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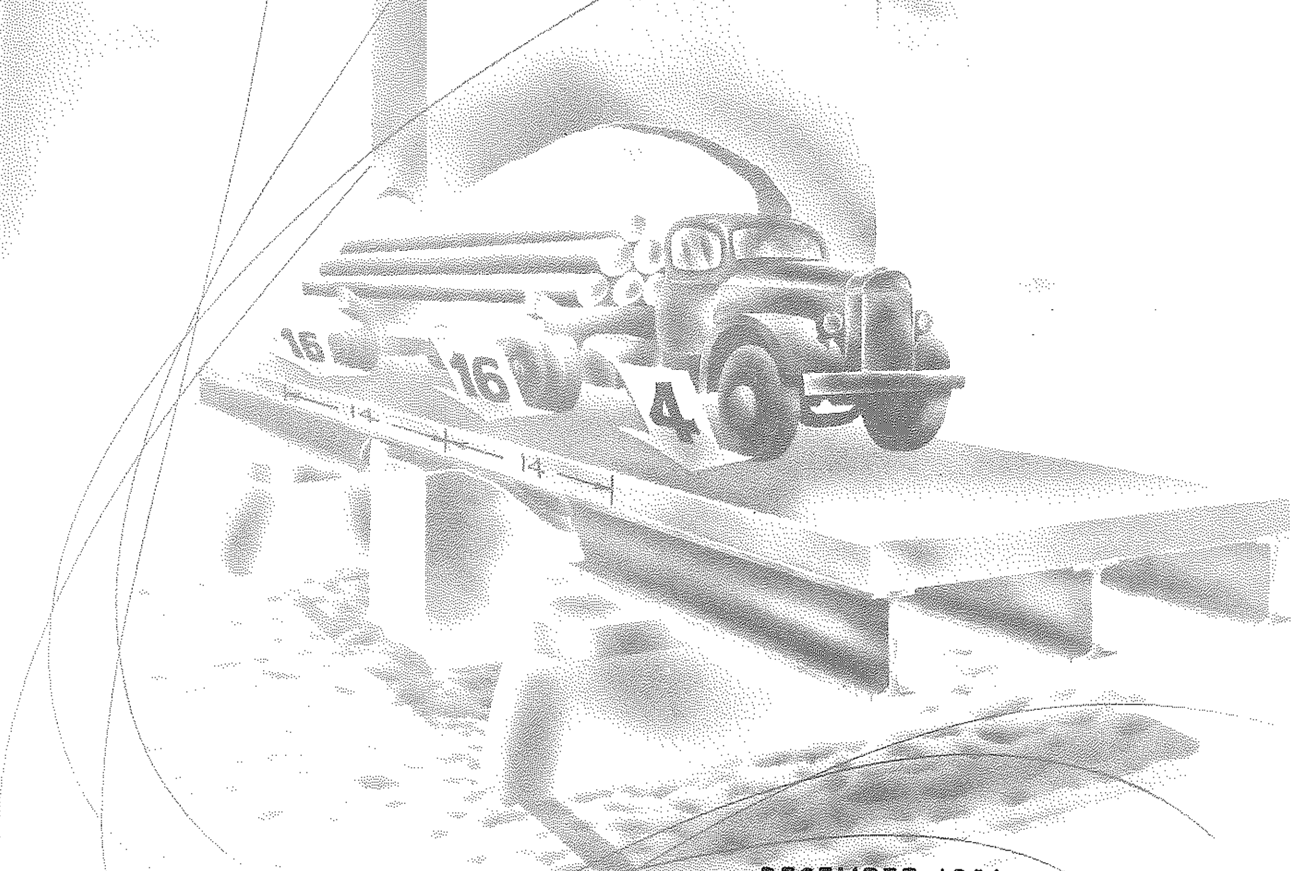
MICHIGAN TEST on ROLLED BEAM BRIDGE using H 20 - S 16 LOADING



DECEMBER 1951

MICHIGAN
STATE HIGHWAY DEPARTMENT

CHARLES M. ZIEGLER
STATE HIGHWAY COMMISSIONER

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State Highway Commissioner

MICHIGAN TEST ON ROLLED BEAM BRIDGE USING H20-S16 LOADING

By

G. M. Foster, Chief Deputy Commissioner

Cooperative Research Project between
Bridge Division and Testing and Research Division
of Michigan State Highway Department

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Progress Report

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MICHIGAN TEST ON ROLLED BEAM BRIDGE USING H20-S16 LOADING

In order to continue an investigation of the effectiveness of shear developers and to study certain lateral distribution features in bridge construction, the Bridge Engineer of the Michigan State Highway Department, in consultation with W. W. McLaughlin, Testing and Research Engineer, proposed a testing program on a six-span bridge near Fennville, Michigan.

The general program was set up by E. A. Finney, Assistant Testing and Research Engineer in charge of Research. Suggestions for the testing of certain features were made by G. S. Vincent, Bureau of Public Roads; T. Y. Lin, Institute of Transportation and Traffic Engineering, University of California; E. C. Hartman, Aluminum Research Laboratories; C. T. G. Looney, Yale University; G. B. Woodruff of Woodruff and Samson, Engineers, San Francisco; H. E. Hilts, Bureau of Public Roads, and others. Aids in testing methods were obtained from reports on (1) the San Leandro Creek Bridge, Oakland, California, and (2) the Paramata Bridge in New Zealand.

The field tests were supervised by L. D. Childs, Physical Research Engineer. M. Rothstein, Bridge Design Engineer, analyzed the data. C. B. Milroy, Bridge Project Engineer, worked directly with the test crew in the field and expedited the work. V. J. Spagnuolo, Phyiscal Testing Engineer, supervised the operation and maintenance of the recording equipment.

This report is a record of the progress to date. Testing of the structure will continue with a more detailed study of impact and vibration effects from rapidly moving vehicles.

OBJECTIVES OF THE TEST PROGRAM

The general purpose of the investigation was to obtain stress and deflection data which could be correlated with theoretical values to accomplish efficiency and economy in the design of highway bridges. The information will also be used in a study of the live load-carrying capacity of existing highway structures under loads imposed upon them by present day heavy motor transport units.

The specific objectives of the test program as proposed in the original outline were as follows:

1. To determine the stress distribution in the girder system under static, dynamic, and impact loading.
2. To study the effect of diaphragm connection and method of spacing upon lateral distribution of loads.
3. To measure the degree to which the concrete deck slab influences stress distribution to supporting members.
4. To observe the differences in stress conditions in supporting steel members when deck slabs are anchored and unanchored to these members.
5. To check design values with field data.
6. To observe the effects of temperature upon stresses in the structure.
7. To obtain vibration data on spans with different design features.
8. To measure slippage between the deck slab and the supporting beams.
9. To measure the mid-span deflections of spans with different design features and under several load conditions.
10. To attempt to measure lateral stresses in the concrete deck both by surface gages and by gages attached to the reinforcing steel.

Although the specific objectives were not achieved in their entirety due to limitations of equipment, some data was obtained for each phase of the study. A continuation of the tests should supply sufficient additional information to fully accomplish all of the objectives.

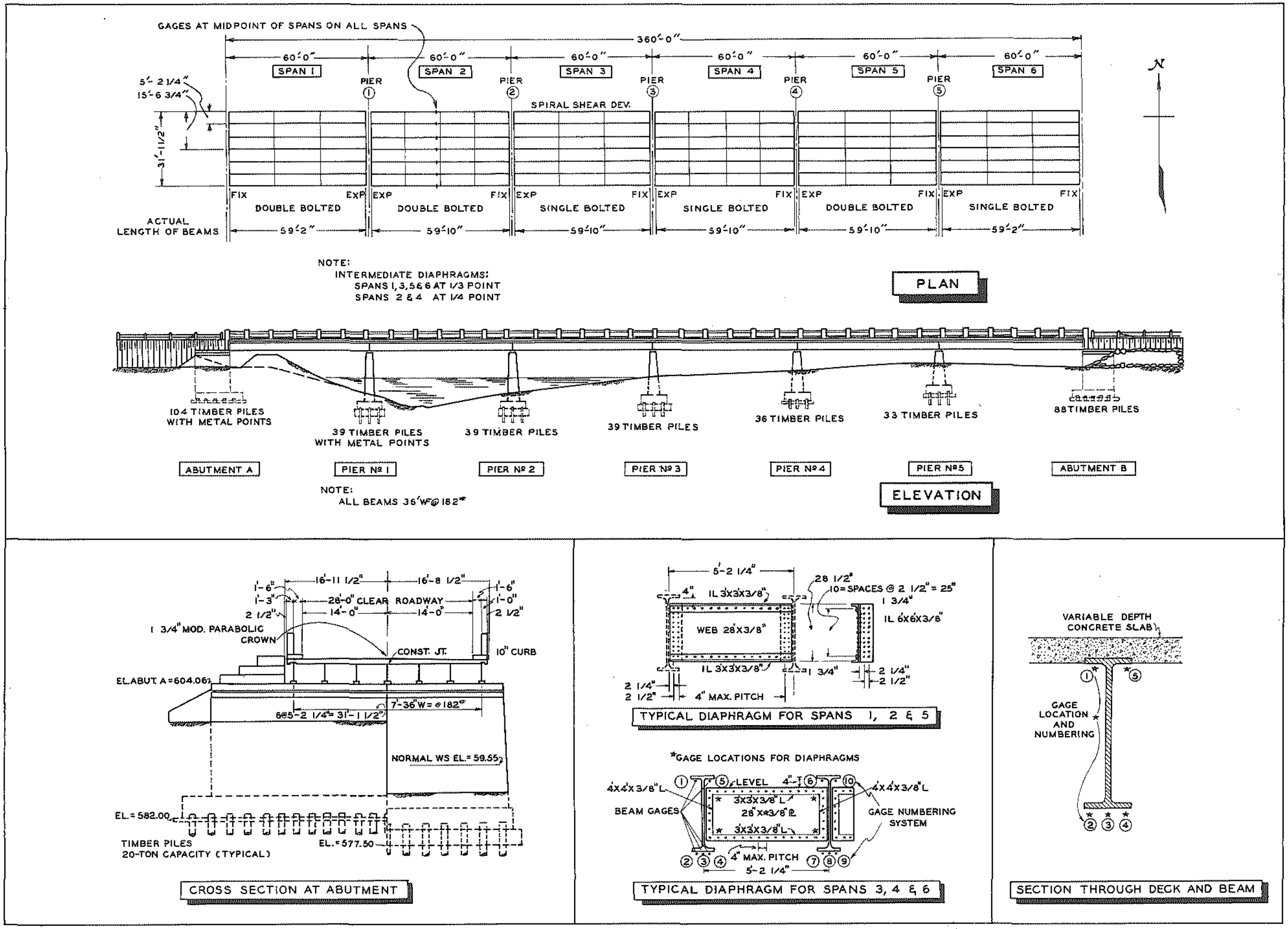


FIGURE 1. FUNDAMENTAL DETAILS OF STRUCTURE

THE STRUCTURE

Fundamental dimensions of the structure are given on the plan in Figure 1. The bridge consists of six simple spans, each nominally 60 feet in length with an overall deck width of 33 ft. 8 in. and a 90-degree angle of crossing. The deck is constructed of reinforced concrete with variable slab thickness to provide the required crown at the center and to allow for dead load deflection of the beams. The deck is reinforced transversely with 5/8 in. deformed bars at 6-in. centers top and bottom. It is supported by seven lines of 36-in. W. F. 182-lb. rolled beams spaced 5 ft. 2¼ in. on centers.

The six spans are alike except for the following features:

Span 1 - West end of beams embedded in concrete backwall; two rows of diaphragms double-bolted to beams; actual span length from center to center of bearings is 58 ft. 5 in.

Span 2 - Three rows of diaphragms double-bolted. Span length 59 ft. 3 in.

Span 3 - Composite construction using spiral shear developers. Two rows of diaphragms single-bolted. Span length 59 ft. 3 in.

Span 4 - Three rows of diaphragms single-bolted. Span length 59 ft. 3 in.

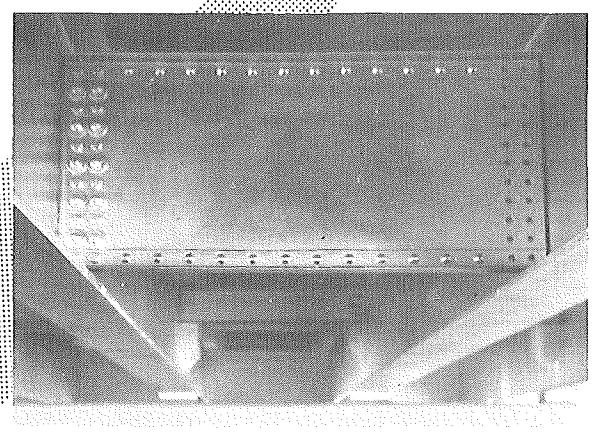
Span 5 - Two rows of diaphragms. This span tested under three conditions; (a) with no diaphragm connections, (b) single-bolted, and (c) double-bolted. Span length 59 ft. 3 in.

Span 6 - Two rows of diaphragms single-bolted. The east ends of the beams are embedded in the backwall. Span length 58 ft. 5 in.

A general view of the bridge at the time of testing is shown in Figure 2. The field program was not begun until the water had subsided to its minimum level. At this stage, Spans 5 and 6 were dry, Spans 1 and 4 extended over water for about half their length, and Spans 2 and 3 were completely over water.

FIGURE 2. GENERAL VIEW OF BRIDGE AT TIME OF TEST

FIGURE 3. DOUBLE BOLTED DIAPHRAGM WITH ONE SIDE UNBOLTED FOR TESTS ON SPAN 5



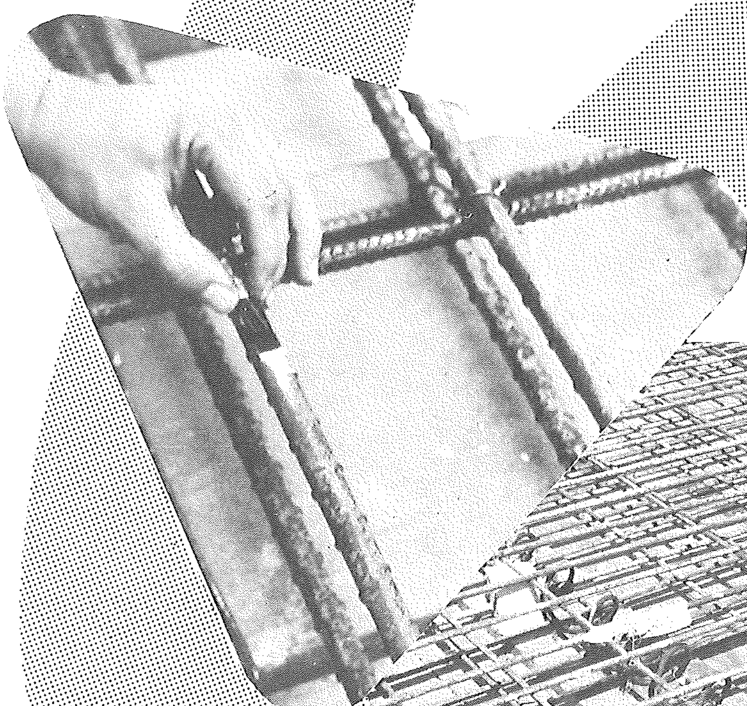


FIGURE 4. REINFORCEMENT DETAILS AND METHOD OF PLACING SR-4 GAGES

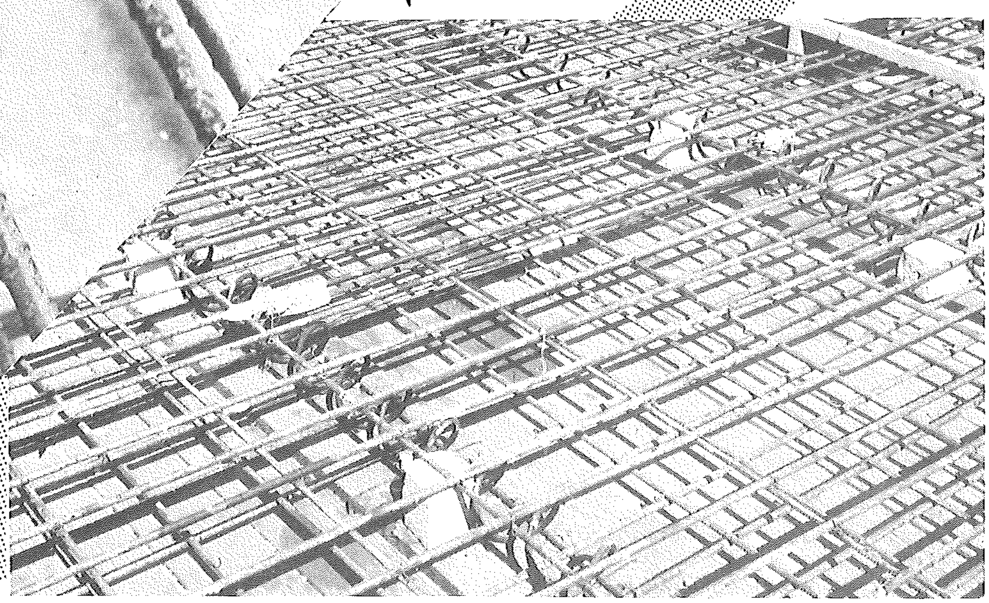


FIGURE 5 SPIRAL SHEAR DEVELOPERS IN REINFORCEMENT FOR SPAN 3

Several design features are illustrated in the accompanying photographs. A double-bolted diaphragm is pictured in Figure 3. Two rows of turned bolts fasten it rigidly to the beam web. In this illustration, the bolts on one side have been removed for the purpose of testing Span 5 under the "no diaphragm" condition.

