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MICHIGAN
STATE HIGHWAY DEPARTMENT
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FIELD CHECK OF DESIGN OF POSITIVE TYPE SHEAR DEVELOPERS

A Cooperative Project Between the Bridge Division and the Research Laboratory of the Testing and Research Division

Research Project 48 F-17

Progress Report No. 1
by

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FIELD CHECK OF DESIGN
OF POSITIVE TYPE SHEAR DEVELOPERS

Highway bridges with composite concrete and steel type superstructures in which the concrete floor slab is tied to the steel stringers by positive shear transfer devices have been constructed in recent years in various parts of the United States, particularly in the East. The use of this type in the middle western states, however, is not too common.

At present day construction prices in Michigan, composite construction has considerable economic advantage for beam spans over 50' in length or in locations where thin superstructures are necessary because of underclearance requirements. It is anticipated that additional stiffness is also obtained by the use of the composite type.

Of the various shear developers which have been proposed, the type selected for use by the Michigan State Highway Department is the so-called "Spiral" Shear Developer, consisting of a plain reinforcing bar rolled into a helix with the loops of proper size and spacing to develop proper shear between the concrete slab and the beam. The bar is welded to the steel beams in the field and is incorporated into the slab, together with the regular reinforcing steel, when the deck is poured. Other types of positive shear developers have been found by others to satisfactorily tie the slab and beams together, but the Department has so far used only the prefabricated "spirals" believing them more easily installed, more economical and more effective. This policy may change, however, with varying conditions.

The Problem:

The Bridge Division of the Highway Department has designed a few bridges with composite decks using spiral shear developers and were proposing to design more. However, it was felt that, even though experiments have been made by others on this type of construction, before proceeding too far, it was advisable to conduct loading tests on such a bridge actually completed. These tests would more or less conclusively establish the safety and adequacy of our methods of stress computation and furnish a sound basis for proceeding with the use of the composite type construction. Accordingly a structure was selected for study and the Research Laboratory of the Testing and Research Division was enlisted to perform the actual tests.

Equipment for the Study

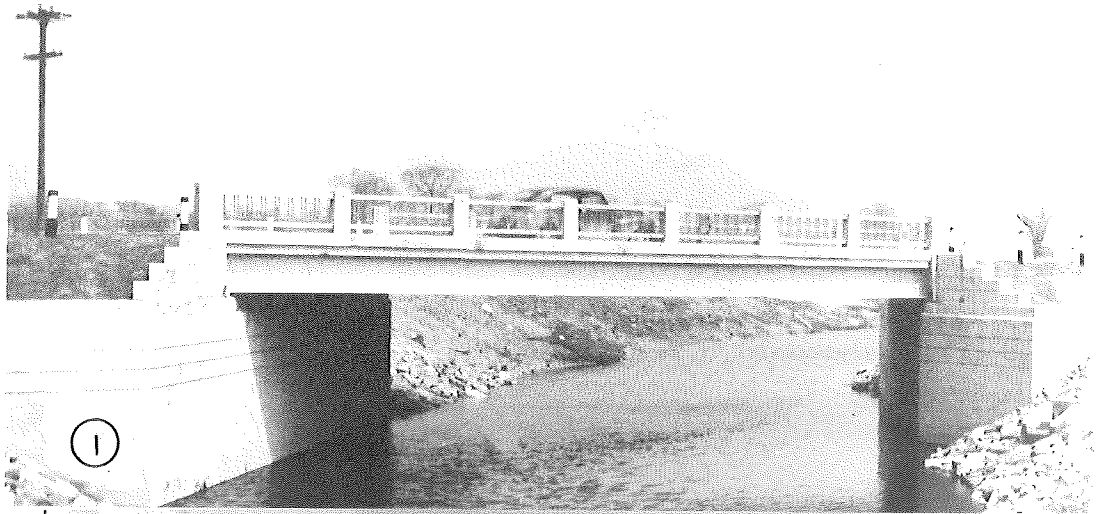
The bridge chosen for the investigation is located on Highway M-46 over the Black River near Carsonville, and is designated by the Department as B1 of 74-3-2. It is a skew structure as one may see in Figure 1. This fact somewhat complicated the computations but made little difference in the field work of the experimental study.

