Load Distribution between Girders on San Leandro Creek Bridge

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● IN THE spring of 1950, the Institute of Transportation and Traffic Engineering, University of California, in cooperation with the Bureau of Public Roads and the California Division of Highways, initiated an extensive program of strain and deflection measurements on a state freeway bridge crossing San Leandro Creek in Oakland. One of the main subjects under investigation was the distribution of the load between the girders. It is the purpose of this report to discuss briefly some of the results of the tests concerning load distribution as affected by (1) composite action of the concrete slab with the steel girders, (2) longitudinal and transverse position of the load, and (3) steel diaphragms.

These test results are compared with theoretical analysis and with AASHO specifications

Figure 1 indicates the framing of the tests spans and the locations of the principal gage stations. The bridge is composed of an 8-in. concrete deck with sidewalks, supported by three longitudinal steel girders on 11-ft. centers. There are two parallel structures of two lanes each; each structure having 23 spans. Every third span consists of a suspended span, hinge-supported on cantilever arms which are continuous over two spans on either side Diaphragms were placed at the quarter points and center of the continuous spans and near the hinges and center of the suspended spans. Two representative spans on one of the structures, Spans 19 and 20, were chosen for test; 19 being a typical suspended span, and 20 a typical continuous span.

The framing plan indicates the three supporting girders, designated as right, middle, and left The principal gage locations, designated as 19.5, 20.0, 20.5, and 21.0, are indicated by dotted lines.

Figure 2 shows the steel framing in the test spans and the installation of the numerous wires connecting the gages to the recording equipment About 350 SR-4 strain gages, 16 Carlson strain meters, and 8 induction-type deflectometers were mounted on the test spans. It will be noted that the exterior girders rest on the columns and the middle girder is supported by the cross-beams The hinge plates and diaphragms also appear in the photograph

Figure 3 shows the completed bridge with the Euclid test vehicle loaded to a gross weight of 67,000 lb. with sand and steel ingots The rear axle carried a load of 50,000 lb. and the front axle 17,000 lb The spacing between axles is 13 ft.

Figure 4 shows the five transverse positions of the test vehicle designated as left, half-left, center, half-right, and right. The locations of the SR-4 gages on the girders and the Carlson strain meters in the concrete are also shown.

In order to estimate the effect of composite action, the concrete deck was assumed to be divided into three sections; each section was considered as belonging to one girder. On the basis of composite action, assuming $n = 10$, it will be noted that the moments of inertia are three to four times larger than for the steel alone. The left girder has the highest composite moment of inertia because the slab was made thicker on that side to provide for transverse drainage.

In order to determine whether composite action existed, strain measurements for the three girders at Station 19.5 were plotted. These measurements were taken from oscillograph recordings of strain when the rear axle of the slowly moving vehicle was at midspan. Figure 5 shows the strains for each girder for two transverse positions of the load, the positions being those which produced the largest strains in the girder. It will be noticed that for each of the loading conditions, the four values of strain lie practically on a straight line

The theoretical neutral axes were computed on the basis of full composite action assuming the sections shown in previous Figure 4 It will be noted that the experimental neutral axes coincide closely with the theoretical axes for all three girders For the middle girder a strain diagram assuming no composite action has been added for comparative illustration. This shows a bottom flange tensile strain about 70 percent higher than the observed strain On the top flange, the assumption of no composite action resulted in high compression, whereas the observed strain was almost zero, as should be the case for full composite action. Since no shear connectors were