Figures 4 and 5 exhibit the placement of the reinforcing steel in the deck. Also, in Figure 4, the method of application of the strain gage to the reinforcing steel is shown. The spiral shear developer, which is welded to the tops of the beams of Span 3, may be seen in Figure 5.

TEST EQUIPMENT

Loading Vehicles

A special test vehicle meeting the H20-S16 requirements was constructed by the Maintenance Division. A Walters truck was modified by extending the wheel base to 14 ft. and mounting a fifth wheel directly above the rear axle. A set of outside wheels was added to the rear axle to assure support for the 16-ton load without excessive overload on the tires. A semi-trailer was built with the distance between the truck and trailer axles also equal to 14 ft. The axle lengths were 6 ft. from center to center of wheel on the first and last axle, and 6 ft. 4 in. on the center one. These were sufficiently close to the measurements of the theoretical design vehicle to be used for direct comparison of design and field measured results.

Ballast blocks for loading the axles to the required 4, 16, and 16 tons respectively were made of plain concrete and were 1 ft. x 2 ft. x 4 ft. in size, with a weight of about 1200 lbs. each. They were cast in wood gang molds which were set up on the bridge deck. Before the concrete had set, a small amount of the mix was removed from the top of the block at the center and a "U" shaped piece of reinforcing steel embedded at this point, with the bend flush with the surface. This provided a loop for the crane hook and facilitated handling without interfering with the stacking of the blocks.

Several photographs of the loading equipment are shown. Figure 6 is a view of the test vehicle loaded to meet H20-S16 requirements. Figure 7 exhibits the peculiar arrangement of the ballast necessary to produce proper load distribution. In Figure 8, several features may be seen. In the foreground are the gang molds in which the ballast blocks were cast. Behind these is the crane which loaded the blocks onto the test vehicle. To the right is the vehicle with the two heavy axles resting upon loadometers. Fortunately, the front axle 4-ton requirement was met without the use of ballast on the truck, so four loadometers were sufficient to check the load distribution.

After some testing with the single design vehicle, it was concluded that better results might be obtained with heavier loads. A second design vehicle was not available, but a standing load was readily constructed from beams and blocks. This was placed in the lane adjacent to the one used by the moving truck in such a position as to oroduce maximum bending moment. Figure 9 shows this simulated vehicle and an actual test picture of both vehicles in use is shown in Figure 10.

Measuring Instruments

Strains and deflections were measured at mid-span on all spans. The Baldwin SR-4 bonded strain gage was the heart of the instrumentation. These gages were cemented to the beams' flanges, to the diaphragms, to the bottom of the bridge deck, and on certain lateral reinforcing bars. They were also used on short thin cantilevers to make possible a permanent record of deflections.

The type A-1 gages were used more than any other, although some AR-1 and A-8 gages were used in the diaphragm study, and A-9 gages were cemented to the bottom of the concrete deck in the study of lateral load distribution. Figure 11 is an installation of gages on a diaphragm, and the application of a gage to the reinforcing steel was shown in Figure 4.

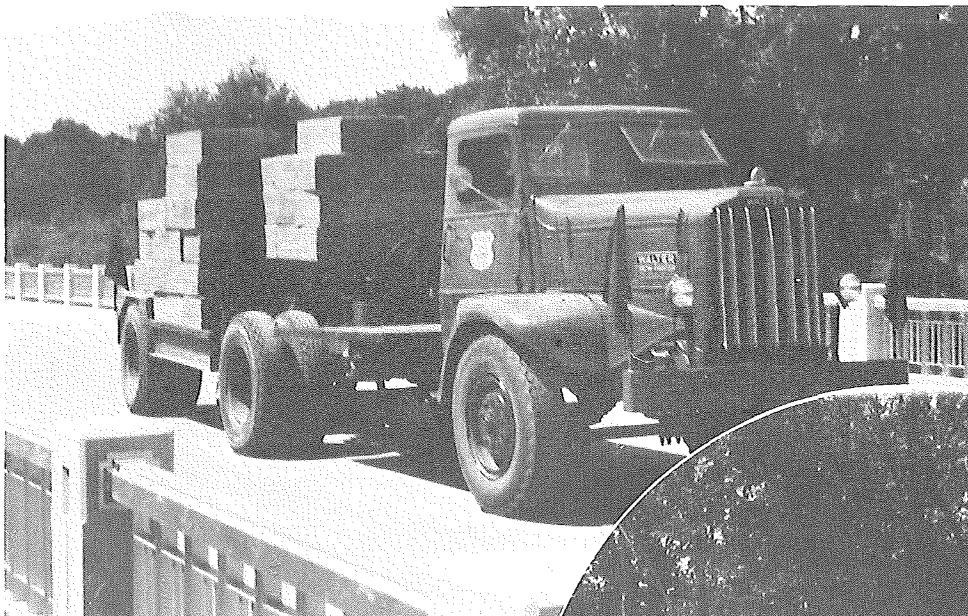


FIGURE 6.
H20-S16 TEST VEHICLE

FIGURE 7. DETAILS OF LOAD DISTRIBUTION TO MEET H20-S16 REQUIREMENTS

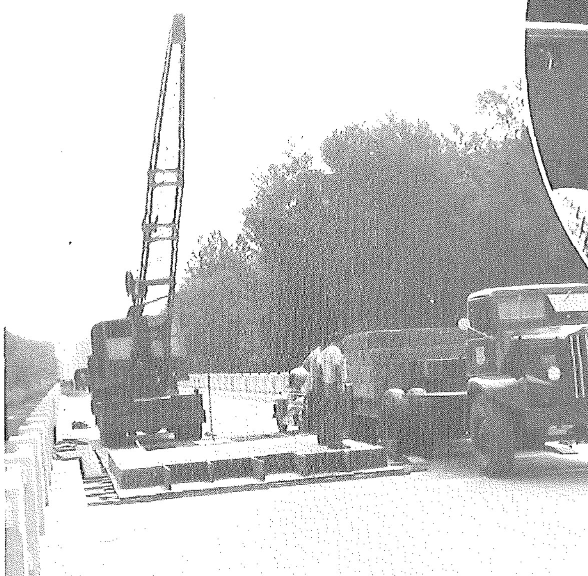
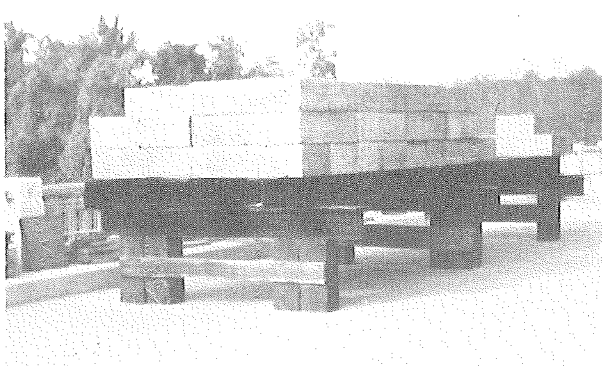
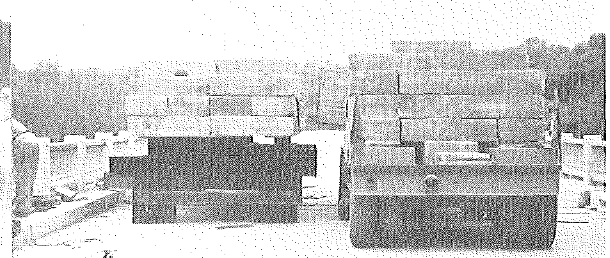


FIGURE 8. MOLDED BALLAST BEING PLACED ON TEST VEHICLE

FIGURE 9. SIMULATED VEHICLE PLACED IN SOUTH LANE

FIGURE 10. METHOD OF OBTAINING TWO-TRUCK LOAD CONDITIONS





Deflectometers were laboratory-built. Figure 12 is an installation on a beam and an accompanying explanatory sketch. The device was constructed in such a way that depressing the beam actuated both a one-thousandth dial and the short cantilever to which the strain gage was attached. The dial permitted visual observation of the deflection and the cantilever transducer provided means of actuating an oscillograph galvanometer to provide a permanent record on sensitized paper. The combination of visual and electric indication made the calibration of the electrical record very simple.

The installation of gages and deflectometers under Span 3 is pictured in Figure 13. At the time this photograph was taken, the static tests had been completed and the wires to the middle gage at the bottom of each beam flange had been clipped. The gage heads were then attached for the dynamic tests. The operator was in the act of setting the deflectometer dials to the initial zero.

The position of the moving truck on the bridge deck was determined by the use of rubber tubes and pneumatic switches. The tubes were stretched across the lane at two locations. The first was at the point where the truck first entered the span and the second was at mid-span. The switches actuated solenoid markers in the oscillograph and formed small pips on the record.

Slippage between the deck and supporting beams was read on dials sensitive to one ten-thousandth inch. A dial mounted for this purpose is pictured in Figure 14.

Recording Devices

Two types of devices were used for recording the test data. For static tests, strains were measured by an SR-4 portable indicator and deflections were read directly from the dials. The indicator and Anderson switching units are seen in Figure 15. When moving load and impact tests were made, both strains and deflections were recorded upon a photo-sensitive paper strip in a Hathaway 12-channel oscillograph. This strain measuring equipment was mounted on shock mounts in a light truck, and is pictured in Figure 16.

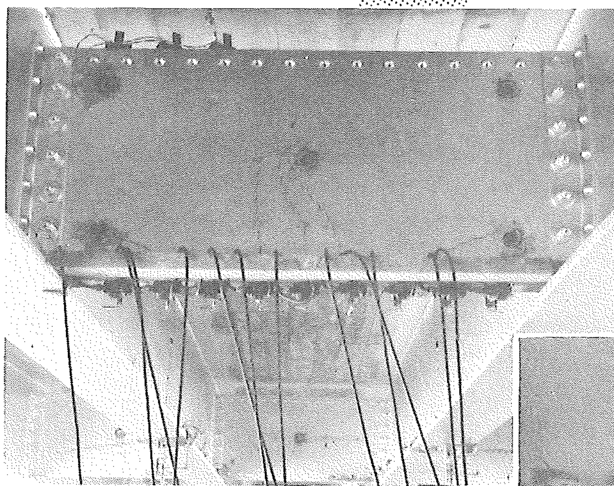
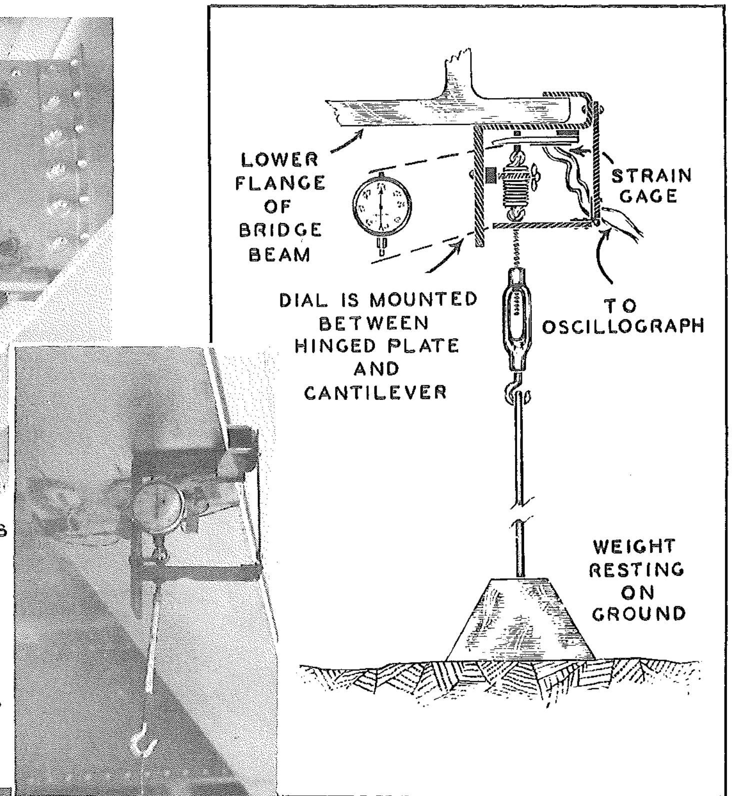


FIGURE 11. ROSETTES ON DIAPHRAGMS WIRED TO HATHAWAY GAGE HEADS

FIGURE 12. DEFLECTOMETER DETAILS



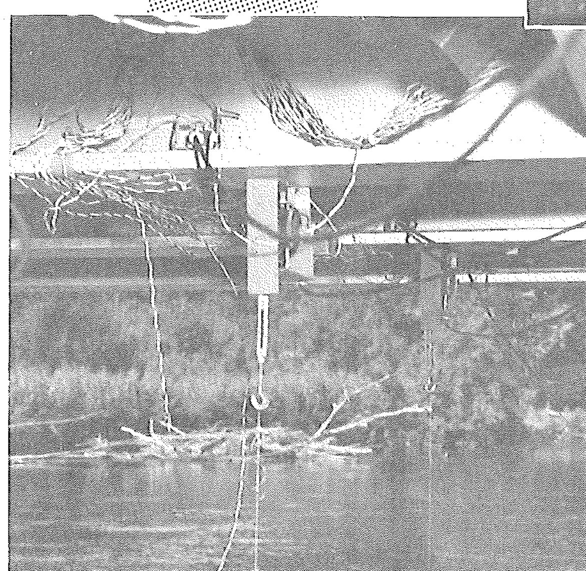
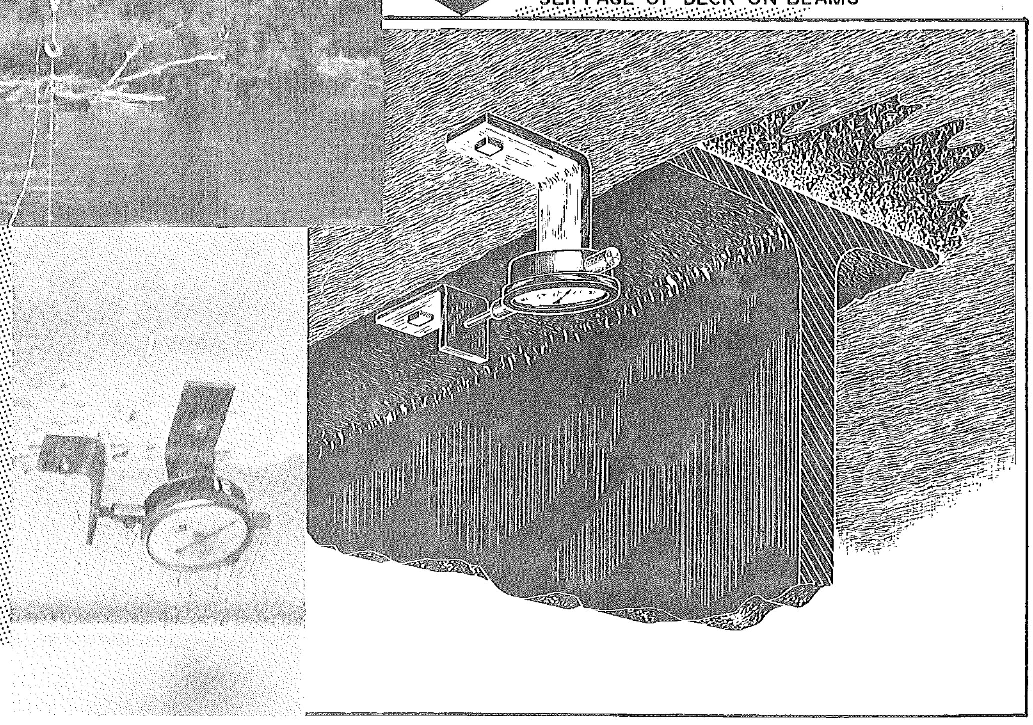


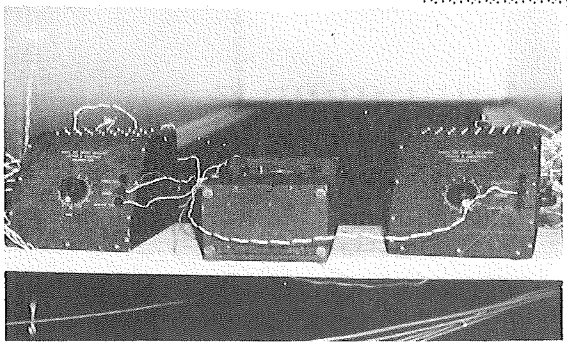
FIGURE 13. INSTALLATION OF GAGES AND DEFLECTOMETERS ON SPAN OVER WATER

FIGURE 14. DIAL INDICATOR FOR MEASUREMENT OF SLIPPAGE OF DECK ON BEAMS



Sample oscillograph records are shown in Figure 17. The vertical lines are timing lines representing one-tenth second intervals. They enable a computer to figure the frequency of oscillation of the span and the speed of the moving vehicle. The pips at the top of the record show the truck wheel positions.