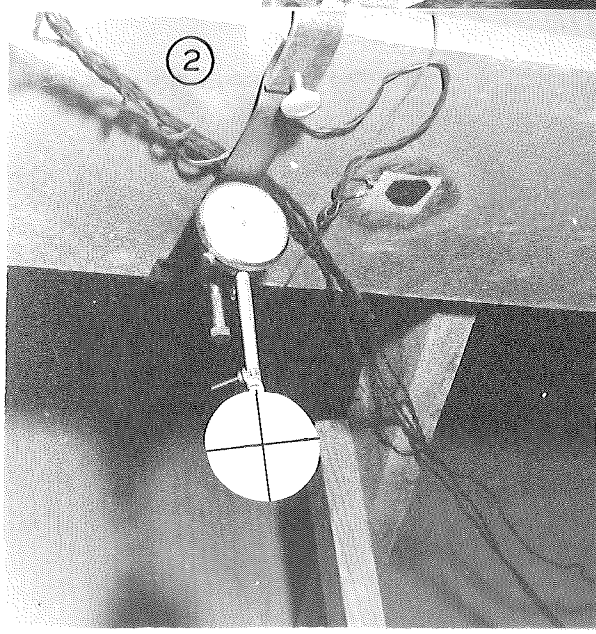
The measurement of strain presented no problem. The Baldwin Southwark SR-4 electric strain gage is a familiar tool and the Portable Strain Indicator is a permanent piece of equipment in the Research Laboratory.

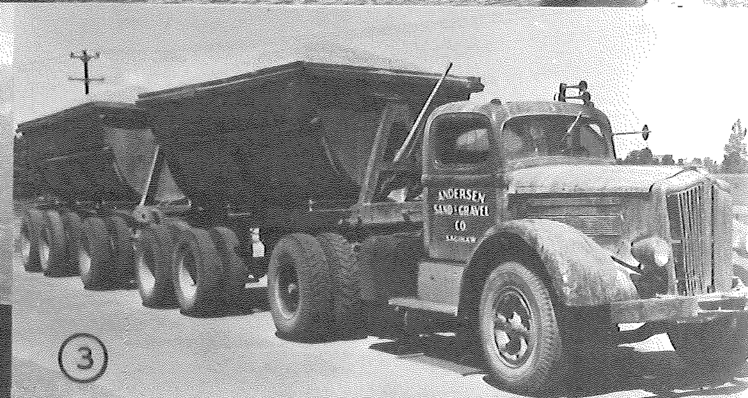
The method of determining deflection required some development. It was known that beam deflections would be relatively small and the engineers level rod would not give results of sufficient accuracy to be of much value. It was impractical to support indicator dials from a beam or from posts driven into the river bed.

The apparatus finally constructed is shown in Figure 2. It consists of a bracket with a movable head. An indicator dial to which a target is attached is fastened to this head, and the bracket is clamped to the lower flange of the bridge beam. The indicator show rests against the beam. Movement between the dial and the beam is accomplished by means of a screw.

In order to use this deflection measuring equipment, a precise level







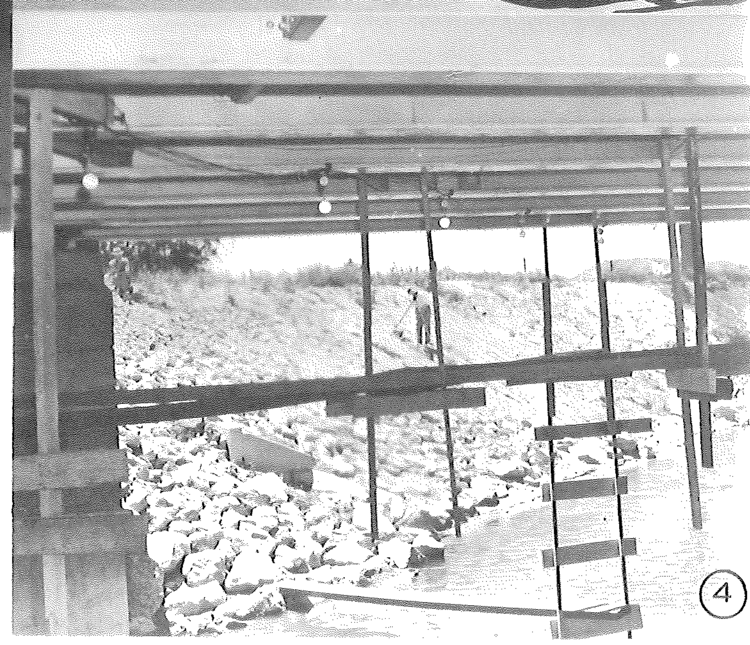


FIG. 1 – BRIDGE SELECTED FOR TEST OF EFFECTIVENESS OF SPIRAL SHEAR DEVELOPERS.