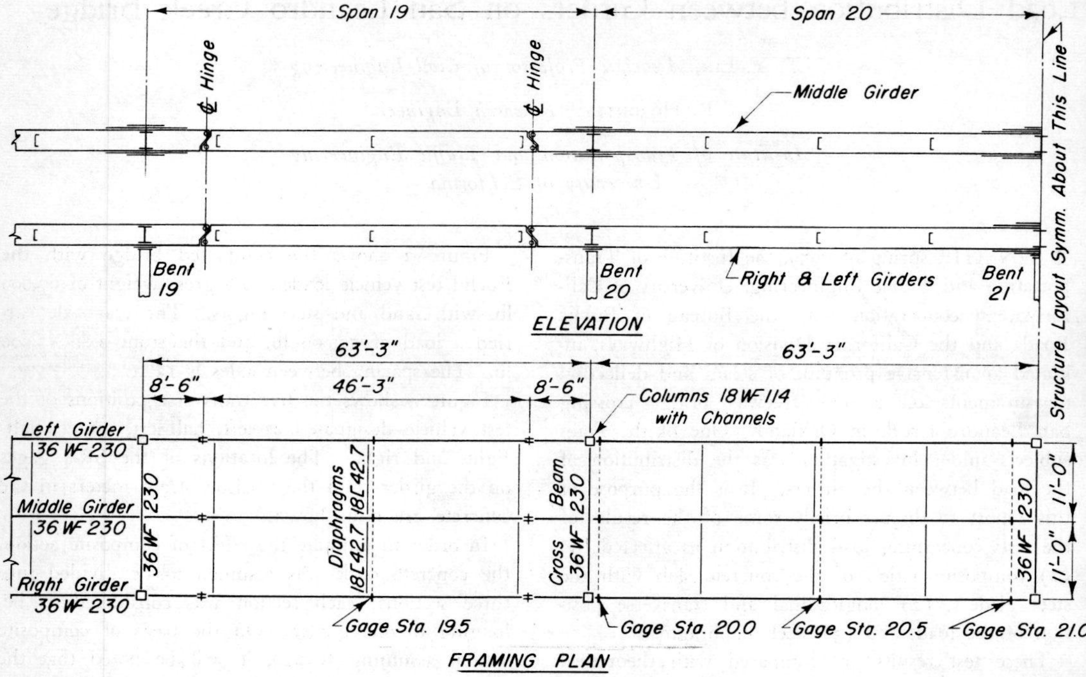


Figure 1. Steel layout of test spans.