The strains and deflections were determined from the traces in the following manner: the ratio of micro-inches per inch of strain to units of chart deflections was first computed from a calibration record. Then the maximum deviation of each trace from its zero line was multiplied by this factor to obtain maximum recorded strain. By this procedure, the strain magnitude at mid span on the lower surface of each beam was found from the upper seven traces on the record. Deflections were computed in a similar manner from the lower five traces. On Beams 6 and 7, the dial indicator readings were used directly because the recording equipment was limited to a total of 12 channels.



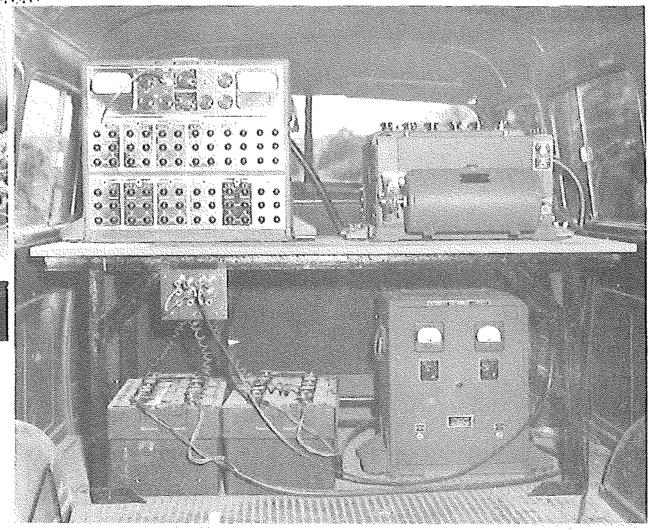
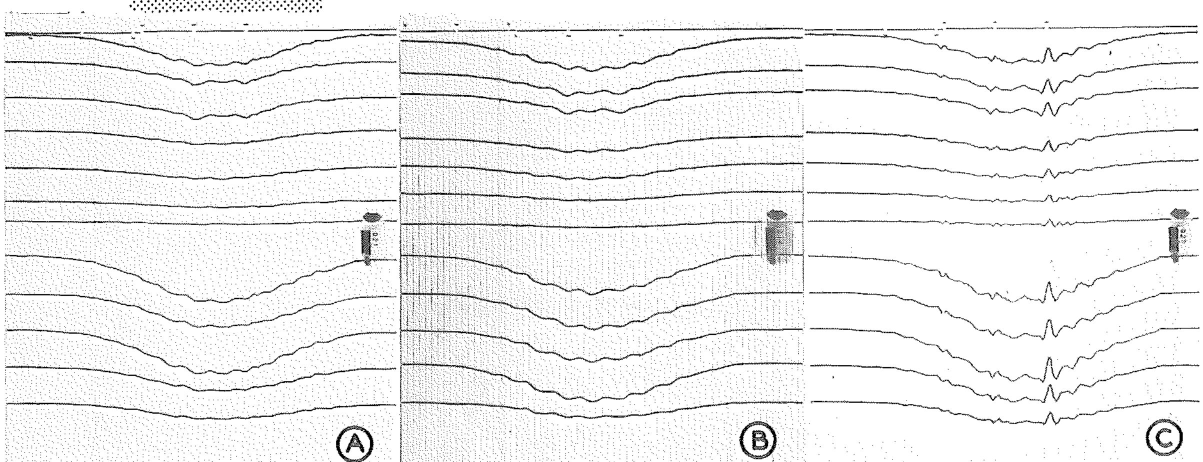


FIGURE 15. SR-4 INDICATOR AND ANDERSON SWITCHES FOR MEASUREMENT OF STATIC LOAD.

FIGURE 16. HATHAWAY 12-CHANNEL STRAIN ANALYSER FOR DYNAMIC TESTS.



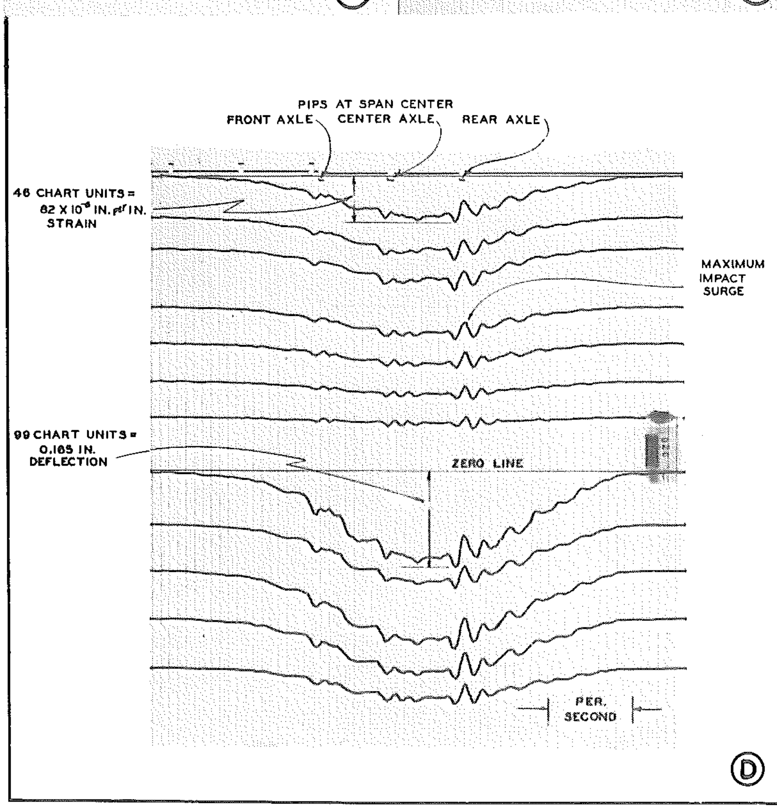


FIGURE 17.

(A) SPAN 1, SINGLE TRUCK AT 4 FT. POSITION NO IMPACT PLATE.

(B) SPAN 2, SINGLE TRUCK AT 3 FT. POSITION NO IMPACT PLATE TIMING LINES SHOWN.

(C) SPAN 4, SINGLE TRUCK OVER 3/4 IN. IMPACT PLATE.

(D) SPAN 5, TRUCK MOVING OVER 3/4 IN. IMPACT PLATE PAST STANDING LOAD IN ADJACENT LANE.

OUTLINE OF THE TEST ROUTINE

Gage and Deflectometer Installation

After a period of preliminary tests and explorations on Span 6, the test settled down to a routine except for a few special features. On Spans 3, 5, and 6, strain gages were cemented to each beam at mid span in five locations. Two gages were placed on the under side of the upper flanges, and three were fastened to the lower face of the bottom flange. They were symmetrically placed so that the two upper gages were equidistant from the web, two of the lower gages were equidistant from the center, and the fifth gage was directly beneath the web. This was illustrated in Figure 1.

When static tests were made, all of the gages were read. However, for dynamic testing it was possible to read only one gage per beam because of the limited number of channels on the oscillograph. The static readings permitted the computation of the location of neutral axis of the beam whereas the dynamic record gave only maximum fibre stress on the lower surface of the beam.

Spans 1, 2, and 4 were tested with only two gages per beam. These gages were symmetrically located on the lower face of the bottom flange.

The deflectometers were clamped within a few inches of mid span and as close to the strain gages as possible. A fine steel cable was stretched tightly from the hinged plate on the deflectometer to a turnbuckle, and again from the turnbuckle to an anchor on the ground. Thus, the hook on the hinged plate which is at the upper end of the cable is always fixed with reference to the ground. The dial and cantilever were actuated when the beam upon which the assembly was clamped deflected under load and lowered the remainder of the deflectometer and forced the dial stem against the plate. Reference again to Figure 12 clarifies this performance. On Span 2, due to the depth of the water and speed of the current, small wood piles were driven into the river bed to hold a beam under the line of gages. The deflectometer cables were fastened to this beam.

A pair of wires was soldered to each gage and a waterproofing material was applied over the gages and exposed soldered leads. The leads for the static tests ran directly to the static strain measuring equipment which was pictured in Figure 15. For dynamic tests, the wires were soldered to gage heads which, in turn, were connected to the dynamic strain analyser by shielded cables.

<u>Placement of the Load</u>

In general, test results were obtained for the load in three or more positions on the bridge roadway. Reference is made to these locations with respect to the distance from the center line to the line of the left wheels of the vehicle. Thus, position "0" indicated that the left wheels were running on the center line. They were three feet from the center line in position "3", and four feet from the center line in position "4". A "C.L." notation was used to indicate that the truck was straddling the center line.

For the static studies, the truck was stopped upon the span when the lateral center line of the span lay midway between the middle axle and the computed center of mass of the vehicle. Experimental placement to produce maximum strain proved that this position was not too critical. An error of 2 ft. in either direction could not be detected on the recorder.

When the simulated truck was assembled upon the span, it was always placed in position "4" in the left lane to represent a second vehicle overtaking and passing the first.

Moving load studies were made with the truck moving through positions "zero", "3", and "4". The speeds at which the vehicle was run are shown in the tabulated data in the appendix.

Impact runs were all made through position "4". Plates about 10 ft. long by 1 ft. wide were laid across the lane at mid-span. These plates were of steel, and had thicknesses $\frac{1}{4}$ in., $\frac{1}{2}$ in., and 3/4 in. They were placed to cause maximum downward impact at the center of the span.

General Procedure

Before each test, the vehicle was moved back and forth across the span a number of times. The intent was to break in the structure and reduce the shear between the deck and the steel beams. However, test results indicated that a more severe break-in treatment should have been used.

Next, the gage circuits were balanced and deflectometers set to zero. For static tests, the bridge was loaded, the readings made, the truck removed, and final readings taken. This procedure was repeated to give three sets of readings for each position.

For dynamic tests, it was always necessary to run a calibration trace after the gage circuits were balanced in order to obtain the ratio of micro-inches per inch of strain or deflection to the chart deviation. After this operation, the vehicle was driven across the span through the prescribed position. Again three records were made for each test.

Use of the Simulated Vehicle

After tests were run with a single vehicle, the standing load was placed on certain spans. Moving load and impact tests were then repeated with the design truck moving past the standing load.

Values representing deflections and strains caused by the combined loads of the simulated and mobile vehicles were obtained by an indirect method. The instruments were set at zero with the simulated vehicle on the span in position "4" in the south lane. The mobile vehicle was run past the simulated vehicle in the adjacent lane through positions "0", "3", and "4". The recorded values were those in excess of the condition of deformation due to the standing load alone. The total deflections or strains for this two-vehicle state were the sums of these measured values and the values due to a single vehicle at position "4".

For impact tests, since the simulated load could not be moved to cause impact, a surcharge of 15 percent was added. This figure was derived from an inspection of an experimental impact record on Span 5. It was thought that the accumulated values of the strains due to the surcharged standing load plus the recorded values shown by the impact record of the design vehicle might more nearly approach the true impact effect which could be caused by two moving trucks. This method has evident shortcomings, since the increased load undoubtedly had some damping effect upon the slab vibrations.

LIMITATIONS

The scope of the investigation was limited by several factors, the first being the difficulty in obtaining heavy design vehicles. Although the H20-S16 vehicle satisfactorily fulfilled the requirements of a design vehicle for static and slow speed tests, its performance was somewhat limited with respect to speed and braking power. Also, a second vehicle would have been much preferred to the simulated truck used in the south lane. This would have made possible the dynamic measurement of total strains and deflections for various lane positions and truck arrangements, and actual impact results from two vehicles could have been obtained directly, obviating the necessity for the surcharge on the standing load.

A second limitation was the fact that it was almost impossible under the circumstances to drive the vehicle across the span at more than 12 mph. This was due to two facts -- one, the difficulty in attaining higher speeds without excessively long approach run, and two, the room required to stop such a heavily loaded vehicle. There was no west approach to the bridge.

About 200 ft. of fill had been placed and gravel surfaced behind the west abutment, but this did not provide sufficient room in which to stop the truck at high speeds. It is probable that high speed runs can be attempted after the road to the west has been completed.

Third is the fact that the recording equipment had 12-channel capacity, whereas there were 14 strains and deflections to be read. As a consequence, an attempt was made to watch the two deflection dials farthest from the load, and note the sweep of the pointers.

Fourth, as in most tests, is the limitation of time. Some sort of a compromise must always be made between thoroughness of each test and the general scope of the project. Although three runs in rapid succession produced results with small variance, larger differences were noticed when similar groups of tests were performed later in the program. It would have been advantageous to have repeated all tests in both lanes and in both directions.

A LISTING OF THE TESTS AND PRESENTATION OF DATA

For an understanding of the scope of the investigation, a summary of all tests performed is given. These have been classified into four groups and are not listed in their chronological order.

1. Static Load Tests

   (a) One H20-S16 mobile vehicle in each of 3 lane positions on all spans except 6, 2, and 5 with single-bolted diaphragms.

   (b) One mobile design truck in each of 3 lane positions with simulated truck in adjacent lane on spans 4 and 5. Span 5 was tested with no diaphragm bolts, single-bolted diaphragm connections, and double-bolted connections.

2. Moving Load Tests

   (a) One design vehicle moving across span at 10 to 12 mph in each of 3 lane positions on all spans except 6.

   (b) One design vehicle moving across span at 10 to 12 mph in each of 3 lane positions with additional standing design load near center of adjacent lane. This test performed on Spans 3, 4, and 5. Span 5 with no diaphragm bolts, with diaphragms single-bolted, and also double-bolted.

3. Impact Tests

   (a) One design vehicle moving over each of 3 sizes of impact plates on Spans 1, 2, 3, 4, and 5.

   (b) One design vehicle moving over impact plates with additional standing load in adjacent lane on Spans 3, 4, and 5.

   (c) One design vehicle over impact plates with standing load surcharged 15 percent in adjacent lane. This program executed on Spans 4 and 5, with Span 5 again in 3 diaphragm conditions.

4. Miscellaneous Tests

(a) A tandem-axle vehicle was run at speeds up to 30 mph over an impact plate on Span 3 to note the effect of speed.

(b) The mobile design vehicle was run at about 12 mph over two impact plates at different locations and various spacings on Span 5 to explore for resonant frequency.

(c) Several diaphragms were fitted with strain gages to find the lines of principal stresses.

(d) Relative displacement of deck and beam was measured on Spans 3 and 5 to determine extent of slippage.

(e) A record of temperatures was kept.

(f) Physical data on the steel beams were obtained from the manufacturer, and flexure, compressive strength and static modulus tests were run on the bridge deck concrete.

TEST RESULTS

A complete tabulation of the data derived from the bridge loading studies is given in the table at the end of this report. Several apparent inconsistencies will be recognized in this tabulation. A possible explanation is the extent of reduction in shear between the deck and the beams. Graphs of the mid-span deflections and stresses are included in Figures 18 through 22. The truck position is shown schematically for each graph, and the effect of this position upon the beam stresses is quite evident.