FIG. 2 – DEVICE DEVELOPED FOR DEFLECTION MEASUREMENT.

FIG. 3 – AGGREGATE TRAIN USED FOR LOADING THE BRIDGE.

FIG. 4 – VIEW BENEATH BRIDGE SHOWING SCAFFOLD AND MEASURING DEVICES.

is set up on the river bank in such a position that the line of sight falls within the working range of the device. The target is brought to line and an initial reading taken. The bridge is then loaded, the target again brought to line, and a final reading taken. The difference in readings is the deflection for that load.

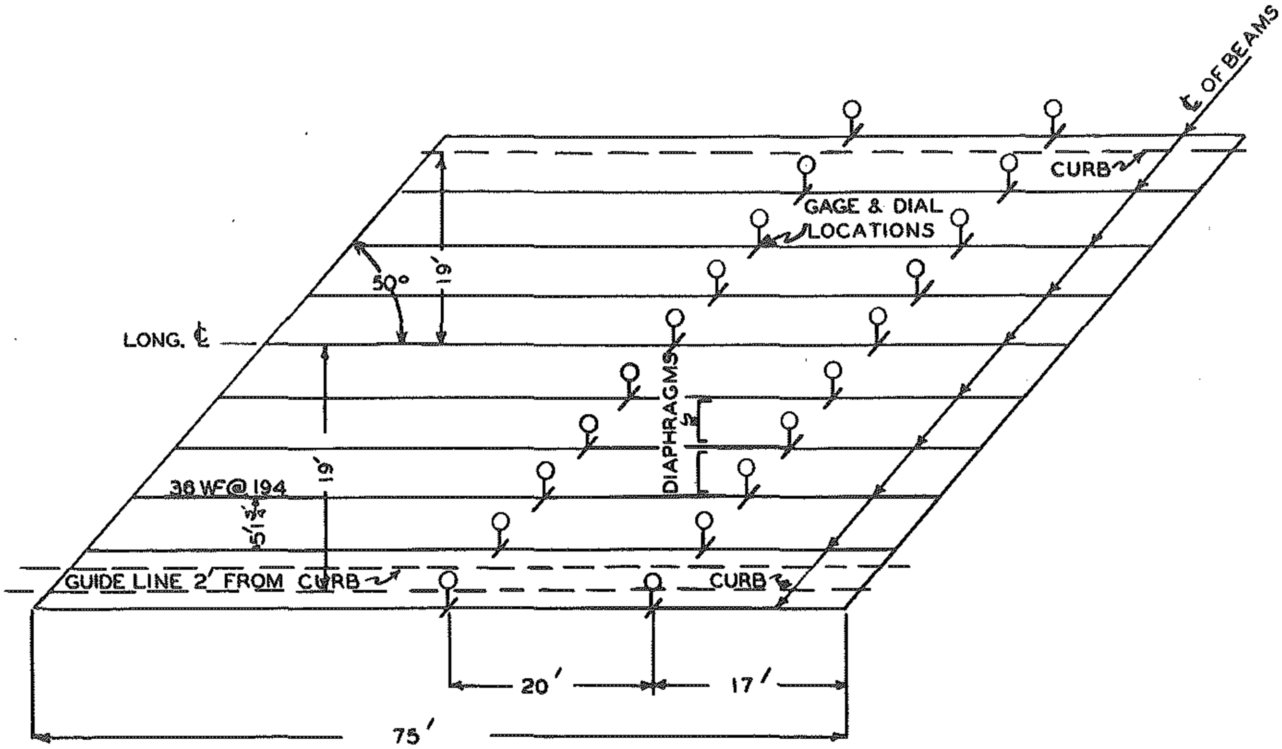
The accuracy of this apparatus was checked by making several series of readings. A variation of five thousand inches or less was recorded, so for this test results were rounded off to the nearest hundredth inch.