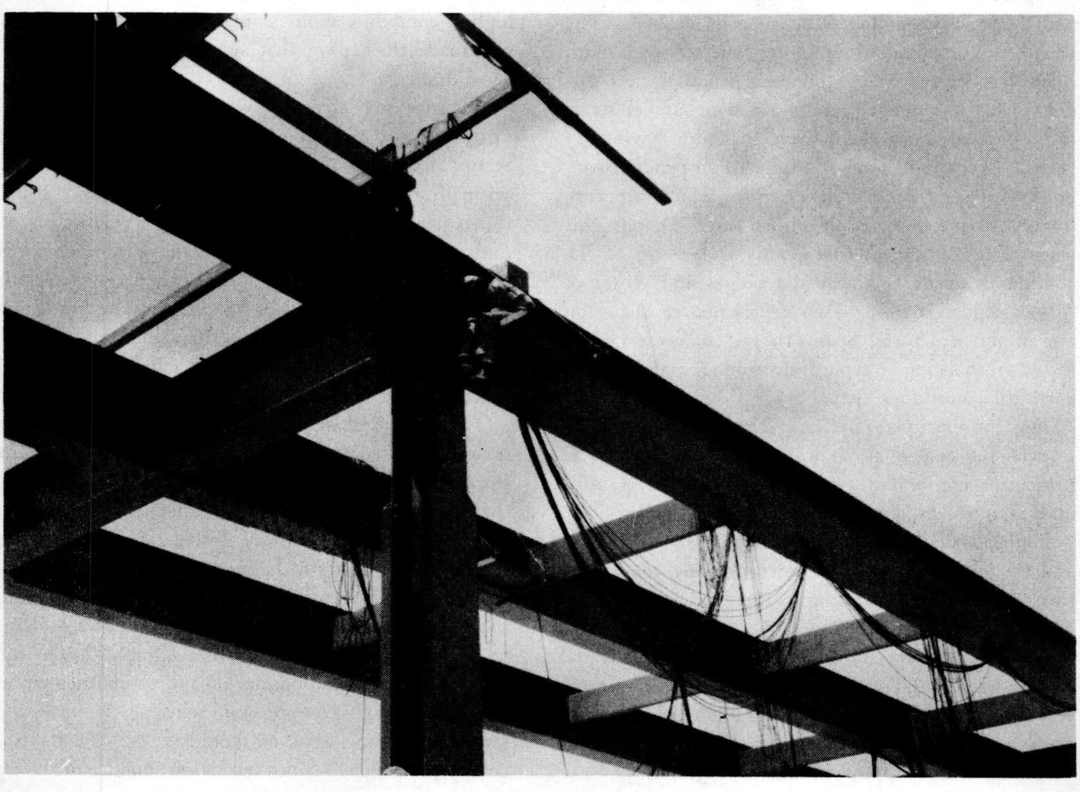


Figure 2. Steel framing prior to placement of deck.

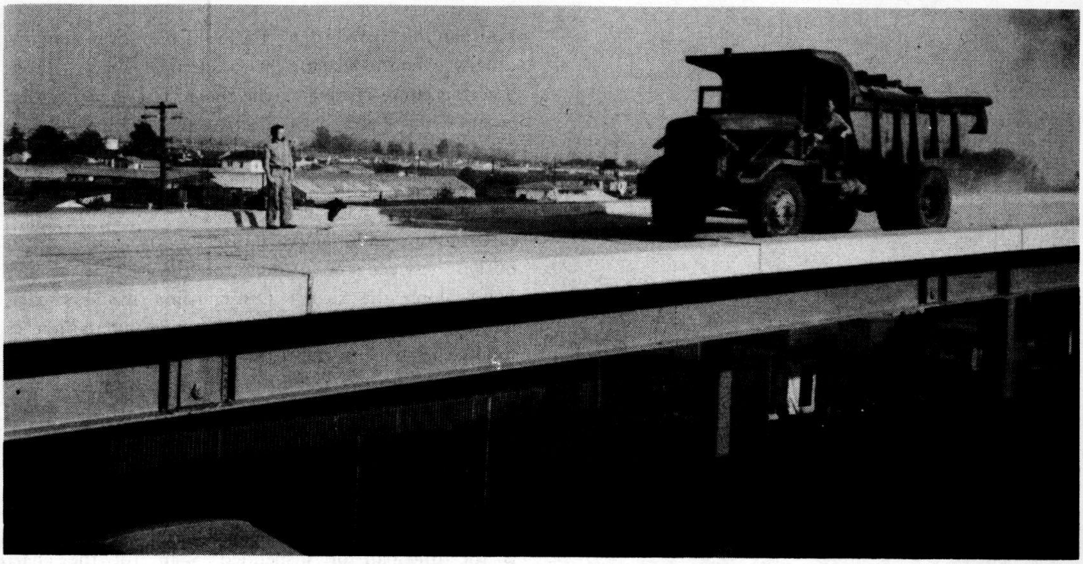


Figure 3. View of 67,000-lb. Euclid test vehicle on completed bridge.

