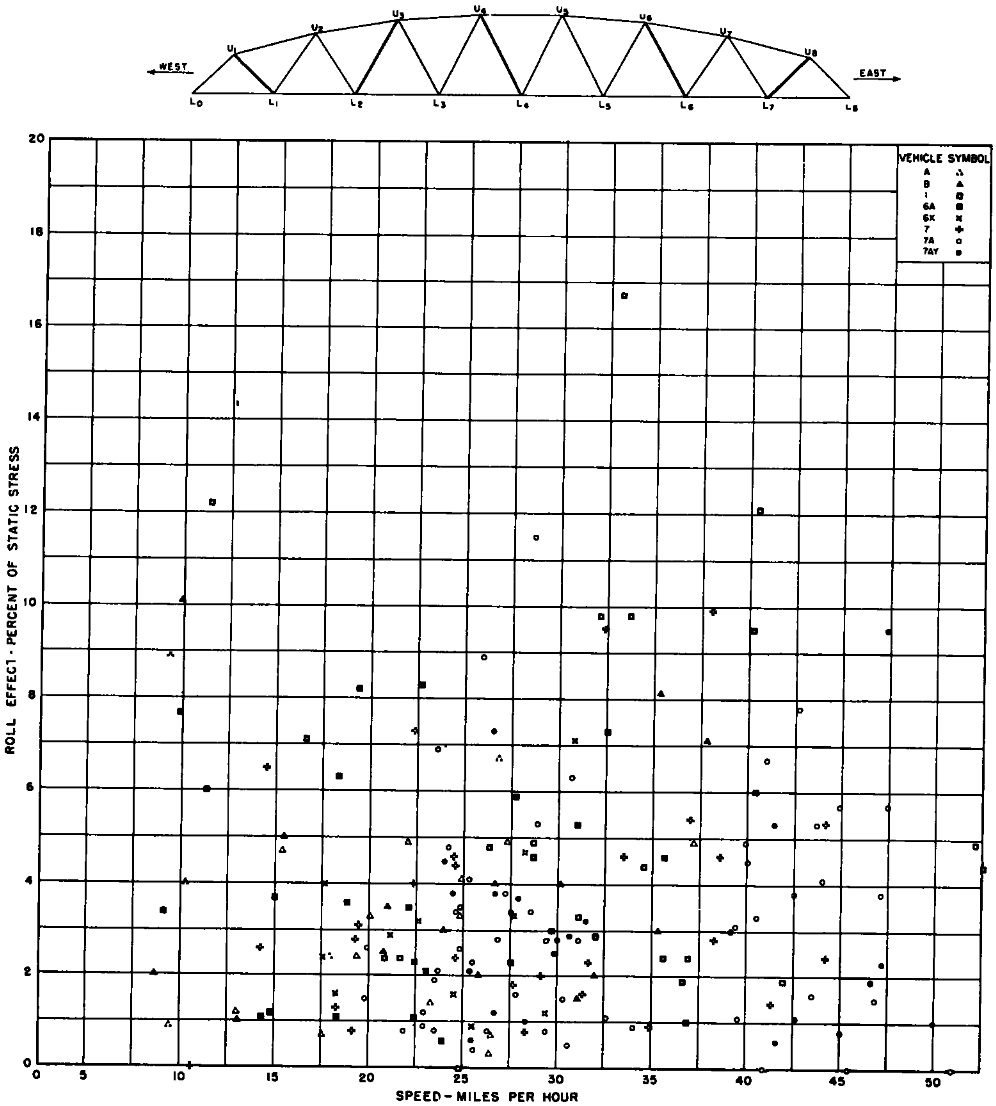


Figure 13. Roll effect in web members.

the deflections to any considerable extent. In these series of tests, the trucks generally traveled along the center line of the roadway, with no intentional eccentricity, although in a few runs, the eccentricity was made 2 ft. 0 in. north or south of the center line. Con-

Truck 6A made 32 runs over the structure. The vertical deflections measured in both trusses are plotted in Figure 17. The average deflection of 0.14 in. in the north truss was 88.5 percent of that calculated for static conditions, while the average of 0.15 in. in

the south truss was 92 percent of that calculated. The deflections under 36 runs made by Truck 7 are shown in Figure 18. In this case, the average north-truss deflection was 0.31 in., practically 100 percent of the computed static value. The south truss had an average vertical deflection of 0.36 in., which was 112.5 percent of the computed static value. There were 50 runs by Truck 7A and the deflections of both trusses are plotted in Figure 19. In the north truss, the average deflection of 0.24 in. was equal to the computed value for static loading. In the south truss, the 0.27-in. average was 110.5 percent of the computed static value.