Comparison of Design Values and Field Data

Design stresses and deflections have been computed for each span, using the Michigan State Highway Department's Standard Specifications for the Design of Highway Bridges. For live load and distribution of load, the Michigan Specifications are the same as the AASHO. However, for impact, the Michigan Specifications use the following formula:

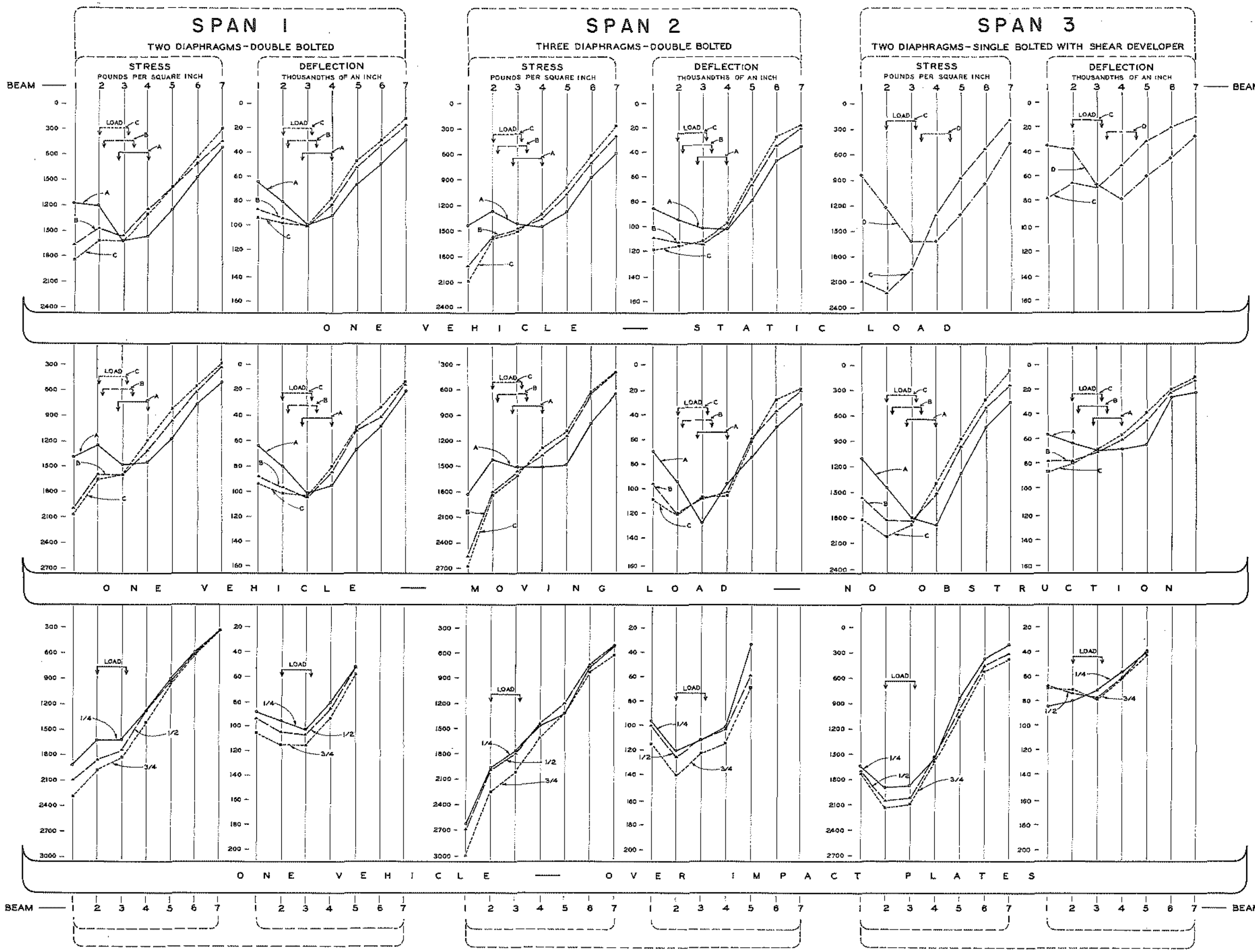


FIGURE 18. DISTRIBUTION OF STRESSES AND DEFLECTIONS ALONG LATERAL CENTER LINE OF SPAN 1, 2 AND 3

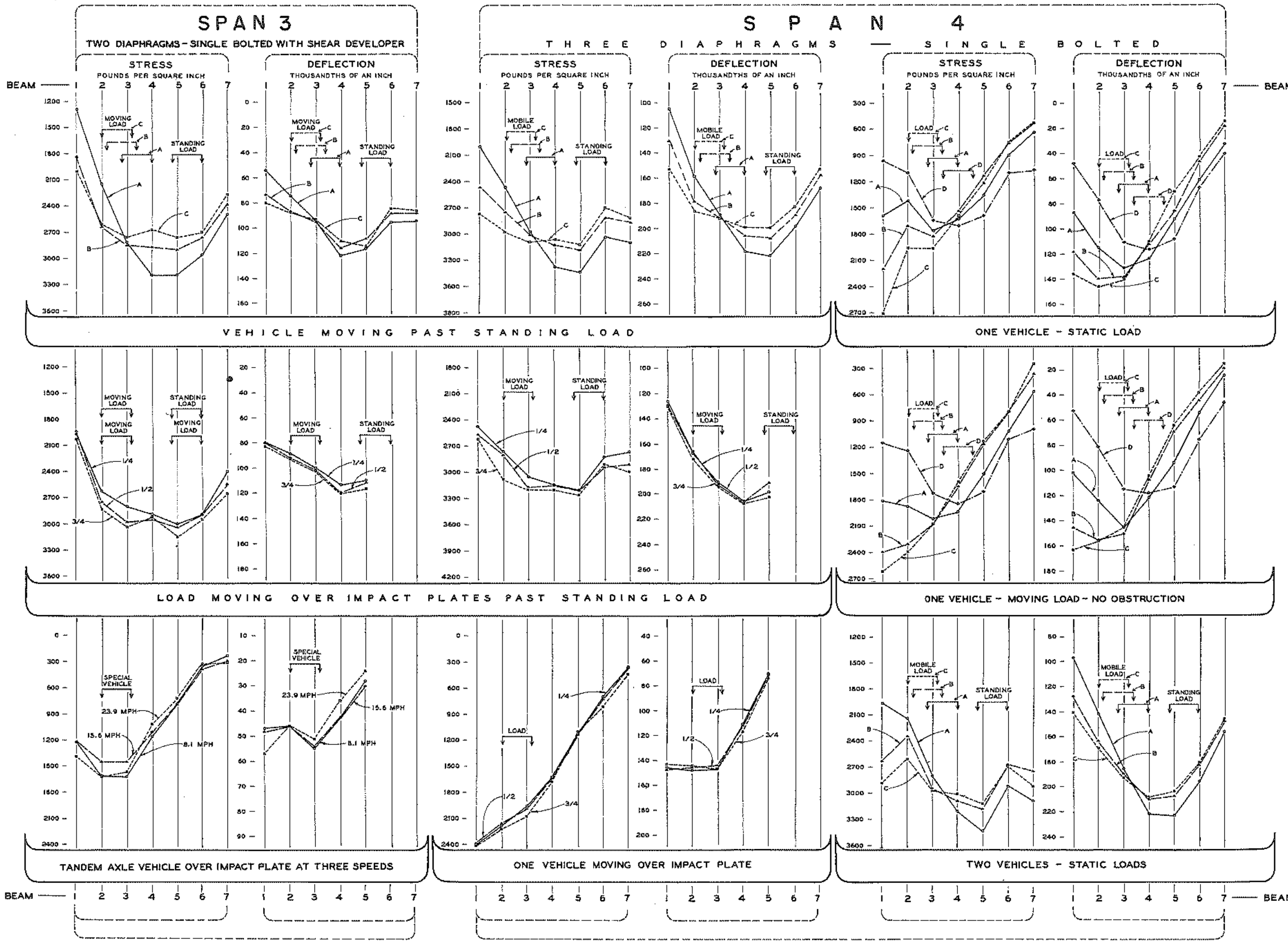


FIGURE 19. DISTRIBUTION OF STRESSES AND DEFLECTIONS ALONG LATERAL CENTER LINE OF SPANS 3 AND 4

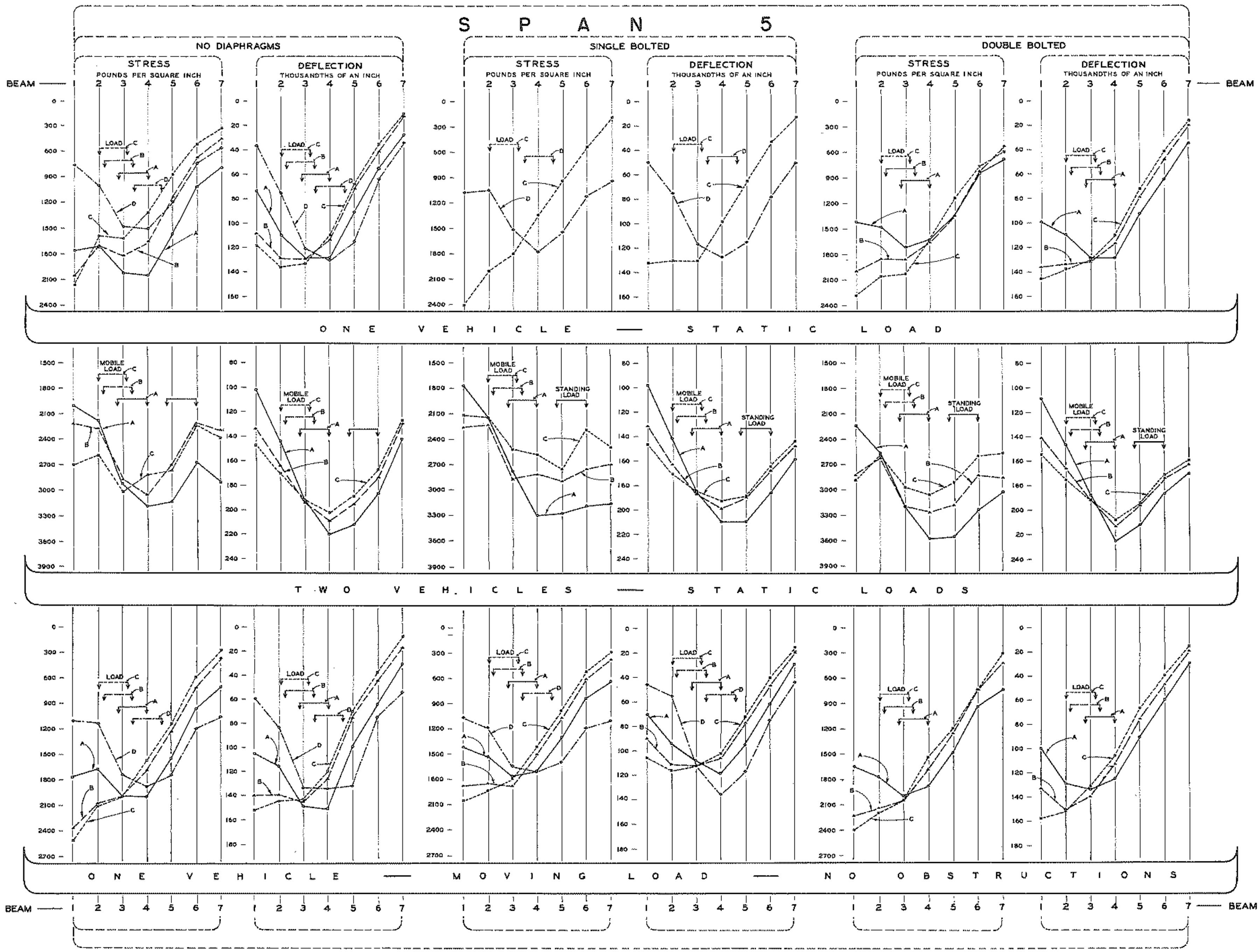


FIGURE 20. DISTRIBUTION OF STRESSES AND DEFLECTIONS ALONG LATERAL CENTER LINE OF SPAN 5

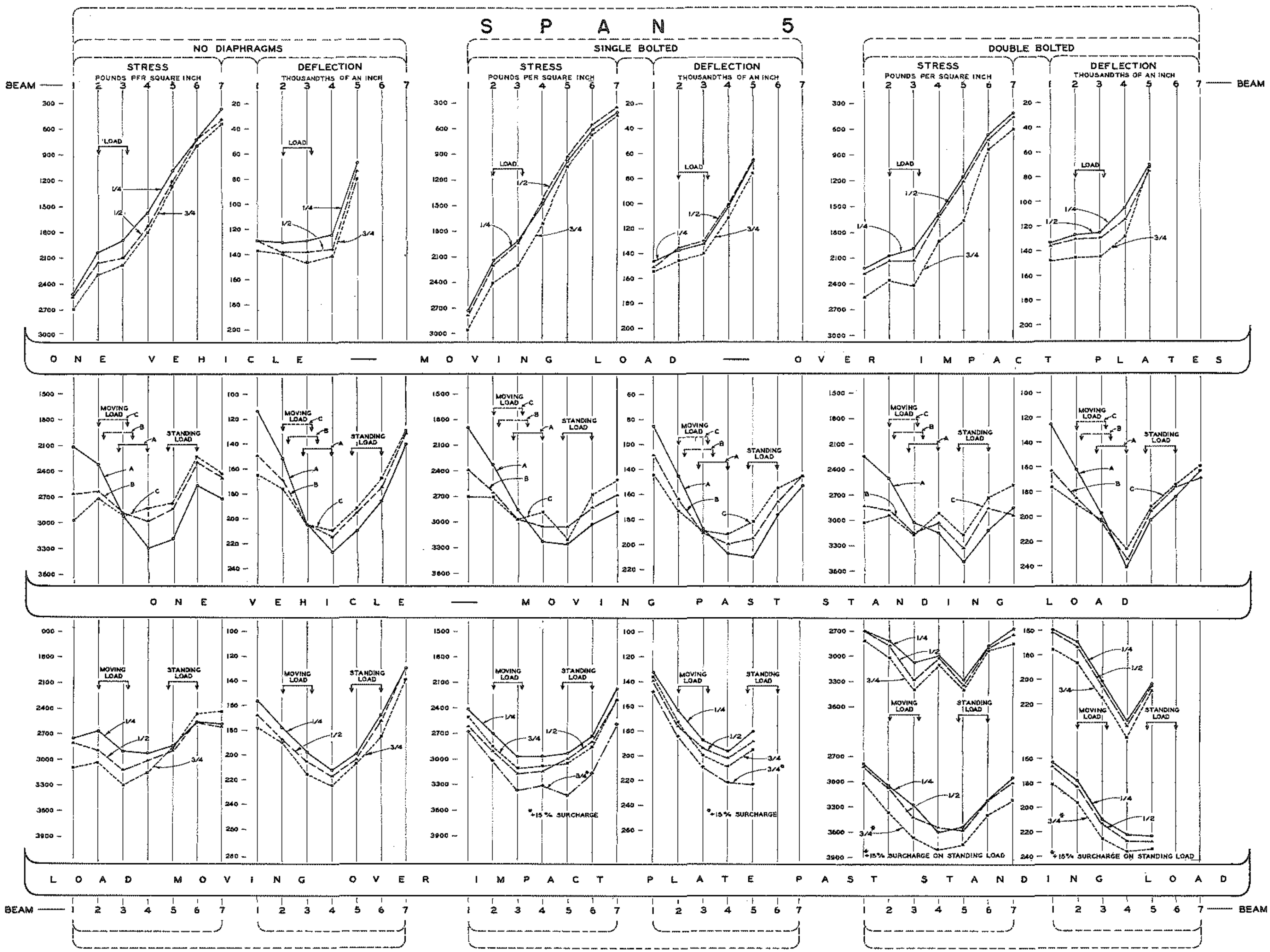


FIGURE 21. DISTRIBUTION OF STRESSES AND DEFLECTIONS ALONG LATERAL CENTER LINE OF SPAN 5

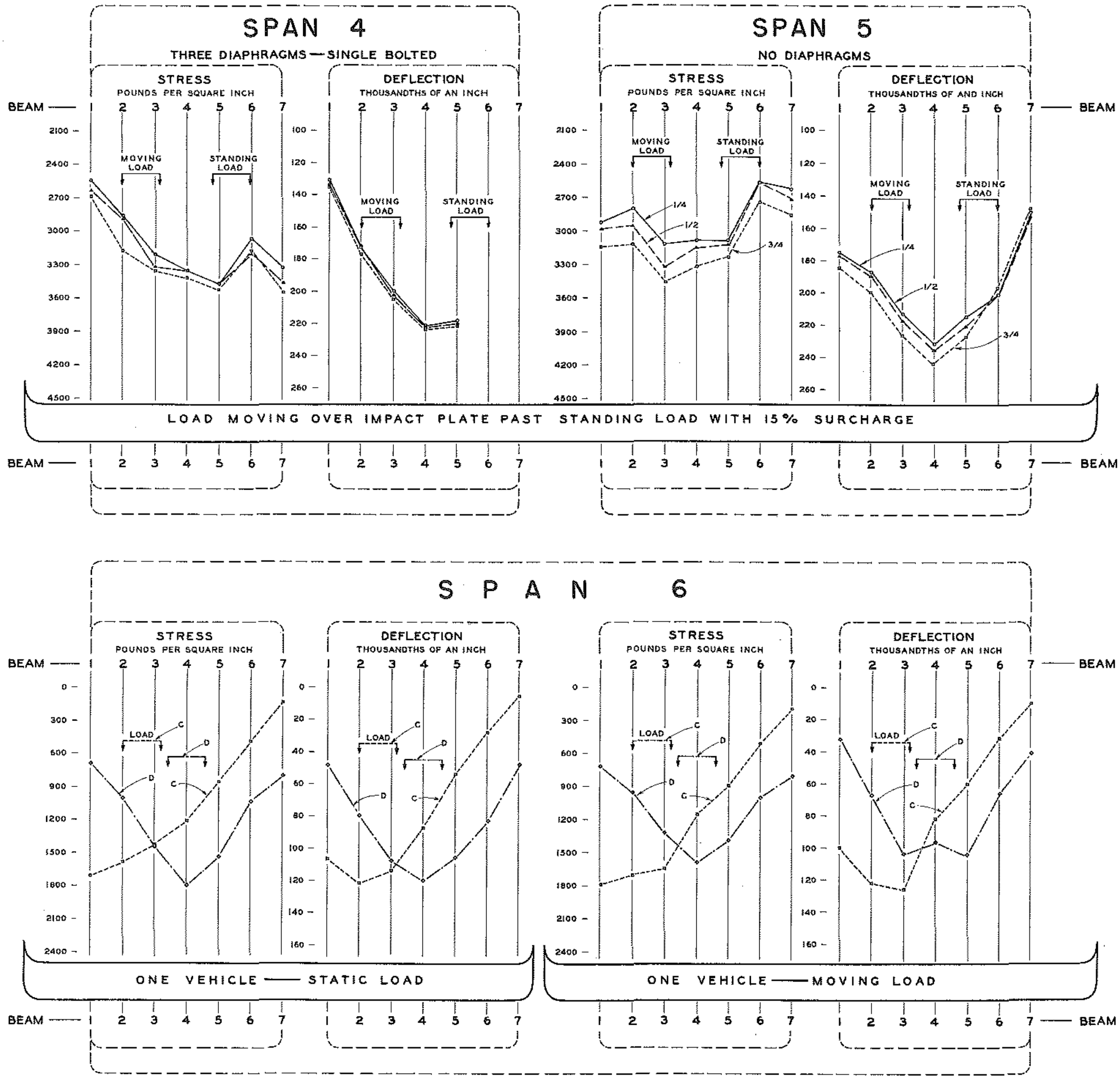


FIGURE 22. DISTRIBUTION OF STRESSES AND DEFLECTIONS ALONG LATERAL CENTER LINE OF SPANS 4, 5 AND 6

$$I = \frac{L + 20}{6L + 20}$$

For the span length involved in this project, an Impact Factor of 21.1 percent is obtained, as compared with 27.1 percent using the current AASHO Specifications. The results are compared directly with measured values in Table I. In this summary, Spans 1 and 6 are grouped because they are end spans with a length slightly shorter than the others. Spans 2, 4, and 5 differ only in diaphragms. Span 3 has assured composite action by use of a shear developer. The shear developers consisted of the Porete Company Alpha type spiral, which in this case was made of a ½ inch plain bar with a 4½ inch mean diameter and a variable pitch, welded to the top of the beam flanges.