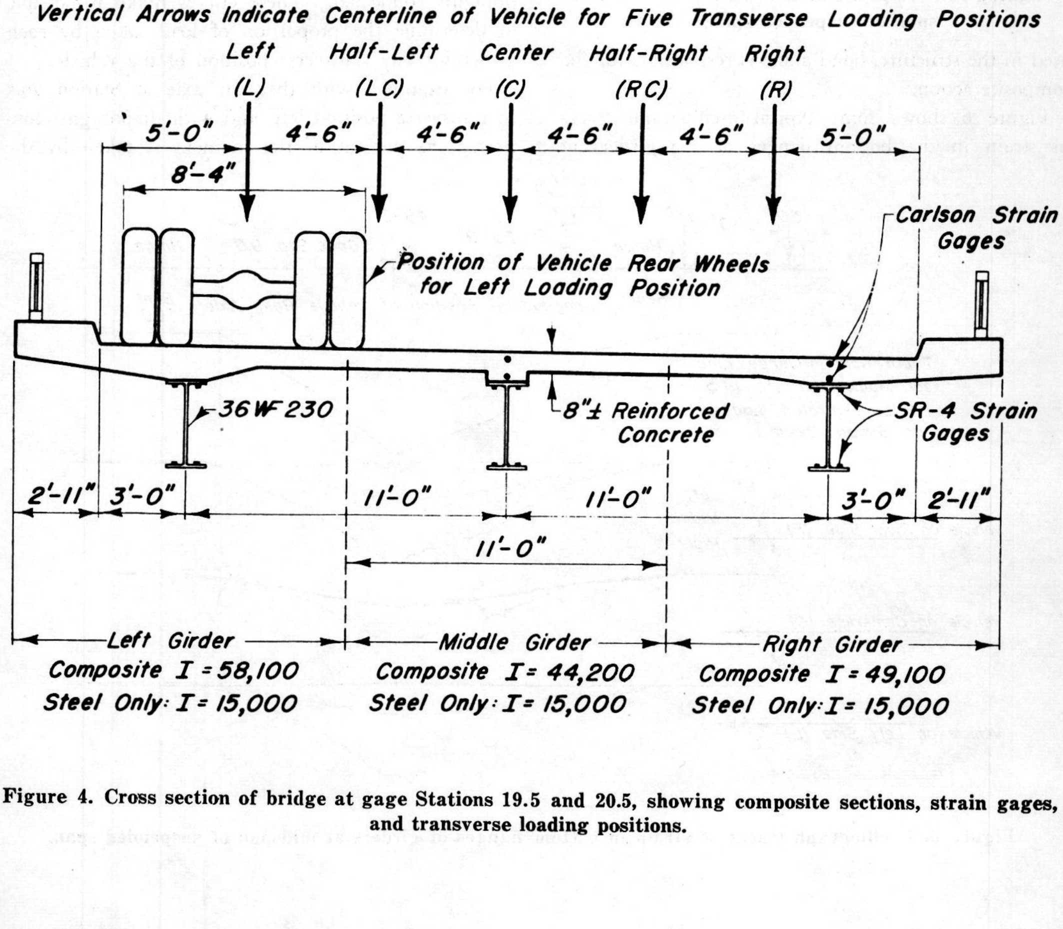


Figure 4. Cross section of bridge at gage Stations 19.5 and 20.5, showing composite sections, strain gages, and transverse loading positions.

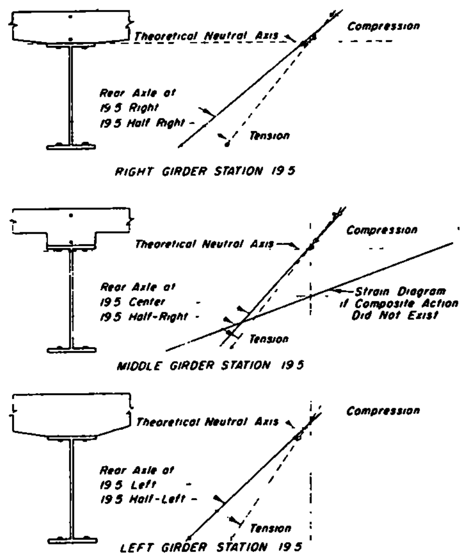


Figure 5. Representative cross sections of strain, indicating full composite action in all girders at midspan of suspended span.

used in the structure, bond alone is responsible for the composite action.

Figure 6 shows some typical oscillograph traces of strain in the bottom flanges of the girders at midspan of Span 19 as the vehicle moves longitudinally over the structure at a speed of about 3 mph The top curve represents the theoretical influence line of moment or strain for the two axle vehicle. Below this are the recorded traces of strain for each of the three girders in three transverse positions, right, center, and left Each group of traces gives the strain distribution and hence indirectly the load distribution between the girders for the vehicle at any point along the span. Disregarding minor oscillations, it will be noted that all the experimental curves follow the shape of the theoretical curve rather closely.