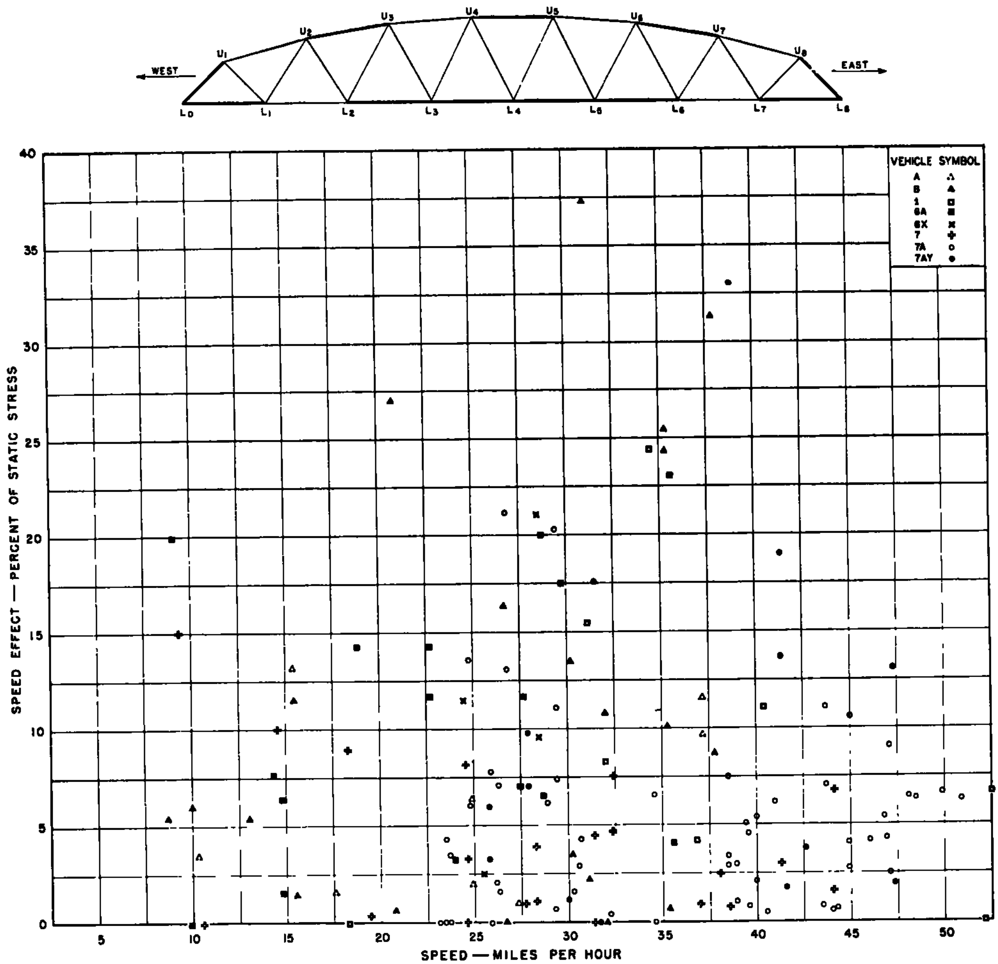


Figure 14 Speed effect in chord members.

Horizontal deflections of both trusses were measured under 36 runs made by Truck 7A, at speeds varying from 5 to 47 mph. The trusses deflected inward, toward the center line of the roadway, the measured amounts being shown in Figure 20. The runs fell into three groups. In the first, comprising 18 runs, the eccentricities with respect to the center line of roadway were small, the average being 4.5 in. south. The average horizontal deflection in the north truss was 0.29 in. and in the south truss 0.20 in. Maximum horizontal deflections occurred under an east bound run at 40 mph., the north truss deflecting 0.36 in and the south truss 0.26 in.

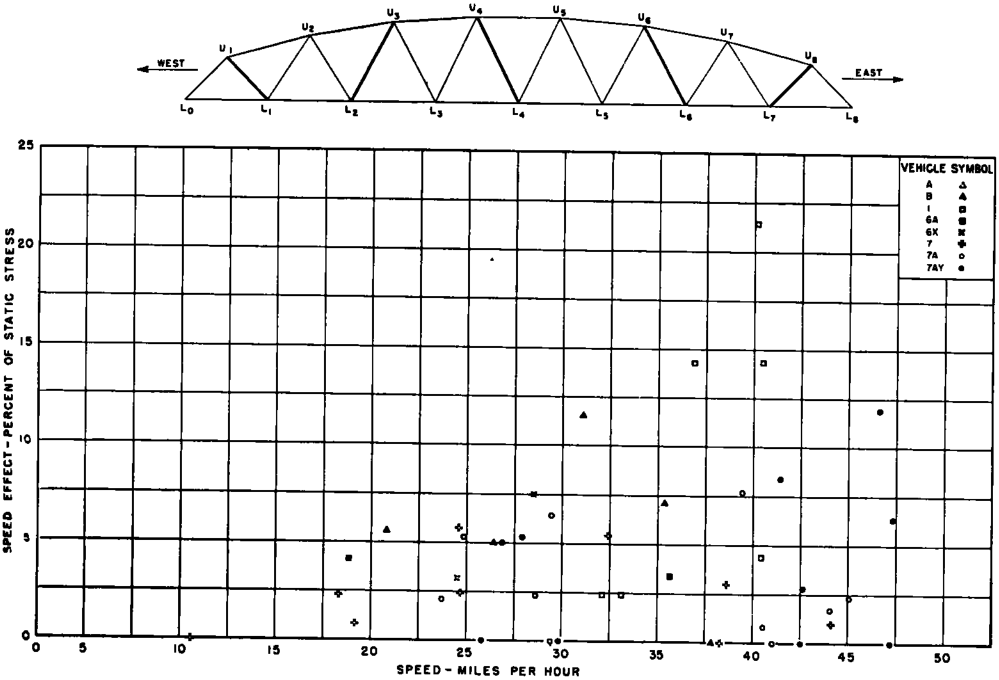
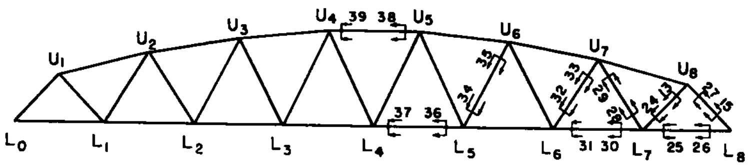


Figure 15. Speed effect in web members.



TRUSS DIAGRAM

SECONDARY STRESSES (BENDING)

MEMBER	SECTION	NO OF RUNS	AVERAGE PERCENT OF RECORDED DIRECT STRESS		
			AXIS 1-1	AXIS 2-2	BOTH AXES
L_7L_8	26	34	397.0	30 0	471 0
	25	34	1620	48 2	2040
U_8L_8	15	34	51	148	21 5
	27	34	874	22.3	1290
L_7U_8	13	34	9 6	201	346
	24	34	294	383	578

MEMBER	SECTION	NO. OF RUNS	AVERAGE PERCENT OF RECORDED DIRECT STRESS		
			AXIS 1-1	AXIS 2-2	BOTH AXES
L_6L_7	30	30	29 3	7.0	37 9
	31	30	37 1	6 9	48 5
U_7L_7	28	30	46 6	20 7	72.4
	29	30	4.5	27.6	32.8
L_6U_7	32	30	11 2	14.5	29 6
	33	30	12 9	16.7	35 8

MEMBER	SECTION	NO. OF RUNS	AVERAGE PERCENT OF RECORDED DIRECT STRESS		
			AXIS 1-1	AXIS 2-2	BOTH AXES
U_4U_5	38	30	13 9	17 8	34.2
	39	30	14.7	16.2	33 2
L_5U_6	34	30	8 2	11 4	21 5
	35	30	2.7	16 4	22 8
L_4L_5	36	30	18 2	3 1	19 8
	37	30	31.0	3 8	34 5

TEST VEHICLE-TRUCK "7A"

Figure 16. Secondary stresses in truss members.