Maximum measured deflections and stresses under single vehicle loading usually occurred when the truck was moving with the inner wheels 4 ft. from the bridge center line (Position 4), and under two vehicle loading when the standing load was at Position 4 in one lane and the mobile vehicle passed along Position 0 in the adjacent lane. Impact stresses were maximum when the 3/4-in. plate was used. Under single truck loading, impact tests were made for the 4-ft. position. This made possible the computation of impact effect on the basis of maximum measured deformation for a single truck. However, for two vehicles, impact was measured with both the mobile vehicle and the simulated truck at Position 4. Since maximum stresses and deflections were realized for two vehicles located at Positions 4 and 0 respectively, the effect of impact in this latter case was based upon deformations slightly less than maximum.

When the bridge was loaded with a single truck, the end spans were stressed to one-third of the computed design stresses, but the measured deflections were only one-sixth of the computed deflections. Spans 2, 4, and 5 developed slightly more than one-third of the design stresses and about one-fifth of the computed deflections. The trucks raised the measured stresses to almost one-half of design, and gave deflections slightly more than one-fourth of computed values.

TABLE I

MEASURED LIVE LOAD DEFLECTIONS AND STRESSES COMPARED WITH DESIGN VALUES

		STRESS IN P.S.I.			DEFLECTION IN INCHES			DEAD LOAD	
Load	Spans	Design	Measured	% of Design	Design	Measured	% of Design	Stress Design	Deflection Design
One Vehicle	1 & 6	6500	1960	33	0.713	0.115	16	8280	0.81
No Impact	2, 4&5	6630	2550	38	0.747	0.147	20	8520	0.85
	3	4690	2030	43	0.314	0.087	28	8520	0.85
One Vehicle	1 & 6	7880	2320	29	0.864	0.116	13	8280	0.81
3/4 in. Plate	2, 4&5	8030	2670	33	0.904	0.145	16	8520	0.85
	3	5680	2150	38	0.381	0.085	22	8520	0.85
Two Vehicles	4 & 5	7950	3495	44	0.896	0.232	26	8520	0.85
No Impact	3	5630	3190	57	0.377	0.116	31	8520	0.85
Two Vehicles	4 & 5	9630	3277	34	1.085	0.219	20	8520	0.85
3/4 in. Plate	4&5 W/S		3683	38		0.229	21	8520	0.85
	3	6820	3132	46	0.457	0.121	27	8520	0.85

Note: W/S indicates surcharge on standing load.

Span 3 showed less than one-half the design stress under single truck loading, and about one-fourth of the deflections. Two vehicles produced slightly over one-half the design stress and between one-fourth and one-third of the computed deflections.

Lateral Distribution of Deflections and Stresses

The distribution of stresses and deflections laterally across each span is seen by the graphs of Figures 18 through 22. It is seen that the deflection or strain exhibited by each beam varies greatly across the span.

In order to readily compare the lateral distribution in the six spans an index was developed. This index is the absolute sum of the deviations of the percent of total deflection or strain for each beam from 14 percent. In other words, the strain index was formed by (1) summing the recorded strains for all seven beams under a certain load condition and designating this total as 100 percent; (2) denoting the strain on each beam as a percent of this total strain; (3) finding the numerical difference for each beam between the percent of total strain and 14 percent, since each beam would be strained slightly over 14 percent of the total strain if the distribution were perfect; and (4) summing these deviations without regard to sign to form the index. A similar index was formed from the deflection data. The average of the index for strain and the index for deflection was used as the lateral distribution index of the span. Table 2 presents these indices.

TABLE 2

INDICES FOR LATERAL DISTRIBUTION

Span	Diaphragms Rows	Diaphragms Bolting	Indices Deflection	Indices Strain	Index of Lateral Distribution
1	2	double	48	46	47
2	3	double	48	42	45
3	2	single	48	52	50
4	3	single	52	48	50
5	0	none	50	48	49
5	2	single	40	46	43
5	2	double	50	44	47
6	2	single	55	45	50

As an indication of the relative values involved, it may be pointed out that if perfect distribution were achieved, i.e. all beams stresses or deflected the same amount, the index would be zero; and further, if no distribution were achieved, i.e. only one beam taking all stress or deflection, the index would be 170. Further, using the AASHO Design Specification for distribution of the loading involved, the index would be 128. Thus it can be seen from Table 2 that for the six spans involved, the range in indices is very small, indicating little difference in lateral distribution. While in general the table shows that more distribution is obtained as the stiffness in a transverse direction is increased, even here there is some discrepancy as indicated by Span 5 with single-bolted diaphragms, which appears to have a lower index than with double-bolted diaphragms.

Assuming that the indices of Table 2, though small, are significant, the following is observed:

1. A comparison of the indices of Spans 1 with 6, and also Spans 2 with 4, shows that double-bolting of the diaphragms offers slightly better lateral distribution than single-bolting.

2. The effect of the number of diaphragms is found by comparing indices for Spans 2 with 5 and Spans 4 with 3. Three rows double-bolted offer a little better distribution than two rows double-bolted, and three rows single-bolted produce the same index as two rows single-bolted.

3. Span 5, with no bolts, gave an index very slightly superior to that for Spans 3, 4, and 6. This might be interpreted to mean that the diaphragms do not aid materially in lateral distribution.

4. The index for Span 3 was one of the highest. This corroborates the fact that composite construction of deck and beams is not an aid in lateral distribution.

Factors in the Determination of Lateral Load Distribution

In an attempt to explain or predict the seemingly low values of stress and deflection obtained in the tests as compared to design values, it was deemed advisable to investigate and evaluate some of the basic factors influencing lateral load distribution. The two primary factors investigated were the load distributing characteristics of the concrete slab and the composite or partial composite action found to exist between slabs and beams.

Although it is well known and adequately demonstrated in the testing that the actual distribution of load to the various stringers is quite complicated, it has been useful in analyzing test data and for design purposes to assign a definite proportion of each wheel load to each beam. The proportion assigned to each beam depends on the beam spacing and on the load distribution characteristics of the transverse members.

In previous analytical, experimental, and field testing work by others, it has been convenient to use a certain dimensionless ratio, usually denoted "H", to represent the stiffness of the longitudinal beams relative to the stiffness of the slab in a transverse direction.

Extensive model testing and analytical work carried on at the Engineering Experiment Station of the University of Illinois by N. M. Newmark, S. P. Siess and others is reported in the Transactions of the ASCE, Vol. 114, 1949. From analysis of data obtained from many model tests, it was found that the proportion of a wheel load carried by a beam, or in other words the width of lateral distribution of a wheel load, could be expressed as a function of the relative stiffness factor "H".

It should be pointed out here that the concrete slab on the Fennville job is actually much thicker than the 7 in. considered in the design for the structure. The minimum slab thickness is increased by the incasement of the top flange, the transverse crown, and the amount added for dead load deflection. Thus, the slab thickness varies from about 9 in. at the fascia beam to more than 10-3/4 in. at the center line beam.

It can be readily seen that because of the thicker slab involved on the test bridge, the relative stiffness of the beams "H" will run comparatively low, and in fact varies from about 1.6 to 2.4 on the non-composite spans and from 3.7 to 4.1 on the composite span. In the University of Illinois Experiment Station investigations, it was assumed that representative designs of a 60-ft. rolled beam span would have an "H" value of from 3 to 8 for non-composite construction, and from 5 to 15 for composite construction. However, even though the "H" values for the Fennville structure are outside the range of values considered in the development of the formula for transverse distribution, the formula will be used later in making comparisons between predicted and field measurement values.

An additional complicating factor in these tests was the stiffening effect of the heavy safety curb. It is apparent, from a brief study of the tabulated test data, that the curb is acting with the slab in a transverse direction, resulting in a very stiff member. In many cases, the data shows the fascia beams are more highly stressed than the adjacent beams, even though the nearest line of wheels is over the first interior beam.

In the various series of static tests, where both bottom and top flange strains were recorded, it is of course possible to determine the location of the neutral axis of the beams. The tests reveal that even in the five spans where no shear developers were used, a large amount of composite action exists as evidenced by the position of the neutral axis well above the mid-depth of the steel beam. In order to make comparisons between measured strains and deflections with design and predicted values, it was necessary to evaluate the effect of the partial composite action. Without attempting to fully analyze this action, it was believed that a fair basis of comparison of test data would be to use values for Moment of Inertia and Section Modulus determined by direct proportion between no composite action and full composite action as given by the location of the neutral axes.

Analyses were made, using a width of lateral distribution given by the formula of N. M. Newmark, mentioned previously, and taking into account the partial composite action in the manner described above. To avoid complications from factors difficult to evaluate, only the results for the five center beams were considered. This eliminates the transverse stiffening effect of the curb and its further action as a composite section. Further, only the tests without impact were considered.

By formula, the width of lateral distribution for the non-composite spans for a line of wheels is 6.5 ft. and 5.8 ft. for the full composite span. In seven series of tests on Span 5, the percent of composite action varied from 34 to 70, with an average of 46. The measured stresses varied from 60 to 72 percent of predicted, with an average of 66 percent, while the measured deflections ran from 48 to 57 percent, with an average of 53 percent.

Some justification for the method of considering partial composite action was given by a study of three series of tests on Span 3, the one with full composite section. Here, the measured stresses varied from 65 to 69 percent of predicted, with an average of 66 percent, while the deflections varied from 36 to 38 percent, with an average of 37 percent.

The failure of measured stresses to reach more than about two-thirds of predicted values, even when thickened slab and partial composite action were taken into account, can be explained by the stiffening effect of the heavy safety curb, and the fact that the 12-in. wide beam flanges, partially encased in the slab, introduce restraining moments at each beam. It would be impossible from the test data available to evaluate each effect individually. Certainly, it can be predicted that in a wider bridge the effect of the curbs would be lessened on the beams near the center of the bridge.

Span Stiffness

Some consideration was given to the thought that the different diaphragm arrangements and fastening methods might affect the longitudinal stiffness of the spans. This stiffness was compared by noting the rank of numbers obtained by summing the deflections for all of the beams in each span, and also by comparing numbers representing the sum of the maximum strains for all of the beams in each span. These sums are tabulated in Table 3 for a single vehicle at Position 4.

TABLE 3

SUMS OF MAXIMUM STRAINS AND DEFLECTIONS OF BEAMS
FOR ONE VEHICLE AT POSITION 4

Span	Diaphragms		Sum of Deflections (10^{-2} in.)	Rank	Sum of Strains (10^{-5} in/in)	Rank
	Rows	Bolting				
1	2	double	47	2	28	2
2	3	double	55	4	32	5
3	2	single	36	1	30	3
4	3	single	68	7.5	37	8
5	0	none	68	7.5	35	6
5	2	single	56	5	31	4
5	2	double	66	6	36	7
6	2	single	53	3	27	1

Assuming the deflections and strains of equal importance, the values of total deflections must be weighed with those of total strain to arrive at a value for comparison. A simple average of ranks places the two end spans on the same level as Span 3 with the shear developer.

If the emphasis is placed upon deflections and the strain magnitudes are disregarded, we have the following pattern:

1. Span 3 with the shear developer is much stiffer than any other span.
2. Of the two end spans, 1 and 6, the span with double-bolted diaphragms is the stiffer.
3. Of the spans with three diaphragms, namely Spans 2 and 4, Span 2 with double-bolted connections is stiffer.
4. Span 2 with three diaphragms double-bolted is stiffer than Span 5 with two diaphragms double-bolted.
5. Span 5 with no diaphragms is of the same rank as Span 4 with three rows of single-bolted diaphragms, and the stiffness of Span 5 is only slightly improved by double-bolting the diaphragm connections.

Effect of Impact upon Stresses and Deflections

In the impact study, the vehicle was run through Position 4, which was directly over Beams 2 and 3. For the single vehicle test, these two beams usually showed maximum values of deflections and strains under this load position, and for that reason the computation of impact factor was based upon these values.

The data for two vehicles usually showed highest values on Beams 4 and 5. It seemed logical to use these values for the computation of impact factor under the double load conditions.

Table 4 is a summary of the deflections and stresses resulting from tests made by running the design truck over the 3/4-in. impact plate at speeds from 10 to 12 mph. The average impact factor is the arithmetic average of the percent increase in deflection and the percent increase in stress. These increases are the differences between the values found when the truck was run over the plate, and the values recorded when no plate was used.

The impact factors are seen to vary from 0 to 23 percent. There seems to be no correlation between impact factor and span construction.

Reliability of data might be questioned because Span 4 showed no factor under single truck loading. This irregularity may be due to inaccuracies in load placement or drift in the electronic measuring equipment, or possibly the impact developed by the moving load without the plate was comparable to that when the plate was used. There certainly was some effect due to impact, because the record traces showed the usual pip just to the right of the center as illustrated in Figure 17. It is hoped that more successful tests may be performed at a later date, using heavier loads traveling at higher speeds.

Vibration Characteristics

The undulations observed in Figure 17 are typical of all of the strain and deflection records. Although there is much variation in amplitude, there is regularity in frequency. The duration of vibration is limited to the interval that the span is loaded. The rate of damping is so great that there is no evidence of vibration after the load has moved off the span.

A tabulation of results is shown in Table 5. The data was taken from the deflection records for one vehicle at Position 4. The traces used were those for Beams 3 or 4, whichever exhibited the largest amplitude of vibration.

TABLE 4

EFFECT OF IMPACT UPON STRESSES AND DEFLECTIONS

(Single vehicle at position 4)

Span	Impact Plate	Defl. .001 in.	Stress p.s.i.	Defl. .001 in.	Stress p.s.i.	Defl. .001 in.	Stress p.s.i.	Av. Impact Factor %
1	none	102	1650	104	1590	103	1620	
	3/4 in.	116	2000	116	1860	116	1930	16
2	none	121	1830	107	1600	114	1715	
	3/4 in.	141	2230	123	2000	132	2115	20
3	none	80	2030	69	1890	75	1960	
	3/4 in.	71	2150	79	2120	75	2135	5
4	none	157	2380	145	2060	151	2220	
	3/4 in.	145	2260	147	2090	146	2175	-
5N*	none	145	2120	144	2000	144	2060	
	3/4 in.	140	2290	146	2180	143	2235	4
5S	none	116	1940	112	1800	114	1870	
	3/4 in.	145	2410	140	2180	142	2295	23
5D	none	152	2200	131	2060	141	2130	
	3/4 in.	144	2380	143	2440	143	2410	7
		(Two vehicles with surcharge on standing load)						
4	none	199	3130	199	3190	199	3160	
	3/4 in.	222	3450	222	3570	222	3510	11
5N	none	210	2810	192	2780	201	2795	
	3/4 in.	245	3330	228	3250	236	3290	17
5S	none	191	2870	182	2900	187	2885	
	3/4 in.	222	3310	223	3390	222	3350	17
5D	none	227	2900	193	3160	210	3030	
	3/4 in.	236	3800	234	3740	235	3770	18

* Diaphragm connections are designated as
N = no connection, S = single bolted, and D = double bolted.