Figure 7 shows the distribution of the total moment among the three girders at two sections of the bridge, when the load is placed in various transverse positions The chart on the left hand side of the figure shows the influence lines for the girders when the rear axle of the vehicle is at Station 19 5, which is the midspan of suspended Span 19, the chart on the right indicates similar data when the rear axle is at Station 20 5, the midspan of continuous Span 20. The solid lines show the distribution with the diaphragms removed, and the dotted lines with diaphragms connected These curves make it possible to determine the proportion of load taken by each girder for any transverse position of the vehicle

For example, with the rear axle at Station 20.5 in transverse position left, and with diaphragms connected, 74 percent of the moment is taken by the

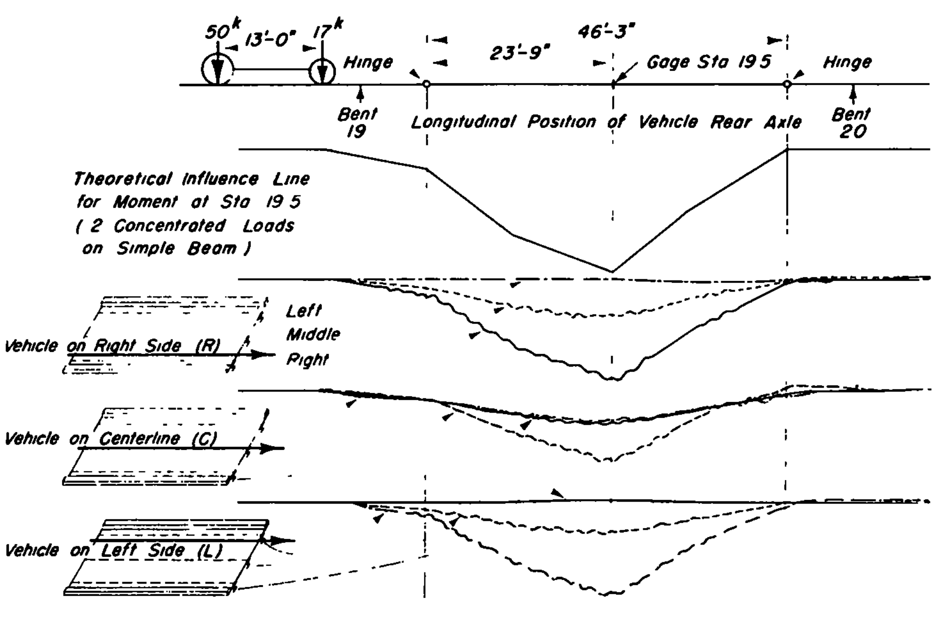


Figure 6. Oscillograph traces of strain in bottom flanges of girders at midspan of suspended span.

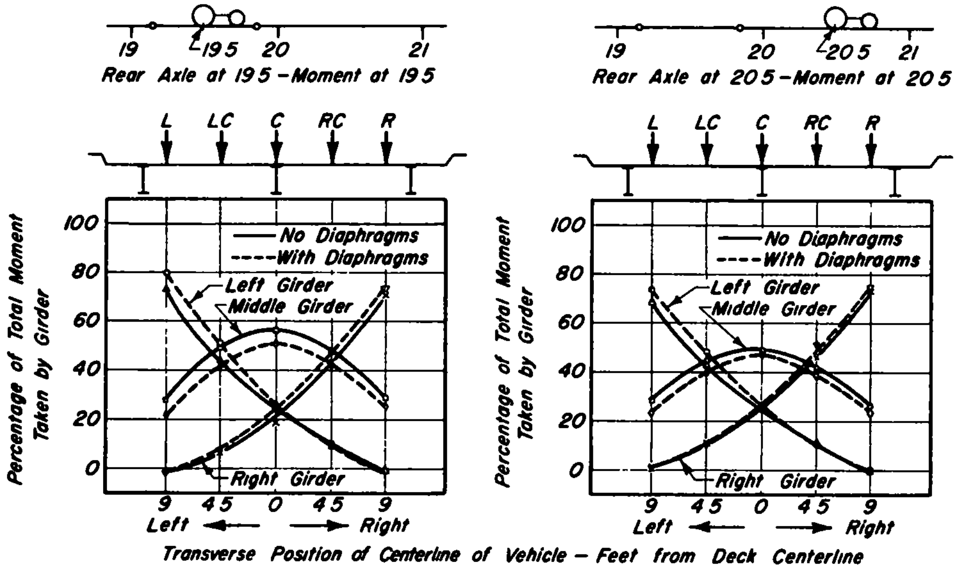


Figure 7. Experimental influence line showing percentage of total moment at section taken by each girder as transverse position varies.