Loads were applied on the bridge deck by a special aggregate train consisting of a tractor, semi-trailer, and full trailer. The maximum load was about fifty tons when the full train was used and this was reduced to 25 tons for edge tests by uncoupling the trailers. A view of this equipment is given in Figure 3.

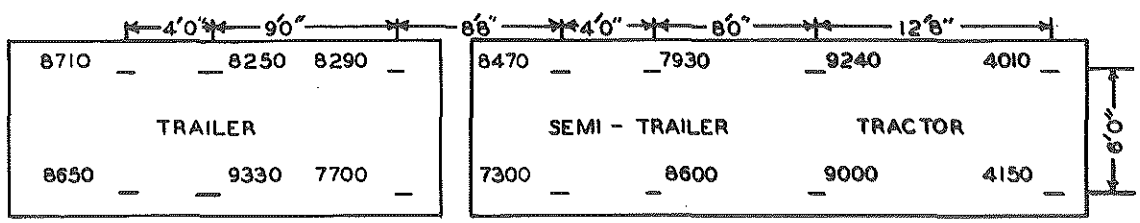
Test Procedure

Arrangements were made with the Saginaw office of the Maintenance Division for the construction of a scaffold beneath the bridge. This framework supported walks beneath the East quarter line and the lateral center line. From these walks an operator could easily cement strain gages to the bottoms of the beams and attach the deflection measuring assemblies. Figure 4 is a picture of a section of the bridge showing the scaffold and the measuring equipment.

The test routine was as follows: One operator was stationed on the river bank with the precise level, one was on the bridge deck with the strain indicator and switching equipment, and a third was on the catwalk under the bridge. While the first and third operators were bringing the deflection reading assemblies to line of sight, the second man was taking initial readings on the strain gages. At the completion of these readings, the load was carefully spotted onto the bridge deck and the reading process was repeated.



SPACING OF BRIDGE BEAMS



DISTRIBUTION OF LOAD UNDER TRUCK TRAIN

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FIGURE 5

Final readings were again taken after the load was moved off from the bridge.

The exact positions of the loads, the gages, and the deflection dials are shown on the attached data. The weight on each wheel of the loading train is indicated as well as the spacing. Since this was a nine beam bridge and readings were made at the quarter and center points of the beams, eighteen deflection devices were used. Two strain gages were attached to the bottom of the diaphragms in addition to the two gages on each beam. The total of SR-4 gages was twenty.