TABLE 5
VIBRATION DATA

Span	1	2	3	4	5	6
Frequency (c.p.s.)	2.25	2.25	2.85	2.12	2.12	2.50
Amplitude (.00001 in.)	98	196	62	190	166	153

The record for Span 3 shows smaller amplitude and higher frequency than any other span. The end spans are next in order, with Span 1 showing lower amplitude and Span 6 giving higher frequency than Spans 2, 4, and 5.

Effect of Composite Deck Construction

The effects of the shear developer in Span 3 were noted in the previous discussions. A recapitulation of the relationship between Span 3 and the spans without shear developer is made, with reference to Tables 1, 2, 3, and 4.

Design computations anticipated a relief of 29 percent in stress and 58 percent in deflections when the shear developer was incorporated in the span. From Table 1, actual relief achieved under single truck loading was 20 percent in stress and 41 percent in deflections. Table 2 indicates no aid in lateral distribution from composite construction. However, Span 3 ranks first in span stiffness with maximum deflections as listed in Table 3 being only 55 percent of those for the free spans. The vibration chart, Table 4, shows increased frequency and diminished amplitude for Span 3 from those of the comparative spans.

SUPPLEMENTARY TESTS

As the opportunity presented itself, certain tests were made with the aim of supplementing the information gained in the regular testing program. These studies included more impact runs, an attempt to find diaphragm stresses, measurements of strains in the deck steel and on the concrete, effects of temperature, and strain readings on deck beams subjected to the weight of the concrete deck.

Impact Effects Caused by Tandem Axles

The crane used by the Bridge Maintenance Section was capable of attaining higher speeds than the H20-S16 truck, and it was decided to attempt some tests with this vehicle running over the 3/4-in. impact plate. The vehicle was constructed with a single axle supporting 7,650 lbs. in front, a second axle 11.5 feet from the front, and a third 4 ft. from the second. The combined load on the second and third axles was 29,550 lbs.

Runs were made at several speeds, and a final run without the plate was made for zero reference. The strains registered maximum on Beam 2, with Beam 3 giving values very nearly as great. Deflections were largest on Beam 3. The deflection readings for Beam 2 were considerably smaller. A condensation of the data is given below in Table 6.

TABLE 6

INFLUENCE OF VEHICLE SPEED UPON IMPACT EFFECTS

Run No.	1*	2	3	4	5	6	7	8
Vehicle Speed, mph.	8.1	12.8	13.4	14.5	15.6	17.7	23.9	8.7
Strain (10^{-6} in/in)	56	56	54	54	50	52	56	46
Deflection (.001 in.)	55	56	56	57	54	52	51	41

*Note: On Run 1, the vehicle stopped with rear wheels on the span.
On Run 8 there was no impact plate.

The results show a trend toward a minimum impact effect for this vehicle when it was driven at a speed of 16 to 20 mph. The maximum impact factor was 39 percent, based upon deflections, and 22 percent, based upon strains.

Effect of Successive Impacts and Location of Impact Plates

Some exploratory testing for the effect of impact plate spacing was done on Span 5. The 3/4-in. plate and the ½-in. plate were used. They were placed so that the H20-S16 truck first hit the 3/4-in. plate, and then the ½-in. plate, while the truck was traveling fully loaded at 11 mph. There were two series of tests made; first, with a 1-ft. distance from the span center to the edge of the $\frac{1}{2}$-in. plate, then distances of 1, 2, 3, 4, and 5 ft. between plates. The second series differed in that the distance from span center to the $\frac{1}{2}$-in. plate was $3\frac{1}{2}$ ft. The same plate spacings were used.

The record consistently showed maximum strain and deflection values at Beam 1. These maximums are tabulated below.

TABLE 7

EFFECT OF SPACING OF IMPACT PLATES

	Strains					Deflections				
Spacing, ft.	1	2	3	4	5	1	2	3	4	5
Series 1	97	99	97	94	94	178	179	173	167	174
Series 2	102	101	98	102	92	179	180	178	175	160
No Plate	95					173				

It appears that highest values were obtained at 2-ft. spacing in Series 1, and at either 1- or 2-ft. spacing for Series 2. The effect seemed to fall off sharply at the 5-ft. spacing in Series 2. Since both the strain and deflection magnitudes for this distance were below those for the No Plate condition, it is possible that the vibrations were out of phase so that the downward impulse caused by the second plate occurred while the surge from the first impact was upward.

Computing for critical plate spacing using vibration data for Span 5 from Table 5 and a truck speed of 11 mph. (16.1 fps.) we find that in the interval $\frac{1}{2}$.12 sec., the truck traveled 7.6 ft. Unfortunately, the maximum experimental spacing was 5 ft. According to this method of computation, a spacing of 3.8 ft. ($\frac{1}{2}$x7.6 ft.) should have caused a bucking action due to phase shift, and the recorded values for this plate spacing should be low. Some reduction was evident in Series 1, but not in Series 2 at the 4-ft. distance.

Stresses in Diaphragms

Diaphragms on Span 6 were equipped with gages for the purpose of determining magnitude and direction of principal stresses while the span was subjected to load. The gage layout is given in Figures 23 and 24, and the data is shown in Table 8. Three diaphragms were in the east row on Span 6, and were numbered from north to south. The designations 1, 2, and 3 in Table 7 respectively indicate the diaphragms between Beams 1 and 2, 2 and 3, and 3 and 4. Diaphragm 4 is in the west row on Span 6 between Beams 3 and 4. The gage layout on this diaphragm is on Figure 24.

Computations of principal strain magnitudes and directions from the readings of the rosette gages gave the results which are shown schematically in Figures 23 and 24. Most of the values on the diaphragm webs are small, although in the case of the diaphragm connecting Beams 3 and 4, a resulting strain of 86 micro-inches per inch was found. In Figure 24, the largest value shown is 57 micro-inches per inch. In terms of steel with a modulus of elasticity of 30 million psi., these strains indicate stresses of 2580 psi. and 1710 psi. respectively.

The diaphragm directly beneath the load seems to be in the state of highest stress. This is illustrated in the second drawing in Figure 24. Note also that one angle fillet stress is high. The strain of 134 micro-inches per inch is equivalent to 4020 psi. of stress.

Measurement of Relative Movement Between Deck and Beam

Dial indicators were attached to the underside of the deck near the piers. This detail was shown in Figure 14. Exploration on Span 6 proved that the greatest relative movement occurred at the ends, and movement at the center of the span was less than 0.001 inches. Readings at the ends of Spans 5 and 3, representing relative movements per half span length, are tabulated in Table 9.

TABLE 8

STRAINS IN DIAPHRAGMS

(Strains in 0.000001 in. per in.)

Gage Location (Fig. 23)	Truck over 2 & 3 Diaphragm			Truck over C.L. (W.) Diaphragm			Truck over 5 & 6 Diaphragm			Truck over C.L. (E.) Diaphragm		
	1	2	3	1	2	3	1	2	3	1	2	3
A	5	12	30	9	13	22	0	-2	15	7	12	10
B	10	10	80	8	11	13	-5	0	5	5	10	0
C	-5	13	70	7	9	8	-8	2	3	0	8	-10
D	15	20	20	0	11	32	-5	10	13	-5	13	20
E	15	20	20	0	12	37	-8	0	13	3	17	20
F	22	18	20	5	15	38	-10	0	20	3	20	29

Fig. 24	Truck over 2 & 3	Truck over 3 & 4	Truck over 4 & 5	Truck over C.L.
1	3	35	12	26
2	9	56	30	46
3	10	45	19	32
4	8	25	0	12
5	6	27	10	16
6	6	18	0	0
7	10	30	-7	12
8	18	45	5	28
9	18	43	8	26
10	4	28	11	22
11	0	24	16	-
12	-3	12	4	0
13	8	10	-17	-
14	6	6	-15	-
15	11	16	-5	-
16	120	134	39	-
17	-28	17	66	-
18	68	0	-66	-
19	-22	-11	0	-

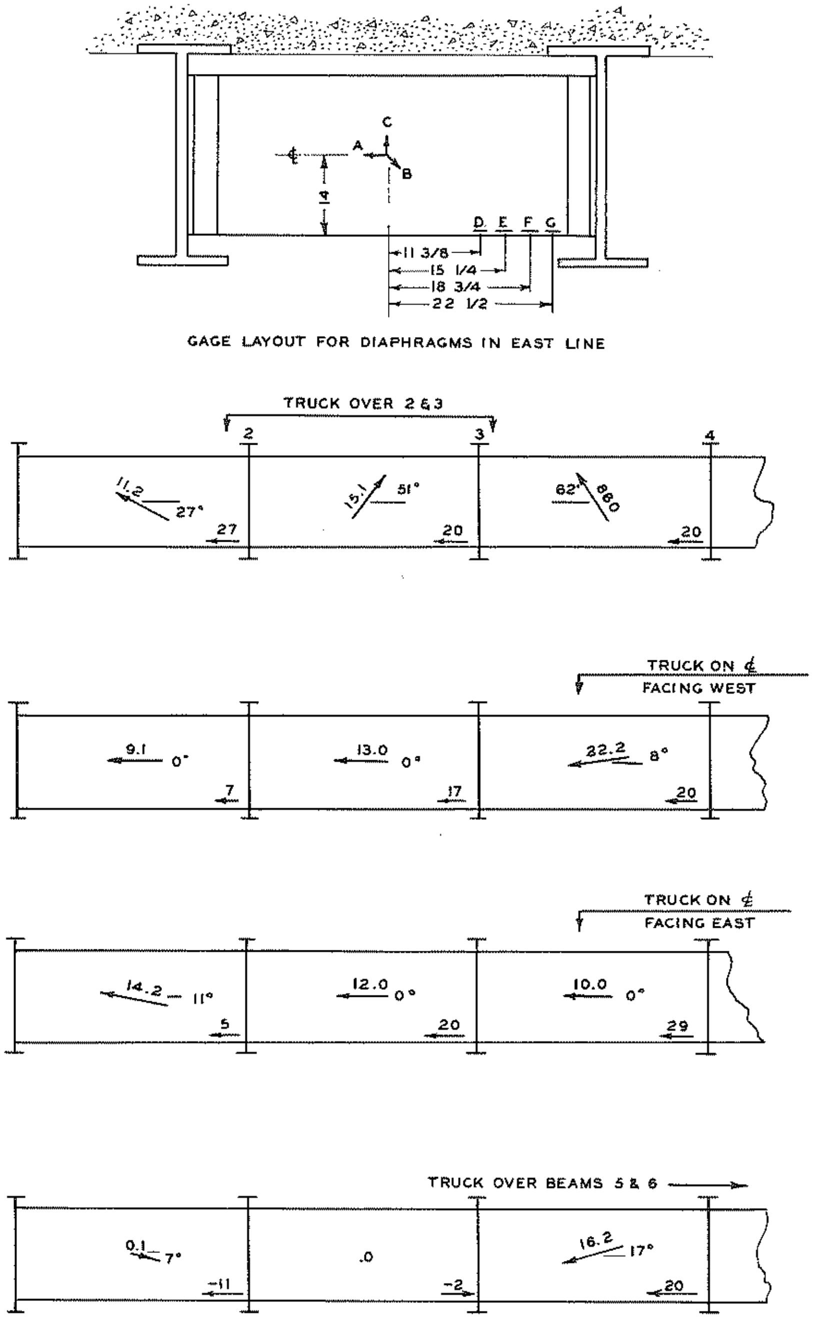


FIGURE 23

STRAINS IN DIAPHRAGMS — SPAN 6

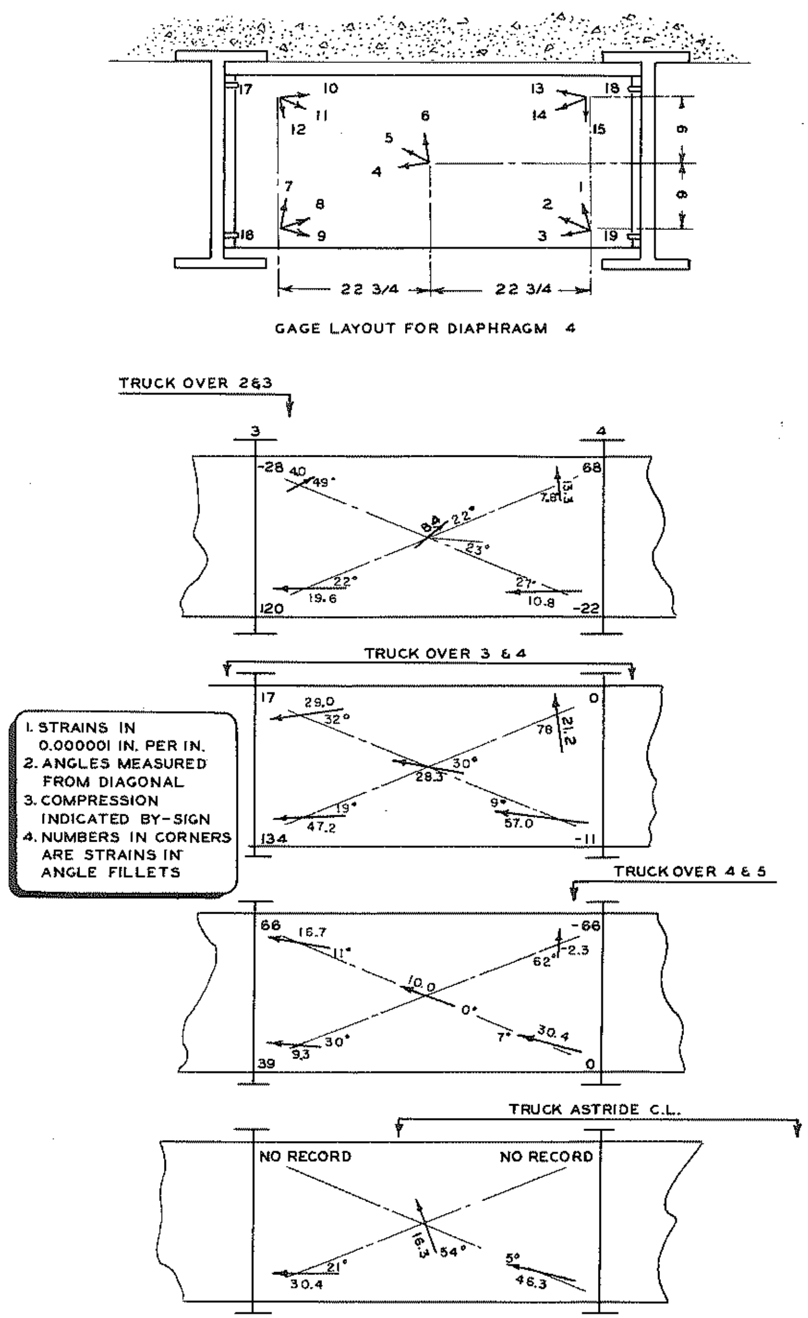


FIGURE 24

STRAINS IN DIAPHRAGM 4

TABLE 9

MOVEMENT BETWEEN BRIDGE DECK AND STEEL BEAMS

(Relative movement in ten-thousandths inches)

Truck Position	SPAN 5												SPAN 3			
	ONE VEHICLE						TWO VEHICLES						Single Bolted Diaphragms			
	No Diaphragms		Diaphragms Single Bolted		Diaphragms Double Bolted		No Diaphragms		Diaphragms Single Bolted		Diaphragms Double Bolted		One Vehicle		Two Vehicles	
	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3	Dial 2	Dial 3
0	99	138	95	135	111	139	112	171	148	218	99	115	5	8	4	7
3	108	139	110	141	132	138	109	203	107	216	96	121	4	8	6	8
4	106	138	112	132	136	128	108	202	178	182	115	122	5	7	6	9

NOTE: Dial 2 - Read movement at Beam 2
Dial 3 - Read movement at Beam 3

Truck positions are distance in feet from C.L. to nearest wheel.

It should be explained that the recorded movement for two vehicles is not a total movement, but is in reality an increment caused by a single truck. The readings were made from an assumed zero after the standing load had been placed. There is no method of accumulating these values, because the mobile truck was not run through the standing load positions, nor were dials attached to Beams 5 and 6.

The results indicate relative movement of 0.01 to 0.02 in. near the ends of the span for Span 5. No effort was made to determine where, along the span, slippage was sufficient to cause bond breakage.

The Span 3 data shows no movement as great as 0.001 in. This seems to be conclusive evidence of composite action.

Observations on Temperature Effects

The fact that the deflectometers used in this study behaved erratically when the reading interval was of a duration longer than half an hour led to a study of the effects of temperature upon these readings. The specific objectives were: (a) to observe the behavior of a free indicator under temperature fluctuations; (b) to measure the vertical movement at the span center and try to correlate this movement with temperature; (c) to observe the effects of temperature change upon relative movement between deck and beam; (d) to measure variations in expansion joint width; and (e) to check the reliability of the deflectometer reference system by comparing readings of the deflectometers using steel cables attached to anchors on the soil surface with the readings determined from dials supported by steel and wood columns.