left girder, 25 percent by the middle girder, and 1 percent by the right girder

It will be noted that the effect of the diaphragms on load distribution is rather small This is probably due to the fact that in this bridge the diaphragms are rather flexible compared to the transverse section of the concrete slab and the large composite section of the longitudinal girders.

In general the influence lines for the girders in the two spans are similar. However note that when the load is over the middle girder in the continuous span, more of the moment is distributed to exterior girders than is the case for the suspended span.

Figure 8 shows a comparison of theoretical and experimental distribution of load between girders when the vehicle is on suspended Span 19 In the chart on the left, experimental values of percentage of total moment taken by the middle girder for different transverse positions of the load are shown by the solid line. The dotted line represents the theoretical percentages computed by the use of Jensen's formulas (*Bulletin No 303*, University of Illinois Engineering Experiment Station). These formulas are not fully applicable to this bridge since the theory assumes a slab supported on three simple girders resting on unyielding end supports In our case, due to the deflections of the supporting cantilever girders and the crossbeams, the hinges settle differentially Thus there exist differential end sags among the girders. Jensen's formulas further assume that the slab is simply supported along the exterior girders. In the actual structure some torsional restraint is evidently exerted on the slab, producing partial fixity at the edges Computations by approximate methods have shown that allowance for both of these conditions will substantially increase the distribution of moment between girders. Points a, b, and c on the chart indicate the change in the peak of the middle girder influence line when (a), end sag, (b), half-fixity and sag, and (c), full fixity and sag, are taken into account It will be noted that, assuming half-fixity (Point b), the theoretical load distribution agrees closely with the experimental data This amount of torsional restraint is probably contributed by the expansion dams and diaphragms at the ends of the suspended span No confirmation of this idea has as yet been made.

Figure 9 shows experimental values of load distribution and stresses compared with values computed by the AASHO method using the Euclid vehicle in place of the standard AASHO truck. With the heavy axle at Station 19.5, transverse vehicle positions causing the largest moments in each of the three girders at this station are shown. For example, without the diaphragms, 0.73 of the total moment caused by the vehicle in the left lane, and 0.07 of that in the right lane are taken by the left girder, resulting in a total maximum moment of 0.80. With diaphragms the

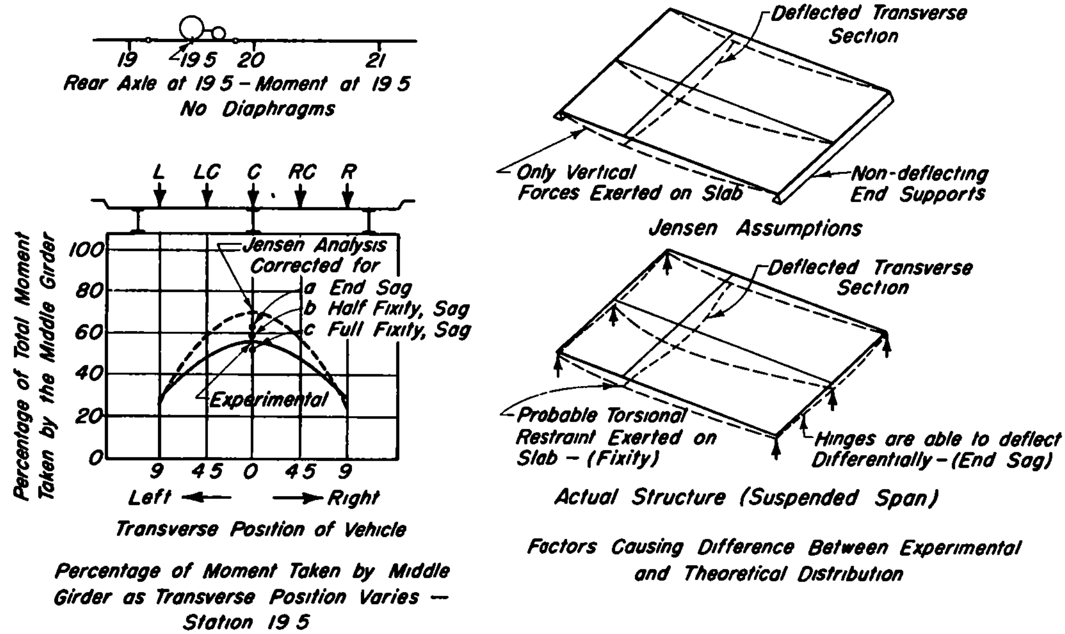
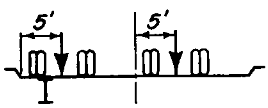