In the second group of 16 runs, Truck 7A ran in the east bound lane, the eccentricity being 4 ft. 7 in. south of the roadway center line. The average horizontal deflection of the north truss was 0.31 in., while that of the south truss was 0.16 in. Maximum horizontal de-

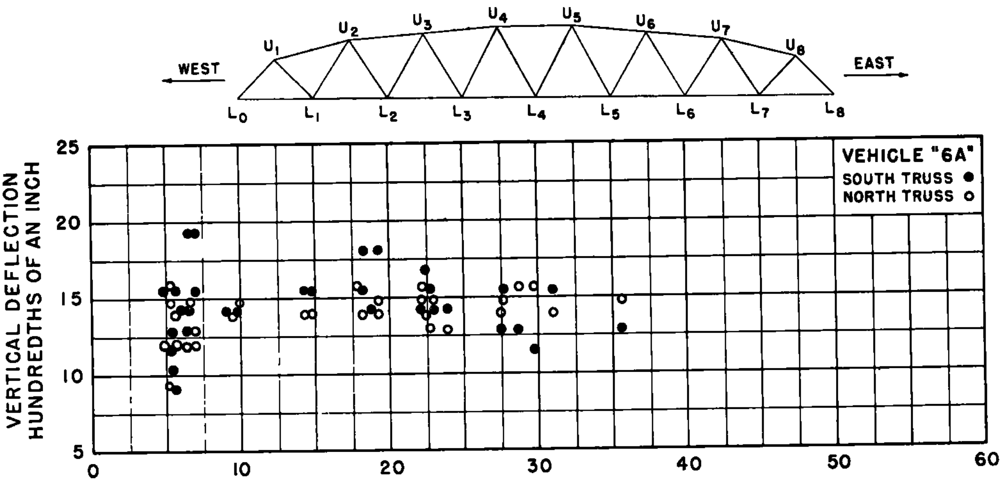


Figure 17. Vertical deflections with Truck 6A.

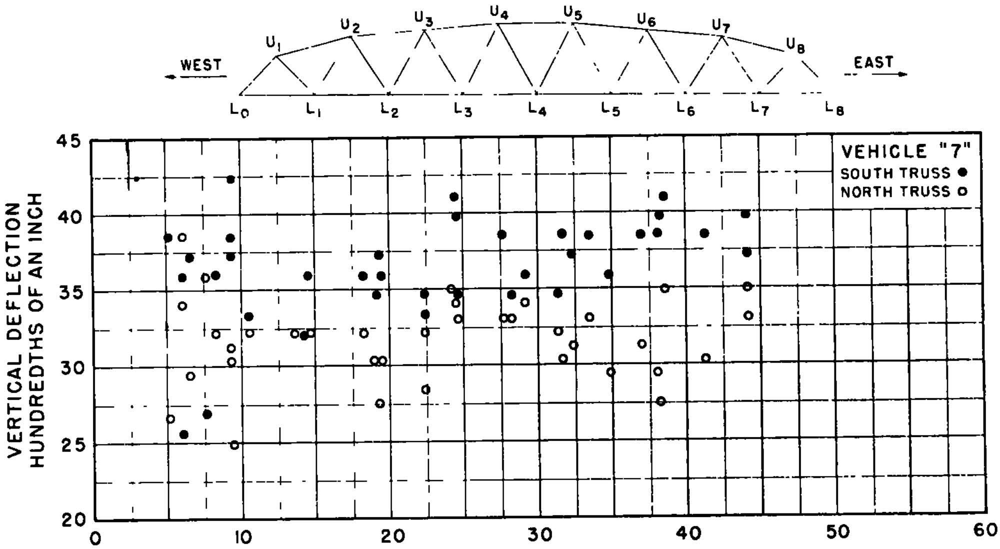


Figure 18. Vertical deflections with Truck 7.

flections occurred with a 38.9 mph. east-bound run, the north truss deflecting 0.34 in. and the south truss 0.22 in.

Only two runs were made in the third group, Truck 7A running in the west-bound lane with an eccentricity of 4 ft. 7 in. north. The speed in both runs was 8.3 mph. The average horizontal deflection in the north truss was 0.19 in. and that in the south truss 0.22 in.

From these three series of runs, it appears that the horizontal truss deflections reached their maximum values at speeds of approximately 40 mph.

STRESSES IN STRINGERS AND FLOOR BEAMS

In order to measure the stresses in the floor system, gages were installed on the

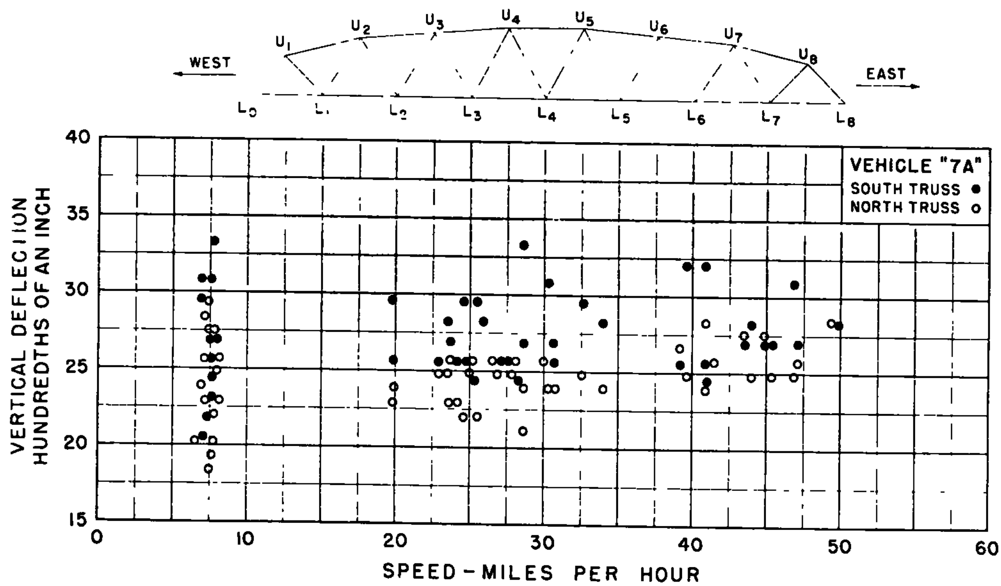


Figure 19. Vertical deflections with Truck 7A.