Indicator Reliability: - The dial indicators were mounted in a position which would subject them to direct sunlight for a part of the day and to shadow for another part. They were allowed to remain here throughout a complete 24-hour cycle, with temperature fluctuations from 58° F. to 95° F. The maximum variation in the reading was .001 in. This was sufficient proff of reliability, and it was concluded that the observed fluctuations on the bridge deflectometers were due to external causes.

Reference Check: - Adjacent to deflectometer locations at Beam 4 and Beam 7 at the south fascia, columns were erected and dial indicators attached to the top with the stems resting against the bottoms of the respective beam flanges. The center column was of wood, and the outside was a $1\frac{1}{2}$-in. steel pipe. Although the dial readings varied throughout the test period, the fluctuations at the center beam were the same for both dials, and similarly for the dials at the outer beam. It was concluded that the steel cable method of maintaining a reference for the deflectometers was dependable.

Study of Vertical Movement of Unloaded Span: - Indicator dials were installed atop steel columns to study the vertical movement of the beams of Span 5 at mid-span. Three positions were selected, one at Beam 1 at the north face, a second at Beam 4, and a third at Beam 7. Readings were made on four consecutive days.

To supplement the dial readings, deck temperatures were read by means of surface thermocouples. Table 10 includes these readings, together with those for the expansion joint width changes and relative movement between deck and beams.

The vertical movement of the span ranged from -.055 in. on one side to +.070 on the other. The record does not seem to show any trend, but rather an unpredictable fluctuation. Daily temperatures seemed to have greater influence than the temperature differential in the deck. However, the data makes evident the difficulties encountered in the measurement of deflections due to load when the time interval is large.

Expansion Joint Width Changes: - Two parallel lines were scribed upon each end of the metal plates of the expansion joint between Spans 5 and 6, for the purpose of measuring changes in joint width. Periodical readings of the distance between these lines gave the data shown in Table 10. The maximum width change was 0.06 in. for a temperature change of 22° F. Since these joint width changes represent the expansion in two span lengths, the measured value was only about one-third of the predicted 0.20 in. which should occur under free expansion.

TABLE 10

EFFECTS OF TEMPERATURE CHANGES

Day of Month	Time	Deck Temp. (°F)		Changes in Exp. Jt. Width (inches)		Vertical Movement of Span (.001 in.) Beams(1)			Relative Movement of Slab & Beam (.0001 in.) Beams(2)	
		Top	Bottom	N.	S.	1	4	7	2	3
18	4:00 p.m.	80	78	0	0	0	0	0	0	0
19	8:00 a.m.	66	66	-.01	-.01	62	43	-55	-5	-1
	11:00 a.m.	71	70	0	0	35	23	-8	-4	1
	2:00 p.m.	77	70	-.02	-.02	22	3	32	-2	1
	5:00 p.m.	80	75	-.04	-.04	18	8	14	-2	1
20	8:00 a.m.	58	58	.02	.02	46	58	-50	9	-11
	11:00 a.m.	67	67	.01	.02	62	43	-9	17	-10
	2:00 p.m.	76	70	0	0	68	38	62	17	6
	5:00 p.m.	80	75	-.01	0	70	48	51	17	0
21	8:00 a.m.	64	64	0	.02	85	0	-20	17	-6

Note 1 - A negative sign indicates an upward deflection.
Note 2 - Relative movement here is due to causes other than load.

Measurement of Strains in the Concrete Deck

Before the decks of Spans 3 and 4 were cast, gages were cemented to the lateral reinforcing steel as shown in Figure 13. There were two lines of gages on each span, one line being 5 ft. from the end and the other at the center. A plan of the installation on Span 4 is shown in Figure 25. Gages A, C, and E were on the bottom face of the lower reinforcing rod, and they were placed midway between the supporting beams. The remaining gages were attached to the top of the upper rods, and were directly above the beams.

Span 3 was also equipped with gages, in a layout symmetrical to that of Span 4. The end gages were 5 ft. from the east pier in this case.

Readings were taken at the time of installation before the deck was placed, and at various times after pouring. Final readings were made with the span loaded by the design vehicle. The results are given in Table 11.

Analysis of the data on strains in reinforcing steel is complicated by the irregularity of the results. An inspection of the record prior to the loading tests suggests that some electrical distrubance other than change in gage resistance or creep in the bonding material affected the gages. For example, the first line in Table 11 shows a strain of 1500 micro-inches per inch in the steel. Since the steel is bonded to the concrete, a similar strain must be transferred to the surrounding concrete. But concrete can resist only about 150 micro-inches per inch of tensile strain without cracking, and no crack was seen at this point in the deck. There are many entries over 150 micro-inches per inch.

A second consideration is the divergence of the data for Span 4 at the center. Instead of an increase in tensile strain, almost all of the values here are compression.

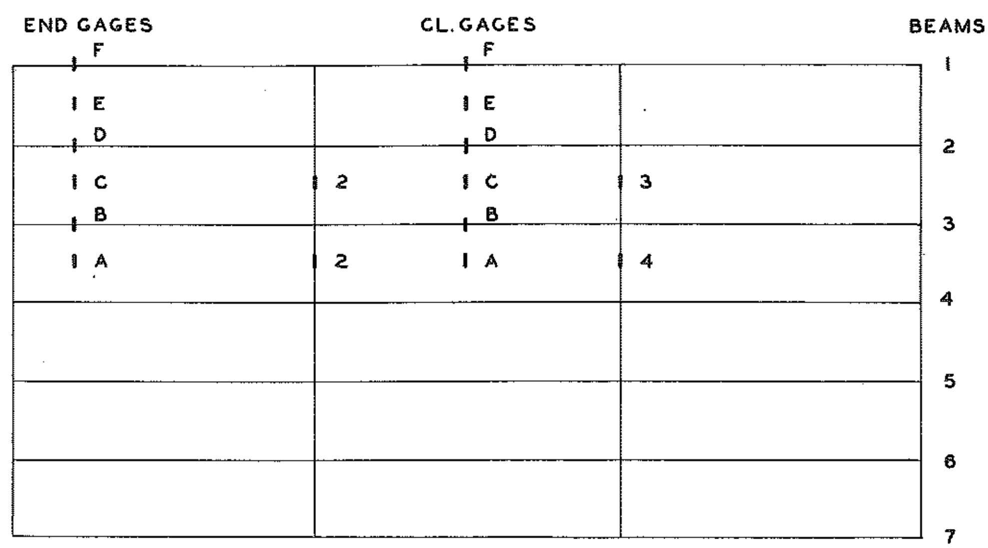
Under the loading study, no trend or pattern has been discovered. Most of the values were very small, although one column of data on Span 4 contained larger strain values.

TABLE 11

STRAINS IN REINFORCING STEEL
(Strain indicator readings in 10^{-6} in/in.)

Gage Location	After Set	Age 2 da.	Age 2 wk.	Age 1 mo.	Age 2 mo.	Load Stresses with Indicated Truck Positions 1	2	3	4	5
Span 3, E.										
A	400	510	1585	1545	1500	-5	-5	10	-15	25
B	60	-10	150	35	-130	21	10	Gage Failed - -		
C	130	20	230	260	250	15	10	30	20	30
D	160	235	255	435	1385	27	0	Gage Failed - -		
E	93	-45	45	68	50	-10	0	25	-10	0
F	180	-150	360	463	1360	10	5	10	15	10
Span 3, ctr.										
A	Gage Failed -									
B	185	325	455	503	295	18	28	15	5	25
C	96	105	545	1085	685	25	5	10	25	40
D	-60	-140	-470	-943	60	12	15	Gage Failed - -		
E	70	230	290	620	375	0	5	8	10	10
F	150	60	160	525	335	5	5	20	10	10
Span 4, W.										
A	-15	-32	-95	1535	4370	-8	-8	0	20	30
B	-15	-32	-305	-15	1095	5	15	15	-10	25
C	25	30	-120	-50	230	-2	-12	-3	-3	40
D	-125	-70	-285	25	-215	-55	-55	80	55	Gage Failed
E	150	185	15	445	1020	7	20	10	0	-5
F	-50	-57	-300	-295	-250	10	13	5	-10	-10
Span 4, ctr.										
A	45	-15	115	-350	70	32	53	70	45	8
B	-88	-180	-1075	-1175	-805	-9	8	125	-25	-40
C	56	-30	-305	-465	0	27	18	60	-5	22
D	-12	-120	-1145	-1355	-1120	-10	15	75	10	-100
E	-74	60	-1530	-1400	-1725	-11	-43	75	5	80
F	18	-95	-960	-970	-660	19	17	65	0	-10

Position 1 - Load over beams 2 & 3, middle axle over center line of gages
2 - " " " " " " " end line of gages
3 - " astride beam 4, " " " center line
4 - " " " " " " " end line
5 - " " " 3, " " " center line



LETTERED GAGES ON REINFORCING RODS
NUMBERED GAGES ON BOTTOM OF DECK

FIGURE 25

GAGE LAYOUT FOR MEASUREMENT OF DECK STRAINS

It seems at present that the gage installation on reinforcing bars is of doubtful value.

Strains on the Deck Surface Due to Live Load

A brief investigation of strain magnitude on the lower surface of the concrete deck was made by cementing A-9 gages directly above the diaphragms. The plan of Figure 25 shows the locations. Data from the study is given in Table 12.

Most of the measured strains were very small. The 70 micro-inch per inch value on Gage 1 was the largest. This is equivalent to about 300 psi. of stress, which is well below the modulus of rupture of the concrete.

Tests on Materials

The bridge deck materials were inspected and tested by the Pittsburgh Testing Laboratory and Michigan State Highway Department inspectors. Table 13 is indicative of the quality of the materials used.

SUMMARY OF OBSERVATIONS

From the foregoing discussion, certain facts are evident and others offer opportunity for discussion. Some of the evident facts are:

1. All spans were conservatively designed. Except for Span 3 with composite action, the measured stresses were less than half the computed values, and measured deflections about one-fourth those computed.
2. Lateral distribution of load was not materially aided by diaphragms. There seemed to be about the same degree of lateral distribution of load whether the diaphragms were single-bolted, double-bolted, or not bolted at all.
3. The positive factors influencing relative span stiffness were limited to the composite action achieved by the shear developer and embedment of beams in abutments. The apparent influence of diaphragms seemed to be nullified as the partial composite action was reduced.

TABLE 12

LATERAL STRAINS ON LOWER SURFACE OF CONCRETE DECK

Truck Position	Mid-Axle Location	A. SPAN 5 - SINGLE BOLTED Readings in .000001 in/in. Gage 1	Gage 2	Gage 3	Gage 4
Astride C.L.	E*	12	13	10	37
	W	21	37	15	28
Outer Wheels on C.L.	E	19	10	45	23
	W	70	20	27	24
Astride Beam 3	E	20	10	37	37
	W	48	30	30	20
		B. SPAN 5 - DOUBLE BOLTED			
Astride Beam 3	E	29	16	38	41
	W	57	34	31	23
Outer Wheels Over 3	E	19	11	22	8
	W	40	15	25	14

* E indicates east diaphragm line, W indicates west.

TABLE 13

TEST RESULTS ON MATERIALS

(a) Steel

Item	Yield p.s.i.	Ultimate p.s.i.	Elongation percent	Chemical Analysis C	Mn.	P	S
WF Beams	37,780	65,100	32.5	.23	.56	.012	.036
5/8 in. def. bar	48,029	81,152	18.6	.39	.42	.010	.035
1/2 in. def. bar	50,530	78,322	20.1	.36	.46	.011	.040

(b) Concrete

Aggregate: Postma 6A coarse
2NS fine

Cement:	Span		Percent Air
	6	Medusa A.E.	4.3
	5	Aetna A.E.	7.0
	3	Aetna Std. + 3/8 oz. Darex	6.4
	3	(corrected) + 1/4 oz. Darex	4.4
	2	Aetna Std. + 1/4 oz. Darex	4.1

6 in. x 6 in. x 36 in. Test Beam

Mod. of Rupture 7 da.	Mod. of Rupture 28 da.	Comp. Strength 28 da.	Mod. of Elast. 28
533 p.s.i.	650 p.s.i.	4,460 p.s.i.	4.83×10^6 p.s.i.

4. The effect of impact upon slab stresses and deflections was not studied sufficiently to provide a satisfactory value for impact factor. Experimental values of this factor varied from 0 to 23 percent, and no cause for such variation was discovered.

5. The frequency of vibration of the spans was dependent upon the span stiffness. The stiffer spans vibrated at higher frequencies and lower amplitudes than the others.

6. The incorporation of shear developers in Span 3 produced a stiff span, but did not aid in lateral distribution of load. Deflections of this span were only half of those found in the spans without composite action under the same loading conditions.

7. Stresses in diaphragms were for the most part of small magnitude. This fact is further corroboration of the statement that diaphragms play a minor role in the lateral distribution of load.

8. Slippage measurements between deck and beam indicate bond breaks in spans without shear developer and composite action in the span with the shear developer. It is quite possible, however, that there could be considerable bond between deck and beam near the center of the spans. The limits of this area of effective bond were not measured.

DISCUSSION OF TEST RESULTS AND SOME CONCLUSIONS

Detailed study of the test results indicates that in general it is apparent that the type or number of diaphragms are not of great importance in lateral distribution of load. While it is true that in most test cases more lateral distribution was obtained with stiffer deaphragms, the amounts were small, and in some instances, as previously mentioned, the effect was just the opposite of that expected. The latter effect is undoubtedly explained by the fact that different amounts of partial composite action were obtained in the various tests, and in general, as expected, there was a gradual destruction of the partial composite action in the later tests.

The change in the amount of composite action in the tests suggests that it would be wise in future tests to make an effort to reduce the composite action to a minimum, if possible, by means of heavy loadings and impacts. That some residual composite action, whether due to bond or friction, would remain can be predicted by results reported in the magazine Civil Engineering, Vol. 21, No. 7, of July, 1951, of tests on the Skunk River Bridge in Iowa. These tests were made on a bridge that had been subjected to heavy traffic during its 28 years of service, and still showed partial composite action.

The failure of measured stresses to reach more than about two-thirds of predicted values, even when thickened slab and partial composite action were taken into account, can be explained by the stiffening effect of the heavy safety curb and the fact that the 12-in. wide beam flanges, partially encased in the slab, introduce restraining moments at each beam. It would be impossible from the test data available to evaluate each effect individually. Certainly, it can be predicted that in a wider bridge the effect of the curbs would be lessened on the beams near the center of the bridge. In the matter of the restraining effect of the wide beam flanges, it is possible that some reduction of this effect would be obtained by the heavy loading tests suggested above.

Of particular interest are the excellent results obtained on the span using the shear developers. The tests on slippage and stress and deflection indicate full composite action was obtained. From a general appraisal of the test results, it would appear that one possibility for future savings in bridge design would be to take advantage of the partial composite action known to exist, and use less conservative methods in designing shear developers. Of course, further testing would be in order before taking such a step. Certainly, the evidence from this test indicates that there is just cause for considering a revision of the AASHO specifications regarding distribution of loads to stringers.

In practically all cases where the specific objectives of the test program were not achieved, valuable information for future test projects was obtained in the matter of instrumentation and test procedure.