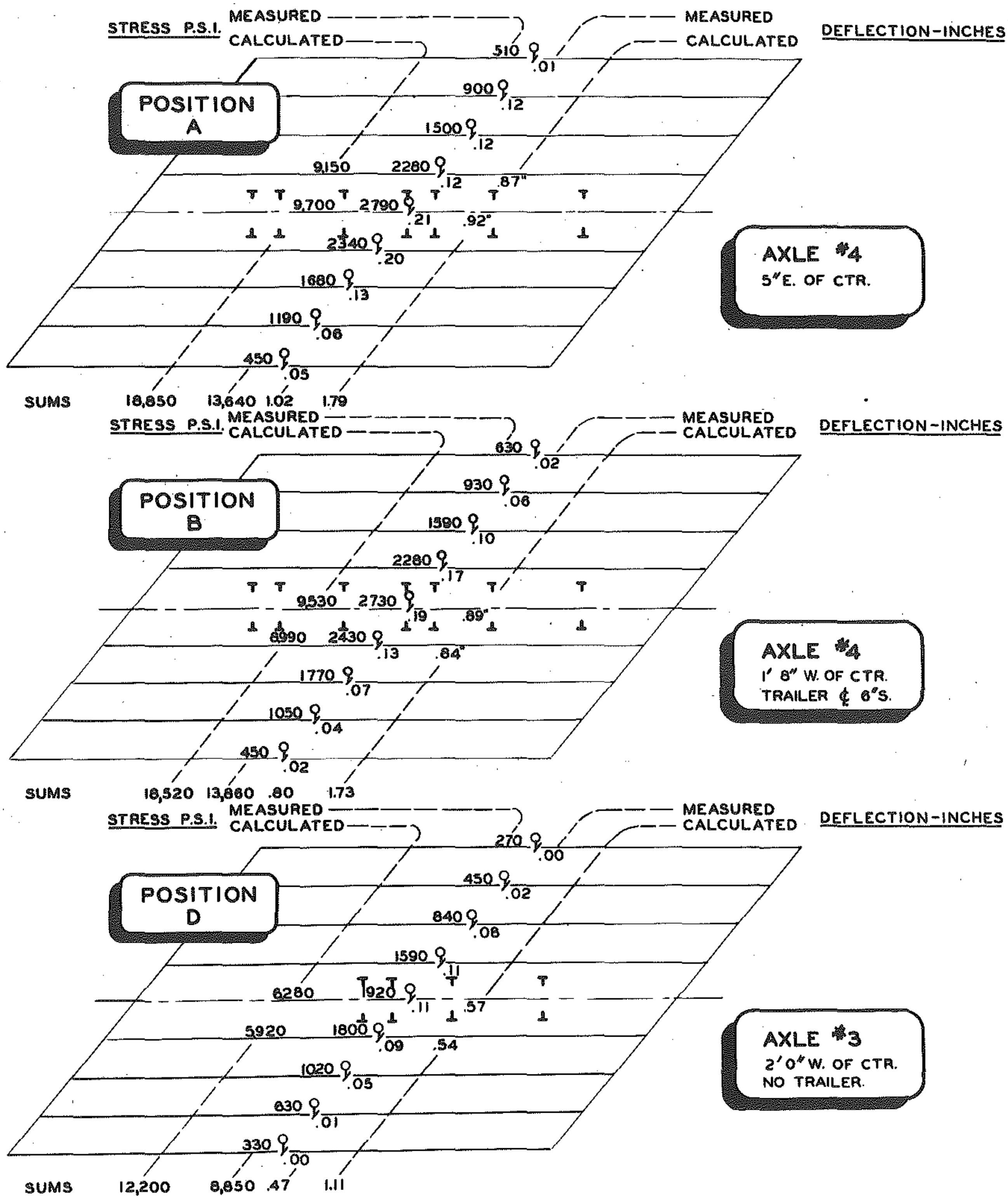
Presentation of Data

Figure 5 is a line diagram of the bridge and loading plan. The whole truck-trailer system was used along the longitudinal center line, but at the edges the trailer was uncoupled and the bridge was loaded with the tractor and semi-trailer only. The truck-trailer loading arrangement produced a load moment of 1,357,000 foot pounds for this span. The corresponding moment for H20-S16-44 loading of the AASHO is 1,004,000 foot pounds. The ratio of these values is 1.3.

Figure 6 is a group of drawings of the bridge members upon which are the measured and computed values of stresses and deflections. Each plan and associated data represents one test. These data are grouped and presented in tabular form in Table I.

Columns 2 to 6, inclusive, for both deflections and stresses as shown in the table, are self-explanatory. The columns marked "l", however, require a few explanatory remarks. In determining the maximum calculated stress for one beam the lane load moment was distributed in accordance with the 1936 Edition of the Michigan State Highway Design Specifications, namely:

$$M' = .514\ M \text{ (As applied to this structure)}$$



STRESS *and* DEFLECTION DATA

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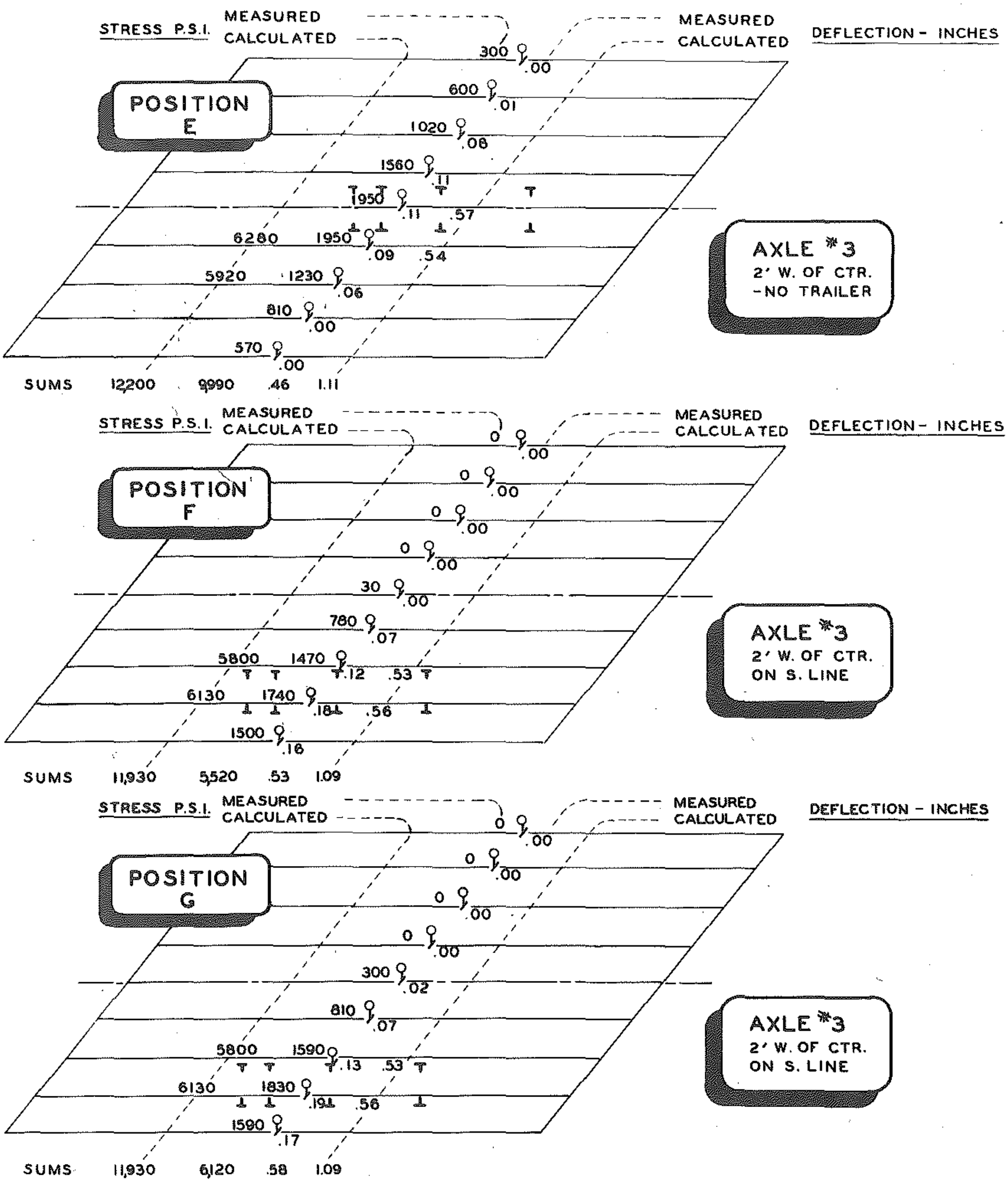
FIGURE 6



STRESS *and* DEFLECTION DATA

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FIGURE 8

TABLE I

A Comparison of Calculated and Measured

Values of Deflection and Stress at Mid-Span

Load Positions	Live Load Deflection at Mid-Span: 1. Calc. Defl. for one beam	2. Meas. defl. of beam carrying heaviest load	3. Ratio of meas. to calc. defl. in percent	4. Sum of calc. defl. for total beams assumed to carry load	5. Sum of Meas. defl.	6. Ratio of sum of meas. defl. to sum of calc. defl. in percent	Live Load Stress at Mid-Span: 1. Calc. stress for one beam	2. Meas. stress for beam carrying heaviest load	3. Ratio of meas. to Calc. stress in percent	4. Sum of Calc. stress for total beams assumed to carry load	5. Sum of Meas. stress	6. Ratio of Sum of meas. stress to sum of calc. stress in percent
A	.92"	.21"	23	1.79"	1.02"	57	9,700 psi	2,790 psi	29	18,850 psi	13,640 psi	72
B	.89	.19	21	1.73	.80	46	9,530 psi	2,730	29	18,520	13,860	75
D	.57	.11	19	1.11	.47	42	6,280	1,920	31	12,200	8,850	73
E	.57	.11	19 Av. Above = 21% 21 X 1.75 = 37%	1.11	.46	41	6,280	1,950	31 Av. Above = 30% 30 X 1.75 = 53%	12,200	9,990	82
F *	.56	.18	32	1.09	.53	49	6,130	1,740	28	11,930	5,520	46
G *	.56	.19	34	1.09	.58	53	6,130	1,830	30	11,930	6,120	51
			Average = 25%			Average = 48%			Average = 30%			Average = 66%

Note:
Distribution factors used in determining calculated deflections and stresses are in accordance with the Michigan State Highway Department's Specifications for the Design of Highway Bridges, 1936 Edition.