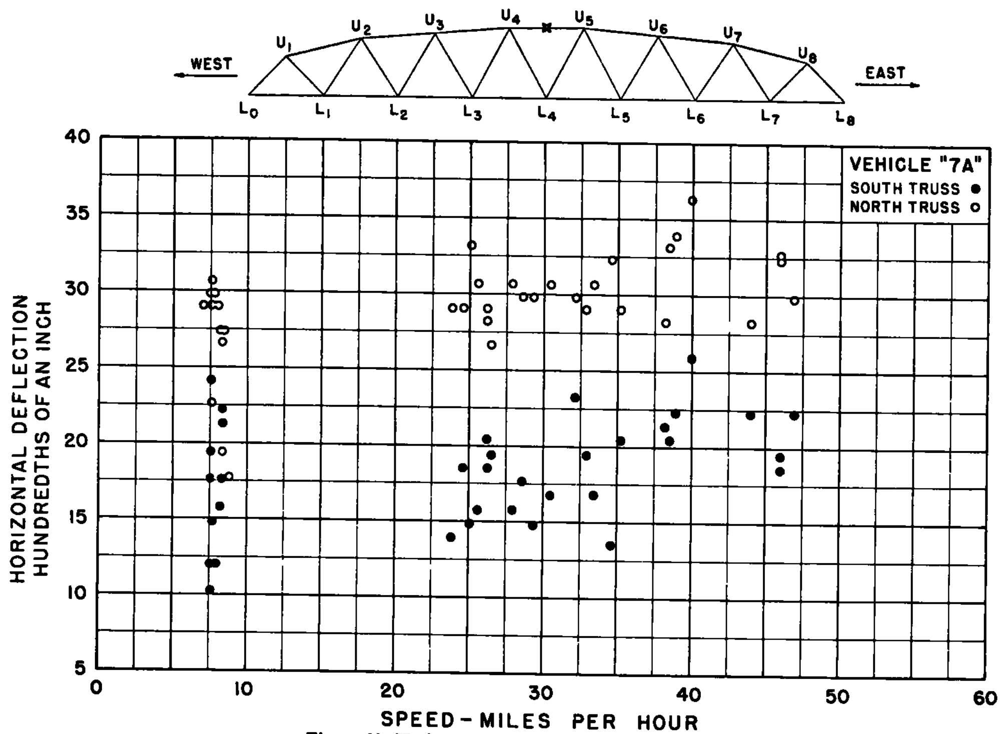


Figure 20. Horizontal deflections with Truck 7A.

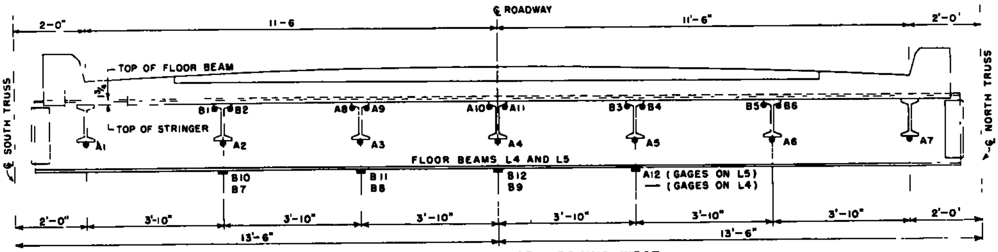
stringers in the panel L_4L_5 and on the floor beams L_4 and L_5. The stringer gages were located in the center of the panel, midway between the adjacent floor beams. Three gages were placed on each of the interior stringers, as shown in Figures 4(a). On the

exterior stringers, the compression flange gages were omitted, leaving only the gages on the tension flanges. Since the compression flanges of the floor beams are imbedded in the concrete floor slab, gages were placed only on the bottoms of the tension flanges, as shown in Figure 4(a). A section through the floor (Fig. 21) shows the stringer and floor beam gages in position. Floor beam L_4 had gages B7, B8 and B9 on the tension flange, while floor beam L_5 had gages A12, B10, B11 and B12.

The stresses in the stringers and floor beams were measured under 84 runs over the structure in both directions at speeds ranging from 5.0 to 49.5 mph. Vehicles A and B traveled along the center line of roadway in all their runs, the small eccentricities being entirely accidental. Truck 7A also made 8 runs along the roadway center line, and 40 runs in the east bound lane, with an average eccentricity of 4 ft. 0 in.

The compressive stresses measured in the top flanges of the stringers were of considerable magnitude and the neutral axes generally were about 5 in. below the tops of the stringers, or about 1 in. above the centroids. Had there been full composite action between stringers and floor slab, the neutral axes would have been close to the tops of the stringers and the stresses in the top flanges would have been very low. It is evident, therefore, that the bond between the stringers and floor slab had broken. Friction between the slab and the stringers and perhaps bearing of the slab on the top flanges of the floor beams were responsible for the small amount of composite action which was taking place.

The stresses recorded by the two gages in the compression flange of a stringer were by no means equal. Frequently the stress on one side of a flange exceeded that on the other side by 25 percent and sometimes one stress was more than twice the other. The floor slab acted laterally as a continuous beam on elastic supports and the deflections under the action of the wheel loads were sufficient to cause unequal bearing pressures across the stringer flanges. Since the gages recorded stress in a longitudinal direction, the difference in stress on the two sides of a compression flange must have been caused by a difference in the amount of friction (composite action) between the stringer and the floor slab over the width of the flange. The smaller flange stress occurred on the side in which the amount of friction (composite action) was the greater. It follows from this that the bearing of the floor slab on any one stringer was greater on the side of the flange in which the compressive stress was the smaller. Hence there must also have been some eccentricity of vertical loading,



SECTION THROUGH FLOOR LOOKING WEST

TENSILE STRESSES IN STRINGERS (GENERAL AVERAGES)

1	2	3	4	5	6	7	8	9	10
VEHICLE	RUNS	ECC	MAX	MIN	AV	SUM	MAX/AV	MIN/AV	MAX/SUM
A	15	0-3"S	5.40	1.45	3.21	22.44	1.68	0.45	0.24
B	21	0'-1"N	4.94	1.26	2.95	2063	1.68	0.42	0.24
7A	8	0'-7"S	4.68	0.35	2.47	17.29	1.89	0.16	0.27
7A	40	4-0'S	4.63	0.31	2.17	15.18	2.15	0.14	0.31

NOTE ALL STRESSES ARE IN THOUSANDS OF POUNDS PER SQUARE INCH

TENSILE STRESSES IN FLOOR BEAMS (GENERAL AVERAGES)

11	12	13	14	15	16	17	18
FLOOR BEAM	VEHICLE	RUNS	ECC	GAGES			
				B7	B8	B9	—
L4L4	A	15	0-3"S	4.04	6.71	7.37	
	B	21	0-1"N	3.05	5.80	6.54	
	7A	8	0'-7"S	3.08	5.58	5.70	
	7A	40	4'-0"S	3.64	5.55	5.52	
				GACES			
				B10	B11	B12	A12
L5L5	A	15	0-3"S	4.04	6.68	6.73	5.13
	B	21	0'-1"N	3.15	5.77	6.18	5.08
	7A	8	0-7"S	3.48	6.33	5.90	4.16
	7A	40	4'-0'S	4.03	5.89	5.07	3.24

Figure 21. Tensile stresses in stringers and floor beams.

which would have resulted in torsional stress. The inequality of the compressive stresses over the width of the flanges indicated that there was bending about the vertical axes. In stringers carrying a fair amount of vertical load, the flexural stresses in the compression flanges about the vertical axes amounted to about 25 percent of those about the horizontal axes.