APPENDIX
SUMMARIZED DATA FROM BRIDGE LOADING PROGRAM

		TEST DESCRIPTION	TRUCK POSITION	BEAMS - NUMBERED NORTH TO SOUTH														REMARKS
				STRAINS MICRO IN./IN.							DEFLECTIONS - .001							
				1	2	3	4	5	6	7	1	2	3	4	5	6	7	
SPAN 1	10-11-50	Static, 1 Vehicle	4	64	56	56	45	34	22	10	93	97	98	77	47	30	12	Av. of 2 gages on bottom of
			3	57	51	54	43	34	24	10	86	93	100	82	52	34	17	lower flange of each beam.
			0	41	42	56	54	43	30	18	64	80	99	91	65	49	29	
		Moving, 1 Vehicle	4	73	57	55	41	28	19	9	94	102	104	80	48	32	11	1 ga. bot. of each beam
			3	69	55	55	45	33	31	11	88	97	105	84	51	40	13	
			0	48	43	51	50	40	26	17	63	80	102	96	66	47	18	
		Impact, 1 Vehicle	4	80	69	64	50	34	22	12	106	116	116	94	58	--	--	3/4 in. plate at mid-span
			4	73	64	61	45	33	21	12	94	105	107	86	51	--	--	1/2 in. plate at mid-span
			4	67	57	57	45	32	21	12	89	96	103	81	51	--	--	1/4 in. plate at mid-span
SPAN 2	10-10-50	Static, 1 Vehicle	4	72	55	52	45	34	21	9	119	116	112	98	62	28	18	Av. of 2 gages on bottom of
			3	66	54	51	47	37	24	13	109	113	115	102	67	35	21	lower flange of each beam.
			0	50	44	49	50	44	30	20	86	95	112	113	80	47	36	
		Moving, 1 Vehicle	4	91	63	55	44	37	21	13	109	121	107	106	62	28	19	1 gage on bottom of lower
			3	88	62	54	47	39	22	13	96	120	108	102	59	37	20	flange of each beam
			0	63	49	52	52	51	34	22	70	95	124	96	75	50	32	
		Impact, 1 Vehicle	4	103	77	69	55	45	28	21	115	141	123	115	70	--	--	3/4 in. plate at mid-span
			4	92	68	61	49	41	25	17	100	126	111	103	59	--	--	1/2 in. plate at mid-span
			4	90	67	60	50	45	26	17	97	121	112	101	57	--	--	1/4 in. plate at mid-span
SPAN 3	8-31-50	Static, 1 Vehicle	4.5	72	77	67	45	30	18	6	78	65	69	51	31	20	11	Av. of 2 ga. on bot. of bot. flange
				-1	-3	4	5	6	4	3								Av. of 2 ga. on bot. of top flange
			C.L.	29	42	56	56	45	32	16	35	38	67	79	60	45	27	Av. of 2 ga. on bot. of bot. flange
				6	3	0	-16	-6	-3	0								Av. of 2 ga. on bot. of top flange
	9-5-50	Moving, 1 Vehicle	4	63	70	65	48	30	14	7	87	80	69	57	39	20	10	1 ga. bot. of each beam
			3	54	63	63	52	33	17	8	78	78	71	61	46	23	13	
			0	38	50	62	65	44	25	15	57	64	70	69	55	27	23	
	9-6-50	Impact, 1 Vehicle	4	57	66	65	54	29	12	7	85	80	72	57	40	--	--	1/4 in. plate at mid-span
			4	59	71	70	53	34	16	11	68	74	77	62	39	--	--	1/2 in. plate at mid-span
			4	60	74	73	55	37	18	13	70	71	79	61	43	--	--	3/4 in. plate at mid-span
		Impact, Special	4	42	56	56	40	27	12	8	47	46	55	42	28	--	--	15 tons on tandems at 8.1 mph
				42	50	50	38	27	13	10	48	46	54	42	30	--	--	15 tons on tandems at 15.6 mph
				48	56	54	35	25	11	11	57	46	51	36	24	--	--	15 tons on tandems at 23.9 mph
	9-11-50	Moving, 2 Vehicles	4	69	90	95	92	95	93	78	80	87	93	110	114	84	86	Accumulated values of strains at
			3	63	91	98	99	100	95	82	73	86	95	115	108	88	88	bottom of each beam
			0	44	74	97	110	110	102	86	54	74	93	122	116	95	94	
		Impact, 2 Vehicles	4	67	90	96	99	103	99	82	80	89	100	114	110	--	--	1/4 in. plate - accumulated
			4	66	94	102	101	104	99	87	79	92	102	120	112	--	--	1/2 in. plate - accumulated
			4	69	97	104	100	108	101	91	83	93	103	121	117	--	--	3/4 in. plate - accumulated
SPAN 4	8-23-50	Static, 1 Vehicle	4.5	81	59	58	42	29	19	7	111	115	115	75	55	25	7	2 gages - bot. of each beam
			C.L.	33	38	57	59	55	38	37	48	77	111	116	108	67	40	(Av. of 2 directions)
	8-25-50	Moving, 1 Vehicle	4.5	87	66	65	48	36	19	9	126	132	123	99	63	34	12	1 ga. bot. of each beam
			C.L.	39	42	59	63	58	37	33	53	81	115	118	113	76	46	(Av. of 2 directions)
	10-9-50	Static, 1 Vehicle	4	94	68	68	55	42	26	18	136	146	140	110	70	42	17	1 ga. bot. of each beam
			3	76	59	63	53	39	26	18	118	139	138	112	75	46	19	
			0	55	49	61	56	47	31	22	87	115	131	123	91	61	32	
		Moving, 1 Vehicle	4	90	82	71	54	38	26	7	163	157	145	104	64	38	15	1 ga. bot. of each beam
			3	82	79	71	56	39	26	11	145	155	150	107	69	44	18	
			0	62	64	69	66	51	34	18	102	124	145	121	87	54	34	
		Impact, 1 Vehicle	4	84	76	68	57	39	24	12	146	148	147	111	71	--	--	1/4 in. plate at mid-span
			4	83	75	69	56	39	25	12	147	146	144	112	70	--	--	1/2 in. plate at mid-span
			4	84	78	72	58	38	28	15	143	145	147	117	74	--	--	3/4 in. plate at mid-span
	10-5-50	Static, 2 Vehicles	4	100	85	102	104	108	93	101	144	169	193	209	206	181	146	Accumulated values
			3	91	81	103	107	110	92	104	130	163	190	210	210	183	148	
			0	68	74	97	111	119	101	112	100	142	186	222	225	196	157	
		Moving, 2 Vehicles	4	98	105	109	108	110	95	99	146	186	191	199	199	182	152	Accumulated values
			3	87	97	106	110	112	99	102	129	178	190	205	207	189	157	
			0	71	87	105	119	121	107	109	102	158	189	218	221	198	168	
	10-6-50	Impact, 2 Vehicles	4	86	96	106	109	111	97	102	128	166	189	204	197	--	--	1/4 in. plate - accumulated
			4	89	97	110	109	111	102	107	130	164	191	205	199	--	--	1/2 in. plate - accumulated
			4	91	107	111	111	113	101	110	131	171	193	205	201	--	--	3/4 in. plate - accumulated
		Impact, 2 Vehicles with surcharge	4	89	100	112	117	121	107	116	131	172	200	221	218	--	--	1/4 in. plate - accumulated
			4	92	101	116	117	121	112	121	133	170	202	222	220	--	--	1/2 in. plate - accumulated
			4	94	111	117	119	123	111	124	134	177	205	222	222	--	--	3/4 in. plate - accumulated
SPAN 6	8-9-50	Static, 1 Vehicle	4.5	59	55	49	42	30	17	5	101	121	114	87	53	27	4	Av. of 3 ga. bot. of bot. flange
				-10	-36	-37	-28	-15	-5	-2								Av. of 2 ga. bot. of top flange
	8-10-50		C.L.	24	35	49	62	53	36	28	47	82	106	120	108	79	47	Av. of 3 ga. bot. of bot. flange
				-5	-21	-32	-31	-30	-16	-4								Av. of 2 ga. bot. of top flange
	8-17-50	Moving, 1 Vehicle	4.5	62	59	57	40	31	18	7	100	122	125	82	60	32	10	1 ga. bot. of bot. flange
			C.L.	25	33	46	55	48	35	28	32	67	105	96	104	66	41	

SUMMARIZED DATA (SHEET 2)

Span	Date	Test Description	Truck Position	Strains Micro In./In. 1	2	3	4	5	6	7	Deflections - .001 1	2	3	4	5	6	7	Remarks
SPAN 5 NO DIAPHRAGM BOLTS	8-18-50	Static, 1 Vehicle	4.5	70	68	66	51	32	20	12	122	133	132	96	59	24	3	Av. of 3 bot. ga. - 2 directions
				-22	-40	-38	-20	-4	5	5								Av. of 2 ga. on bot. of top flange
			C.L.	27	35	52	53	42	26	20	37	76	121	131	116	64	34	Av. of 3 bot. ga.
				-5	-19	-32	-47	-39	-28	-10								Av. of 2 ga. on bot. of top flange
	9-27-50	Static, 1 Vehicle	4	76	56	57	46	31	18	12	118	136	133	110	68	35	11	Av. of 2 ga. bot. of bot. flange
				-19	-52	-52	-35	-18	-9	-1								Av. of 2 ga. bot. of top flange
			3	72	60	64	58	40	24	16	108	128	129	113	71	41	16	Av. of 2 ga. bot. of bot. flange
				-18	-48	-48	-30	-14	-8	-1								Av. of 2 ga. bot. of top flange
			0	62	60	71	72	55	36	29	79	110	129	128	91	55	28	Av. of 2 ga. bot. of bot. flange
				-1	-30	-38	-38	-16	-7	-8								Av. of 2 ga. bot. of top flange
	8-21-50	Moving, 1 Vehicle	4.5	97	82	65	48	36	21	9	135	139	148	101	74	34	15	1 ga. bot. each beam
			C.L.	38	39	60	65	60	41	36	61	84	134	133	132	75	55	
	9-28-50	Moving, 1 Vehicle	4	87	73	69	54	39	20	9	152	145	144	120	71	39	15	1 ga. bot. each beam
			3	82	72	69	58	42	24	12	140	140	145	126	75	45	18	
			0	61	58	69	69	53	33	24	103	118	149	151	99	65	32	
		Impact, 1 Vehicle	4	88	70	65	54	37	24	12	129	131	129	124	65	--	--	1/4 in. plate at mid-span
			4	89	74	72	59	41	24	16	128	138	138	136	72	--	--	1/2 in. plate at mid-span
			4	93	79	75	62	43	27	18	137	140	146	141	78	--	--	3/4 in. plate at mid-span
	9-27-50	Static, 2 Vehicles	4	94	90	105	97	96	78	83	147	170	192	202	189	167	127	Accum. values, bot. of bot. flange
				-23	-49	-50	-54	-48	-49	-17								Accum. values, bot. of top flange
			3	77	79	100	105	94	77	80	134	164	194	209	195	173	129	Accum. values, bot. of bot. flange
				-32	-56	-56	-61	-56	-55	-23								Accum. values, bot. of top flange
			0	70	76	102	110	109	93	100	102	144	192	220	212	187	142	Accum. values, bot. of bot. flange
				-7	-39	-56	-67	-55	-49	-10								Accum. values, bot. of top flange
	9-28-50	Moving, 2 Vehicles	4	103	104	101	97	96	77	84	165	176	205	210	192	168	130	Accum. values, bot. of bot. flange
			3	92	91	100	102	98	80	86	150	169	205	215	196	175	131	
			0	73	80	101	113	110	89	95	114	152	206	228	210	186	140	
		Impact, 2 Vehicles	4	95	92	100	100	98	79	87	156	179	198	213	199	168	130	1/4 in. plate at mid-span
			4	97	100	108	104	100	81	90	167	188	206	219	204	173	130	1/2 in. plate at mid-span
			4	107	106	114	107	102	85	94	178	199	216	225	207	186	140	3/4 in. plate at mid-span
	9-29-50	Impact, 2 Vehicles with surcharge	4	109	108	120	115	112	95	99	185	200	227	245	228	197	147	3/4 in. plate at mid-span
				103	102	115	109	108	89	94	177	190	218	236	221	201	149	1/2 in. plate at mid-span
				102	97	108	107	107	89	91	176	188	214	233	215	201	151	1/4 in. plate at mid-span
SPAN 5 DIAPHRAGMS SINGLE BOLTED	8-23-50	Static, 1 Vehicle	4.5	83	69	62	46	32	18	6	131	128	129	97	65	32	12	Av. of 2 ga. on bot. of bot. flange
				-24	-29	-36	-17	-5	3	3								Av. of 2 ga. on bot. of top flange
			C.L.	36	36	52	61	53	37	32	48	74	116	126	114	77	49	Av. of 2 ga. on bot. of bot. flange
				0	-20	-36	-38	-32	-17	2								Av. of 2 ga. on bot. of top flange
		Moving, 1 Vehicle	4.5	84	69	57	48	37	20	9	130	135	127	114	70	35	12	1 ga. bot. of bot. flange
			C.L.	38	41	54	49	57	40	37	41	75	116	136	116	78	56	
		Moving, 1 Vehicle	4	71	67	62	49	34	18	10	106	116	112	102	72	40	15	1 ga. bot. of bot. flange
			3	65	64	65	52	37	21	13	63	111	112	106	77	46	19	
			0	49	53	61	59	45	29	22	28	94	108	119	95	62	29	
	9-25-50	Impact, 1 Vehicle	4	94	74	66	51	34	21	14	146	137	132	101	65	--	--	1/4 in. plate
			4	96	76	67	49	32	19	12	145	136	129	100	64	--	--	1/2 in. plate
			4	102	83	75	59	36	23	15	154	145	140	111	75	--	--	3/4 in. plate
			4	97	76	70	53	34	23	11	149	138	134	102	67	--	--	No plate
	9-22-50	Static, 2 Vehicles	4	73	74	87	89	95	79	86	147	171	186	193	189	164	144	Av. of 2 ga. bot. of bot. flange
				-21	-43	-48	-49	-46	-34	-31								Av. of 2 ga. bot. of top flange
			3	78	87	99	97	100	95	95	132	163	188	199	195	167	148	Av. of 2 ga. bot. of bot. flange
				-11	-32	-37	-42	-39	-37	-28								Av. of 2 ga. bot. of top flange
			0	61	74	96	114	113	110	109	100	142	187	210	210	185	159	Av. of 2 ga. bot. of bot. flange
				-2	-19	-34	-46	-41	-34	-28								Av. of 2 ga. bot. of top flange
	9-21-50	Moving, 2 Vehicles	4	93	93	102	99	100	92	86	142	173	191	191	182	154	145	1 ga. bot. of beam
			3	82	91	102	105	105	97	92	128	163	192	200	195	165	145	
			0	65	80	98	111	112	104	99	105	146	189	209	211	176	153	
	9-21-50	Impact, 2 Vehicles	4	83	93	102	102	101	94	94	133	164	189	197	181	--	--	1/4 in. plate at mid-span
			4	90	100	109	108	103	96	99	140	176	195	202	189	--	--	1/2 in. plate at mid-span
			4	86	98	107	106	104	98	98	138	173	201	209	196	--	--	3/4 in. plate at mid-span
		Impact, 2 Vehicles	4	92	104	116	114	117	109	112	148	184	211	222	223	--	--	3/4 in. plate at mid-span
SPAN 5 DIAPHRAGMS DOUBLE BOLTED	10-4-50	Static, 1 Vehicle	4	79	71	70	56	39	26	20	145	137	130	109	72	41	16	Av. 2 ga. bot. of bot. flange
			4	-30	-49	-48	-27	-10	-2	3								Av. 2 ga. bot. of top flange
			3	69	64	64	57	36	28	15	135	133	131	116	78	47	19	Av. 2 ga. bot. of bot. flange
				-31	-47	-48	-32	-14	-2	-1								Av. 2 ga. bot. of top flange
			0	49	51	59	56	46	29	23	99	109	128	128	92	64	34	Av. 2 ga. bot. of bot. flange
				-21	-40	-49	-24	-12	-1	--								Av. 2 ga. bot. of top flange
		Moving, 1 Vehicle	4	83	76	71	53	41	25	10	158	152	131	106	67	40	16	1 ga. bot. of bot. flange
			3	77	74	71	58	43	25	14	133	150	140	113	76	47	20	
			0	57	61	69	65	51	32	25	100	129	134	125	91	60	30	
		Impact, 1 Vehicle	4	77	72	69	55	40	23	14	132	126	124	104	68	--	--	1/4 in. plate at mid-span
			4	79	65	74	56	42	25	16	134	129	128	113	74	--	--	1/2 in. plate at mid-span
			4	89	82	84	66	48	29	21	147	144	143	127	71	--	--	3/4 in. plate at mid-span (accum. value)
	10-3-50	Static, 2 Vehicles	4	99	89	102	105	100	89	88	155	175	192	208	194	171	159	Av. 2 ga. bot. of bot. flange
				-30	-52	-56	-54	-62	-59	-39								Av. 2 ga. bot. of top flange
			3	97	90	110	112	109	97	88	141	166	191	213	196	174	162	Av. 2 ga. bot. of bot. flange
				-18	-44	-49	-55	-58	-51	-32								Av. 2 ga. bot. of top flange
			0	77	88	110	123	122	111	104	109	147	188	225	212	186	170	Av. 2 ga. bot. of bot. flange
				-2	-27	-42	-58	-52	-54	-31								Av. 2 ga. bot. of top flange
		Moving, 2 Vehicles	4	104	101	108	100	109	94	89	177	191	203	227	193	175	160	1 ga. bot. of bot. flange (accum.)
			3	97	99	108	104	113	98	91	163	188	205	235	196	176	160	
			0	77	85	104	108	120	106	97	126	163	198	243	204	185	164	
	10-13-50	Impact, 2 Vehicles	4	97	104	117	107	117	101	98	175	186	214	247	209	--	--	3/4 in. plate at mid-span
			4	93	99	113	104	115	100	94	161	173	205	237	204	--	--	1/2 in. plate at mid-span
			4	93	97	106	103	113	99	92	159	169	201	233	204	--	--	1/4 in. plate at mid-span
		Impact, 2 Vehicles with surcharge	4	96	105	113	124	122	111	102	163	178	210	222	223	--	--	1/4 in. plate at mid-span
			4	97	106	118	122	123	111	104	166	183	213	227	227	--	--	1/2 in. plate at mid-span
			4	104	116	126	131	129	117	111	181	196	225	236	234	--	--	3/4 in. plate at mid-span