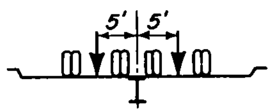
Figure 8. Distribution of moment between girders: Comparison of experimental results with theoretical analysis.

	Portion of Total Moment Taken by Girder for Indicated Vehicle Positions		Total No. of Truck Loads Experimental	AASHO	Max. Steel Stress - p.s.i. Experimental*	AASHO**



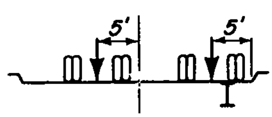
No Diaphragms	.73	.07	.80	.82	4,200	7,900
With Diaphragms	.80	.07	.87		4,600	

LEFT GIRDER — STATION 19.5



No Diaphragms	.47	.47	.94	1.00	5,800	9,600
With Diaphragms	.40	.41	.81		5,000	

MIDDLE GIRDER — STATION 19.5



No Diaphragms	.05	.71	.76	.82	4,400	7,900
With Diaphragms	.06	.74	.80		4,700	

RIGHT GIRDER — STATION 19.5

* Computed by experimental distribution and composite section modulus
**Computed by AASHO distribution and steel section modulus.

Figure 9. Comparison of experimental maximum truck loading and stresses with AASHO specifications, mid-span of suspended span.

total is 0.87, whereas the AASHO method, assuming simple spans transversely and making no allowance for the diaphragms, yields 0.82. Likewise for the middle girder, the experimental values are 0.94 and 0.81 respectively, while the AASHO value is 1.00.* It will be noted that for this bridge the AASHO values appear to be conservative for the middle girder and agree fairly closely with the experimental values for the exterior girders.

On the right hand side of the table, the experimental values of maximum stress, taking into account both the effect of load distribution and of composite action, are compared with stresses computed by the AASHO method which does not consider composite action. The latter stresses range between 8,000 and 10,000 psi., while the experimental values are between 4,000 and 6,000 psi., or 40 to 50 percent lower.

* Factor of 1.00 is obtained as a result of AASHO Bridge Specification T 15(50), tentative revision adopted December 1950. 1949 Specification 3.3.1 resulted in a factor of 1.09 for the interior girder.

Field work on this project has been virtually completed. It is hoped that a complete report will be available for distribution early in 1953. The project was planned and carried out under the guidance of an advisory committee consisting of R. Archibald and H. R. Angwin of the U. S. Bureau of Public Roads, S. Mitchell, T. E. Stanton, and F. N. Hveem of the California Division of Highways, N. C. Raab of the Division of San Francisco Bay Toll Crossings, H. E. Davis, H. D. Eberhart, R. A. Moyer, T. Y. Lin and R. Horonjeff of the University of California, and G. B. Woodruff, consulting structural engineer, San Francisco. Collection of the basic data was made possible through the cooperation of the Bridge Department of the California Division of Highways, especially the resident engineers, W. C. Names and J. N. Perry, and their staffs. On the Institute staff, R. W. Clough, V. A. Plumb, and C. F. Scheffey contributed a great deal toward the success of the project.