The tensile stresses from the vertical loads are summarized in Figure 21, the values given being general averages of all the runs made by each vehicle. Maximum, minimum, and average stresses in the stringers are shown in Columns 4, 5, and 6, while the sums of the tensile stresses in the seven stringers appear in Column 7. Ratios of maximum to average stress are given in Column 8. For Vehicles A and B, traveling along the roadway center line, the maximum stress in one stringer was 1.68 times the average stress in all the stringers. The same ratio for Truck 7A, also traveling along the center line of roadway, was 1.89. However, when Truck 7A ran in the east bound lane, the maximum stress in one stringer was 2.15 times the average. The maximum stresses in the stringers do not seem to be dependent upon vehicle speed.

Since the stringers were of the same cross section and the location of the neutral axis varied only very little, it was possible to determine the portions of the total live load in the panel carried by the most highly stressed stringer. These are shown in Column 10 by the ratios of the maximum stress to the sum of the stresses. The value is greatest in the case of Truck 7A, running in the east-bound traffic lane. It may be said that the most highly stressed stringer carried 0.31 of an axle load, or 0.62 of a wheel load. For a structure with a 3 ft. 10 in. spacing of stringers, this amounts to an effective lateral distribution of 3.83 ÷ 0.62 or 6.18 ft. per wheel when only one traffic lane is loaded.

The measured tensile stresses in the stringers from the vertical loading were considerably less than the computed static values. For the runs made by Vehicle A, they were about 38 percent, for those by Vehicle B about 52 percent, and for those by Truck 7A about 48 percent. These reductions may be attributed to at least two causes. The composite action between stringers and floor slab, even though it was limited to friction, was the major factor. Although the stringers had only standard connection angles to the floor beams, there was a certain amount of continuity. Loads in either of the adjacent panels produced a small amount of negative stress in the stringers whose stresses were being measured. This, however, was only a minor factor.

In the floor beams, the stresses measured in L_4L_4 and L_5L_5 were in general agreement, and again, they were less than the computed static values. Under Vehicle A the measured stresses were about 58 percent of those computed, under Vehicle B about 67 percent, and under Truck 7A about 82 percent. In computing the static stresses, the usual assumption of simply supported ends was made, taking no end restraints into consideration. There may also have been some composite action with the floor slab.

TABLE 2
STUDY OF LONGITUDINAL STRESSES

LONGITUDINAL STRESSES (AVERAGES)

LOC	GAGE	EAST BOUND RUNS				WEST BOUND RUNS			
		CONSTANT		BRAKING		CONSTANT		BRAKING	
SHOES AT L_8	A1		-0 59		-0 64		-0 64		-0 57
	A2	+1 00		+0 98		+1 05		+0 95	
	A3	+0 85		+0 64	-0 16	+0 42	-0 21	+0 19	
	A4	+1 13	-0 19	+1 10	-0 19	+0 75		+0 71	
	B1		-0 77	+0 26	-0 74	+0 19	-0 58	+0 17	-0 58
	B2	+0 75		+0 78		+0 94		+0 69	
	B3	+0 57	-0 57	+0 41	-0 34	+0 24	-0 43	+0 46	-0 41
	B4	+0 48	-1 35	+0 41	-1 27		-1 45	+0 05	-1 30
BOTTOM CHORDS L_7L_8	A5	+0 77		+0 68		+0 58		+0 58	
	A6	+0 74		+0 48		+0 50		+0 62	
	A7	+0 58	-1 25	+0 05	-1 20		-1 15		-0 96
	A8	+0 58	-1 36	+0 63	-1 58	+0 10	-1 21	+0 10	-1 14
	B5	+0 56	-0 19	+0 54		+0 61		+0 66	
	B6	+0 76		+0 69		+0 67		+0 71	
	B7	+0 57	-1 14	+0 24	-0 98		-1 10		-1 12
	B8	+0 37	-1 40	+0 28	-1 26		-1 45		-1 42
LATERALS	A9	+1 39		+1 33		+1 15	-0 09	+1 22	-0 09
	A10	+2 35		+2 21		+2 16		+2 11	-0 10
	B9	+1 27		+0 96	-0 39	+1 09		+1 18	-0 16
	B10	+2 44		+2 37	-0 09	+2 61		+2 70	

ALL STRESSES ARE IN THOUSANDS OF POUNDS PER SQUARE INCH

LONGITUDINAL STRESSES

In order to measure longitudinal forces in the structure from acceleration and from braking, gages were installed on the shoes of both trusses at the fixed end, on the bottom chord members L_7L_8 and on the members of the bottom lateral system in panels L_6L_7 and L_7L_8. These gages are shown in Figure 4(b). Truck 7A was driven on the structure at varying speeds, brought to a complete stop by a heavy application of the brakes and then accelerated. In Table 2, the individual gage

readings from these runs are compared with readings obtained from runs over the structure at constant speeds. The figures given are averages of the maximum values from each run, and they are not instantaneous values.

The truck made five runs over the structure in the east-bound direction, of which one was at a constant speed of 19.5 mph. In the other runs, the entering speed varied from 7.9 to 24.0 mph. In the west-bound direction, there were two constant speed runs at 7.7 and 19.4 mph. and four runs in which the entering speed varied from 8.2 to 22.9 mph.

In the shoes, tensile stresses seemed to predominate, which makes it appear that a portion of the vertical load may have been carried by the slab to the backwall. The gage readings for the runs at constant speed and for those in which braking occurred are so similar that we must conclude that very little, if any of the longitudinal forces from the braking reached the shoes.

The same conclusion is reached with regard to the bottom-chord stresses. The stresses from the constant speed runs are very close to those from the runs in which braking and acceleration occurred.