* Based on measured stresses and deflections for the 1st interior beam.

Where:

$M' =$ the maximum bending moment to be carried by one stringer.

$M =$ the maximum bending moment for one 10 foot traffic lane.

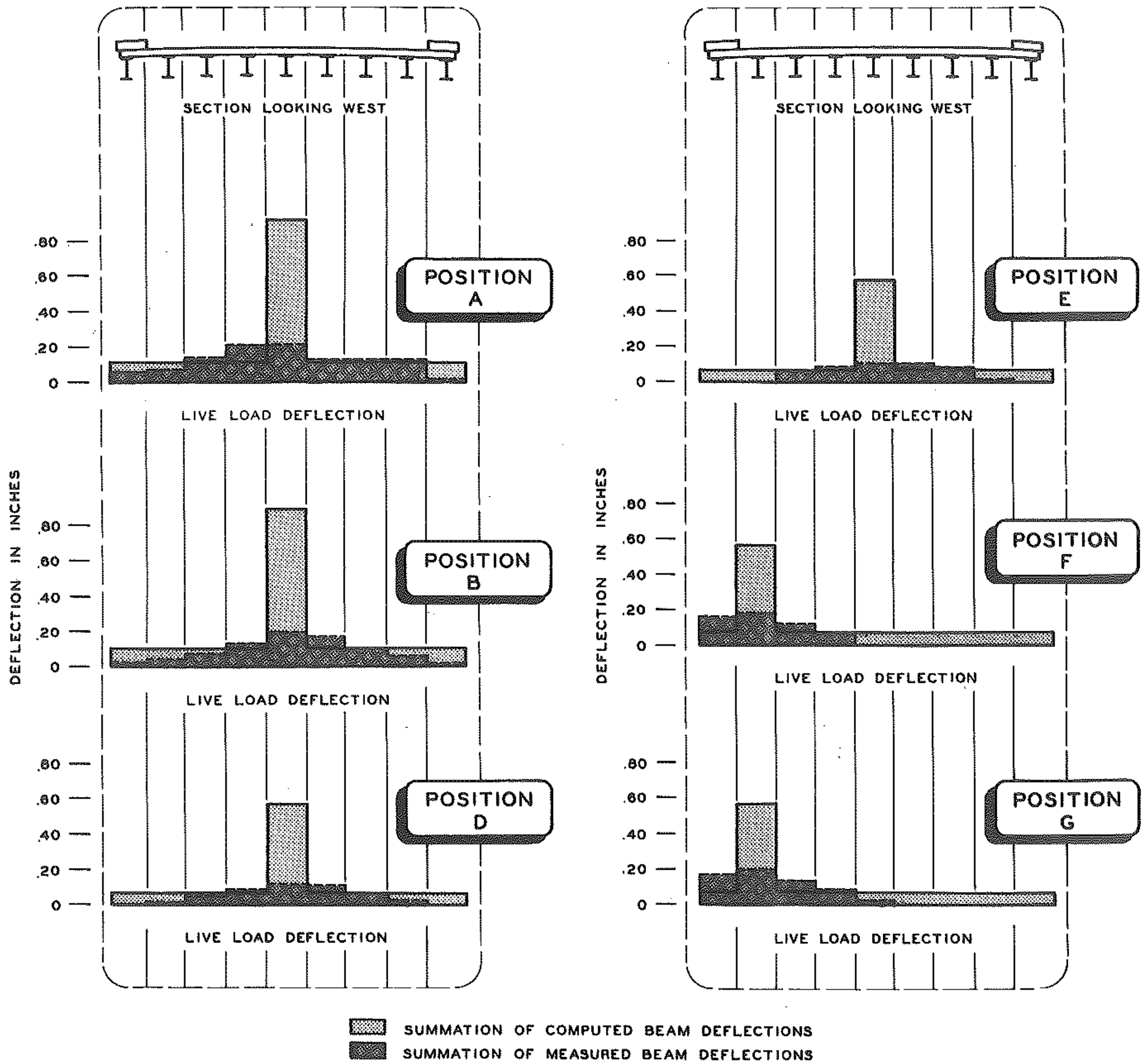
This same distribution was used in calculating the deflections.