In the lateral system, the stresses in the panel L_6L_7 were considerably greater than those in panel L_7L_8. This was true in all cases. Although the Gage B9 recorded a small amount of compression under the east-bound runs, there is no evidence that any considerable amount of longitudinal force was carried by the lateral system.

Since the gages in the shoes, in the bottom chords and in the lateral system did not record any large amounts of longitudinal stress, it is likely that these stresses were carried directly to the backwalls by the concrete floor slab.

SUMMARY

The measured static stresses in the top-chord members of the trusses exceeded the computed values, while the stresses in the bottom-chord members were less than those computed. The floor system appears to relieve the bottom chord of some of its tension, but in doing so, the effective truss depth is decreased.

The total impact in the chord members and in the web members approached 50 percent of the static stresses at the critical speeds. The total impact reached a maximum in the chords at a speed of about 30 mph. In the web members, maximum impact occurred at critical speeds of 27 and 42 mph.

Secondary stresses in the truss members were, in general, moderate. They were very high in the members immediately adjacent to the panel points L_8 at the fixed end of the span. Excessive friction about the 4-in.-diameter pins just below these panel points appears to be responsible for the high bending stresses found in these members.

Truss deflections were not excessive. The vertical deflections of the south truss exceeded those of the north truss somewhat, but the horizontal deflections of the north truss were considerably greater than those of the south truss.

The bond between the floor slab and the stringers had broken and the small amount of composite action between them was the result of friction. It was sufficient to reduce the tensile stresses in the stringers considerably. The floor slab appeared to act laterally as a continuous beam on elastic supports and its deflections under the wheel loads were sufficient to cause unequal bearing pressures across the widths of the stringer flanges. This resulted in bending about the vertical axes of the stringers and in torsion.

Stresses in the floor beams were somewhat less than the computed static values.

The gages placed on the shoes, the bottom chords, and the members of the bottom lateral bracing system adjacent to the fixed end of the bridge did not record any appreciable amounts of longitudinal stress from acceleration or braking.

ACKNOWLEDGMENT

The negotiations with the Association of American Railroads were conducted through Brigadier General Lacey V. Murrow, assistant to the vice president, Operations and Maintenance Dept., Washington, D. C. The tests were performed by the Engineering Division of the AAR, under G. M. Magee, research engineer. E. J. Ruble, structural engineer, planned the program and was in general charge of the work. His aid and helpful suggestions during the analysis of the data are greatly appreciated. L. E. Monson, assistant structural engineer, was in immediate charge of the tests.

E. L. Schmidt, acting secretary of highways, Leo A. Porter, bridge engineer and H. G. Van Riper, maintenance engineer, Pennsylvania Department of Highways, coöperated in every way to make the project a success. They were generous in supplying material and equipment and the services of men necessary in the conduct of the work. Schmidt, then chief engineer of the department, was instrumental in originating the joint project.

D. K. Chacey of the Transportation Corps arranged for the use of the special military vehicles from the Letterkenny Ordnance Depot, Chambersburg, Pa.

Bernard F. Kotalik, highway-bridge engineer, Bureau of Public Roads, analyzed much of the data and prepared the drawings.

DISCUSSION

Thomas C. Kavanagh, *Professor*, and Edward C. Holt, Jr., *Instructor, Department of Civil Engineering, Pennsylvania State College*—The publication of extended data on field tests of bridges, such as those conducted and so well reported by the author, yields important information from which structural engineers may gage the adequacy of commonly accepted or prescribed design procedures. The particular tests of this report are of special value because they were conducted on a so-called half-through, or pony, truss bridge, in which the top chords of the trusses are located substantially above the deck construction, but not so far as to permit cross connection by a system of top laterals.

The half-through type of bridge construction offers certain marked advantages, among which the following may be cited: (1) On highway spans there is an esthetic advantage in openness to the sky; (2) on railway spans an element subject to blast and acid gas attack is eliminated; and (3) in both cases there is removed all limitations upon the height of vehicles. It is believed that there would be more use made of this type if there existed more precise knowledge of the minimum requirements for stability, carrying with it the ability to make estimates of comparative cost with entire confidence.

It is usual in such construction for the top chord greatly to exceed in slenderness ratio the limit required for the primary stability of the chord in the direction perpendicular to the plane of the truss. The chords are restrained laterally by vertical or diagonal members of the trusses to which they belong, and sometimes also by inner and outer brackets at the panel points. In the subject bridge, which is of the Warren type, the verticals are not present and the diagonals take over the function of restraining the chords. All of these elements interact with the top chord when the bridge is loaded, so that the chord is compelled to deflect laterally. At the same time they serve to support the chord and prevent it from buckling laterally. The stability and economy of the system is a function of the design and loading condition of the entire bridge. The variables involved are numerous, and each has a considerable range of practical possibilities.

It is of interest to consider the Fort Loudon Bridge from the standpoint of lateral buckling of the top chord. This chord will buckle laterally when the stress in U_4U_5 is about 32 ksi., slightly below the yield point. The maximum total stress observed in this member, due to Vehicle 7, including dead load and impact stresses, is about 9.0 ksi. The factor of safety against lateral buckling is thus about 3.5 for this particular loading, which is as heavy as any likely to occur. Based on the live load and impact stresses alone, which is more logical, the factor of safety is about 5.8. It is evident, therefore, that the top chord of the bridge is unnecessarily stiff as far as lateral buckling is concerned. It should be kept in mind that the above factors of safety have been calculated on the basis of the observed static stresses in the top chord members which exceeded the computed value for this chord by 17.5 percent. Factors of safety based on calculated stress would be even higher.

This overdesign of pony truss bridges against lateral buckling has been common practice in the past, due largely to a lack of precise knowledge of the minimum requirements for the stability of such bridge chords.

The writers now have under way an investigation of the pony truss problem,[1] sponsored jointly by the Column Research Council of the Engineering Foundation, the Pennsylvania State Highway Department, and the Bureau of Public Roads, which, it is anticipated, will throw some light on the buckling

[1] "Buckling of a Continuous Beam-Column on Elastic Supports", E. C. Holt, Jr., Report No. 1, 1951.

action of this type of structure. The problem is first being treated theoretically to develop suitable methods of analysis and design for such bridges. The theoretical results will be checked experimentally by tests to failure of several large scale model bridges, and also by comparison with the performance records of existing bridges, such as those made available in this report. Finally, a study will be made of the economics and limiting spans for this bridge type. It is anticipated that the use of the pony truss bridge will prove to be feasible for spans considerably longer than have been contemplated in the past.

E. J. Ruble, *Structural Engineer, Association of American Railroads*—The report covering the tests on the Fort Loudon Highway Bridge is interesting as it clearly indicates that the action of a highway bridge is about as unpredictable as that of a railroad bridge. It is obvious that it will be necessary to secure similar data on a large number of structures before definite recommendations regarding impact can be made.

The large variation between the recorded and calculated static stresses reported for this bridge is typical of that found in railroad truss bridges. It has been found in through truss spans that the stresses in the lower chords vary from 60 to 80 percent of the calculated stresses while the top-chord stresses are usually 10 to 20 percent greater than those calculated. In deck-truss spans the opposite is true with low top-chord stresses and higher lower-chord stresses, so the explanation for this action offered in this report appears reasonable. As additional evidence, considerable trouble has been encountered in the past with the breakage of the stringer connection angles and rivets in the through truss spans, indicating that the stringers carry some direct stress.

The diagrams of total impacts indicate that the present impact allowance was not great enough for this bridge. The impact allowance is undoubtedly based upon the results obtained from vertical deflection readings, which only indicate the average impacts in all the individual truss members. With the advent of the electrical strain gages, it has been found that the impacts in the individual truss members are not the same but vary from member to member, depending upon the location of the moving load and the exciting force. In general, the impacts determined from strain gage readings are about twice those obtained from deflection readings.

It is generally recognized that an increase in the live-load weight reduces the natural period of vibration of a structure with a corresponding reduction in the impact; however, it can be seen from Figures 10 and 11 that the total impacts recorded under the passage of the heavy Vehicle A during these tests were just as large as those recorded under the lighter vehicles, indicating that the principal cause of the impact was not produced by a forced vibration of the structure.

The rolling of the spring-borne weight of the test vehicles was very noticeable during the conduct of the tests, and the rolling effect percentages found in the report appear reasonable. The rolling effect in the structures will, of course, vary with the spacing of the particular supporting members, the percentage being large for narrow spacing such as stringers and smaller for wide spaced members such as trusses. The results of this investigation indicate that a separate allowance for this effect, which varies with the spacing of the supporting members, should be included in the specification.

The secondary stresses recorded in the individual members of this bridge were reasonably small, except in the lower-chord member near the fixed bearing, and are about the same as those found in the members of railroad truss bridges. It has been common practice in the past to neglect secondary stresses but experience has shown that they should be considered in determining the long-time carrying capacity of any structure.

The principal effect of a secondary stress on a compression member is to introduce bending moments in the ends of these members, and if these moments are in opposite direction, the buckling strength of the member can be considerably reduced. Several fatigue failures have occurred in tension members of railroad truss bridges, particularly in lower-chord members near the bearings, and strain gage readings taken at similar locations have indicated large secondary stresses. In some cases, the bending stresses have been large enough to produce a final compressive stress on the underside of the lower-chord member near the bearing with a tensile stress on the top side of the end post.

The various factors tending to reduce the total stresses in the stringers are usually present in every truss span, and these results indicate that an appreciable saving can be made in the floor systems by a thorough investigation of typical bridges. It has been found in railroad structures that partial continuity of the stringers is present through the action of the web connection angles and that some composite action between the concrete deck and steel stringers exists even when the concrete deck is precast and is just resting on the top flanges of the stringers.

In conclusion, the author wishes to congratulate Van Eenam on the preparation and presentation of such an outstanding report. It should enable highway-bridge engineers to better understand the actual behavior of their bridges under dynamic loading.

JONATHAN JONES, *Chief Engineer, Bethlehem Steel Co.*—The tests on this structure were handicapped to some extent from the fact that the dead load was so considerable and consequently the live load stresses so small that all the comparisons between recorded and actual stress are comparisons between two rather small things, in which I presume the same errors could be made as would have been made in records of much larger quantities. This, because of the general consistency of the departures from calculated stresses, does not prevent the recognition of trends, but it does make it a little difficult to pursue one particular explanation to a definite conclusion.

For instance, a general increase of top-chord stress on the order of 15 to 20 percent is no doubt ascribable in part to participation of the stringers and floor slab, but since the latter are situated below the level at which they would have to be in order actually to decrease the lever arm of the top chord by 15 to 20 percent, it would appear that there must be another cause or causes operating as well. For instance, we might speculate that a slight inefficiency of the rollers at the expansion end would cause a lag in responding to the stress elongation of the bottom chord, which would tend to produce the results observed. The data do not suffice to pursue this thought to a demonstration.

Similarly, the friction on the pin at the fixed end would, as suggested in the report, cause secondary stresses in the direction indicated by the record; but so far as we can understand would not change average tension to average compression as recorded; so that, here again, there is another undiscovered force at work.

When similar tests are made on other bridges in the future, it would be very interesting to take accurate measurements on the movements of the expansion end, with relation to time, and see whether any information could be picked up in that way.

L. E. GOODMAN, *Research Assistant Professor, University of Illinois.*—The publication of reliable field measurements on full-size structures is always of interest and importance to the engineering profession. There is no complete substitute for field tests. Designers of highway bridges are indebted to Van Eenam and to the agencies which cooperated in the investigation he reports.

This investigation throws considerable light on a number of structural questions affecting the design of pony-truss highway bridges. The importance of including the stiffness of the roadway floor system in computing static stresses, the relative magnitudes of secondary and primary stresses, composite action between floor slab and stringers, longitudinal stresses due to acceleration or braking are all noted. In general, where discrepancies between theory and measurement have been observed, explanations have been advanced to account for them. But the impact effect (Figures 10–15) exhibits such great variation from run to run that at first glance it appears hopeless to attempt any rational prediction of the magnitude of this important source of stress. Actually this is not the case. If the test results are plotted in a different way they can be shown to be in good agreement with small-scale laboratory tests performed with a simplified model. These small-scale test results in turn can be related directly to a rational theoretical analysis.

It can be shown that a single smoothly-running mass traversing a simple-span beam produces a dynamic stress amplification determined by two dimensionless parameters, R, the ratio of the weight of the moving mass to the weight of the bridge and α, the ratio of half the fundamental period of the bridge to the time of transit of the load. Judging from

the observed deflections, the period of the Fort Loudon bridge can be estimated to be approximately 0.33 sec. Even at the fastest speeds used in this series of tests (55 mph.), the time of transit was about 1.36 sec., corresponding to a maximum α of about 0.12. This means that the load was applied relatively slowly compared with the natural period of the structure, and therefore it is not surprising that no pronounced speed effect was observed.