As noted above, the maximum moment to be resisted by one stringer in the calculations is 51.4 per cent of the maximum lane load moment. The stress due to that moment was calculated and is shown on one of the beams in the appropriate column on the individual load position sheets. The remaining amount of the stress produced by the lane load, 48.6 per cent, has been shown on the adjacent beam. It is not intended that these calculated stresses apply necessarily to the beams on which they are shown. It is simply to show that, as far as design calculations are concerned, 51.4 per cent of the stress, or deflection, is figured to be taken by one stringer and the balance, 48.6 per cent, is distributed among the remaining stringers in some fashion. The summation columns "5" were prepared with the thought in mind that the ratio of the sums of the calculated to measured values for all of the beams for any one position of loading would eliminate the variable of load distribution among the stringers and would thus afford a more accurate check on the composite design.

In calculating the stresses in the stringers the usual transformed section method was used with the ratio of the modulus of elasticity of concrete to that of steel equal to ten for both stress and deflection computations.

In order to portray graphically the relation between computed and observed deflections in the light of the foregoing explanation, a series of area diagrams have been constructed (Figure 7).

The hatched areas of the diagrams indicate the summations of the deflections which calculations show that the beams must take to support the load,



COMPARISON of CALCULATED and MEASURED DEFLECTIONS

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FIGURE 7

while the shaded portions show the deflections actually taken by the beams according to field measurements.

The measured values of the deflections of each beam have been plotted as ordinates directly under the respective beams as determined in the field, but it is realized that the calculated values are not actually distributed to the various beams as they are shown. Therefore, as far as the calculated values are concerned, the diagrams must be used for the structure as a whole and cannot be used to determine the deflection of any particular beam.

Analysis and Conclusion.

In analyzing the results as shown in Table I it will be noted that the average ratio for the various positions of loading of the summation of the measured deflections to the summation of the calculated deflections is 48 per cent and that the corresponding ratio for stress is 66 per cent. These ratios seem to indicate conclusively that the composite action assumed is being realized and that assumptions regarding the moduli of elasticity are on the safe side.

It may be also observed from a study of the table that the ratios of maximum measured to maximum calculated deflections vary from 19 to 34 per cent for the different load positions and similar ratios for stress vary from 28 to 31 per cent. It will be noted, however, that for positions A, B, D, and E the wheel lines were not directly over the stringers and, therefore, some increase of the above ratios for those positions of loading might be obtained by shifting the applied load transversely. It does not seem probable, however, that such a shift would increase the percentages by more than 75 per cent. If this percentage is assumed it would bring the average of the measured to calculated maximum deflections for these four positions to 37 per cent, and the comparable average ratio for stress to 53 per cent. The fact that even the adjusted values of the ratios of measured to calculated deflections and stress are less than eight-tenths of the corresponding ratios when the summations are used, indicates that the loads are being distributed transversely among the stringers by the diaphragms and slab to a considerably greater extent than our usual load distribution formulae indicate.

The analysis of the results of this experiment has indicated that when additional experiments of this nature are performed, it would be well to include a set of measurements for multiple lane loading. Such a test is now being planned by the Department in connection with the proposed Kalamazoo River Bridge east of Fennville. It is obvious from a study of these measurements that the maximum stringer stress will be experienced with the bridge loaded with more than one lane and, while the summation of all the stringer stresses are a check on the composite design, multiple lane loading would, nevertheless, eliminate some of the uncertainties. Also it would be well to so place the lanes transversely so as to secure the maximum stress in one of the stringer lines.

Summarizing, it appears that two facts are brought out by this experiment: (1) That our design assumptions with regard to the composite action are safe, and, (2) That considerably more transverse distribution is obtained through the slab and diaphragms than is customarily calculated.