Small-scale laboratory tests[1] to determine the dynamic effect of a smoothly running load on a simple span have been performed at the University of Illinois as a part of a primarily analytical investigation of impact on highway bridges. This investigation is being conducted by the Department of Civil Engineering in

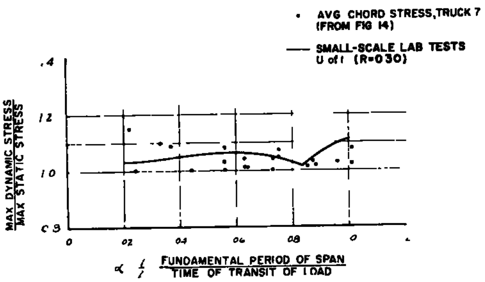


Figure A. Speed effect in Ft. London tests and in small-scale experiments.

coöperation with the Illinois Division of Highways and the Bureau of Public Roads. It is interesting to compare the results of these small-scale tests, which were intended primarily to check analytical results, with the full-scale measurements reported. The comparison is shown in Figure A of this discussion. Circles represent values taken from Figure 14 of the paper, while the heavy solid line corresponds to the results of the small-scale laboratory tests. The truck for which the circled values have been plotted is that designated No. 7 (R = 0.30). While this is a multiple-axle vehicle, the axles are well-spaced so that (counting tandem axles as one load) there is little interference between them. The ordinate of the figure is the average stress amplification factor—the ratio between maximum dynamic and maximum crawl or static stress at a number of points along the span. The abscissa is the quantity α which is proportional to the speed of the vehicle.

When the dynamic stress measurements are plotted in this dimensionless way it can be seen that there is a gratifying agreement between the two sets of results. The laboratory tests, for example, show a rather broad maximum at $\alpha = 0.06$ or about 28 mph. It should be borne in mind, however, that in these tests the small (6 ft.) test span and the rolling load were carefully controlled. In particular the effect of roadway roughness was eliminated so far as possible. It would be interesting to know whether roadway roughness rather than speed was responsible for some of the relatively high amplification factors observed in the Fort Loudon tests at low velocities.

A similar comparison between the small-scale laboratory tests and the Fort Loudon bridge measurements could be made for the other test vehicles. What is important is not so much the extent of agreement between tests performed on such widely different scales as the fact that the result of laboratory tests can be predicted analytically. It is this which opens the possibility that through a combination of field tests and analytical developments the determination of impact allowances for highway bridges may be placed on a firm rational basis.

NEIL VAN EENAM, *Closure*—The writer wishes to express his gratitude to those who have contributed to the discussion of this paper. Several interesting points have been brought out. Ruble's comprehensive remarks are based on his long and varied experience in the testing of railroad bridges and structures. His comments emphasize the major conclusions of the report.

It is true, as Jones states, that the dead-load stresses in the truss members were large and the live-load plus impact stresses relatively small. Unfortunately, the two heaviest test vehicles, Trucks A and 7, made all their runs along the centerline of the roadway, in order to gain as much lateral distribution as possible. It was feared that the heavy axle concentrations might seriously overstress the floor system. Later it was discovered that this precaution was unnecessary, because the composite action between the slab and the stringers strengthened the floor to such an extent that the stresses were moderate. Had

[1] For a complete description of these tests see Section I, Second Progress Report, Highway Bridge Impact Investigation, Civil Engineering Studies, Structural Research Series No. 24, University of Illinois, Urbana, Illinois, April 1952.

these heaviest vehicles run in their normal traffic lanes, live-load plus impact stresses in the members of the adjacent truss would have been increased almost 50 percent. Future tests may benefit from this experience.

While we agree that there may be reasons other than the interaction of the floor system with the bottom chord to account for the in-increased static stresses in the top chord and the decreased stresses in the bottom chord, we cannot concur that inefficiency in the action of the rollers at the expansion end would tend to produce the observed results. A lag in the response of the rollers to an elongation of the bottom chord would induce a horizontal compressive force on the rollers and on the fixed shoe. This eccentric force would produce compression in the bottom chord and tension in the top chord. Since both of these are opposite in sign to the live-load stresses, both top-chord and bottom-chord live-load stresses would be reduced. Ruble, in supporting the view that the stringers of through trusses frequently carry heavy tensile stress, cites the fact that considerable trouble has been experienced by the breakage of stringer connection angles and rivets from tension.

Kavanagh and Holt have computed the strength of the top chord of the bridge and they estimate that it will buckle laterally when the unit stress in member $U_4 U_5$ is about 32 ksi. This value is not questioned, although it probably exceeds the proportional limit of the steel. Their values of the dead-load, live-load, and impact stresses in the structure appear, however, to be too low, and because of this they have seriously overestimated the safety factors against lateral buckling. The dead-load stress in the member $U_4 U_5$ is about 8 ksi., and the provision for live-load and impact stresses is 5 ksi., making a total design stress of 13 ksi. In this connection, it may be recalled that at the time this structure was built, the basic working unit stress in tension generally was 16 ksi. Since both trucks A and 7, running along the centerline of the roadway, produced live-load plus impact stresses in the members $U_4 U_5$ slightly in excess of 4 ksi., it is reasonable to assume that regular truck traffic, using the normal traffic lanes, produces live-load plus impact stresses of at least 5 ksi. and that these members therefore carry their full design stress of 13 ksi. Although the safety factors resulting from the use of these revised values are lower than those stated by Kavanagh and Holt, the writer agrees wholeheartedly with them that in low trusses, safety factors have often been made unnecessarily high.

Goodman's discussion is both interesting and instructive, since it is based on an investigation of impact in highway bridges now being conducted at the University of Illinois. The natural frequency of the unloaded bridge actually is 4.17 cycles per sec., and the fundamental period is therefore 0.24 sec. This will move the plotted points in Figure A somewhat to the left, but it does not destroy the validity of Goodman's comparison. Speed effects are obtained from differences between the average observed mean stresses and the static stresses. In the Fort Loudon tests, some of the readings were taken on bottom chord members, in which the static stresses were less than computed stresses. In particular, a few readings were taken on members $L_0 L_1$, in which the stresses were very low. The three points on Figure A with values of α (alpha) less than 0.04 and with amplification factors greater than 1.0 are in this category, and they represent differences between two very small quantities. This fact, rather than roadway roughness, is responsible for these abnormally high values shown in the low speed range.

The discussions have considerably enhanced the value of the report.