Building research station, Garston, Watford, Herts: research on the strength of bridges: investigation on bridge deck systems

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b) Investigation on Bridge Deck Systems

Untersuchungen an verschiedenen Brückenfahrbahnen

Investigations sur divers types de tabliers de ponts

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General

The main research work now in hand at the Building Research Station in connection with bridges concerns modern steel and concrete bridge deck systems. Three types are being studied:

a) A series of steel joists supporting a reinforced concrete slab.
b) As a) except that the joists are encased in concrete, and concrete jack-arches span between the joists to support the slab; and
c) A filler joist system, with the steel joists completely encased in the concrete slab.

The important features which require investigation in all three cases are, first, the extent to which the system provides lateral distribution, whereby the effects of a concentrated load (such as a wheel load) are shared by a much greater width of the system than that over which the load is directly applied; and second, the extent to which the steel joists and the concrete slab act together in structural combination in resisting the induced bending moments.

The work is divided into three general sections:

I. Tests on actual bridges;
II. Tests on bridge deck systems in the laboratory;
III. Mathematical analysis, supplemented by small-scale model analysis.

A few modern bridges have been tested in this programme and the most illuminating result has been the low values for the measured strains due to specially heavy traffic; so low, indeed, that further development of extra-sensitive strain gauges coupled with the use of very high test loads is necessary before an accurate picture can be obtained of the moment distribution in such bridges. It is at any rate clear that important economies may well result from a greater knowledge of the action of these bridges under load.

The laboratory tests are being made on scale models of bridge decks, the scale being about 1:3 or 1:4, except in the case of the filler joist system, where the scale is about 1:2. The span adopted throughout for all the models is 9 ft., the model being supported within a steel framework so that a concentrated load can be applied at any point of the slab. Some preliminary observations of the tests on decks of types a) and c) are given below.

Test on a model consisting of steel joists supporting a reinforced concrete slab

Construction of the model. The first model consisted of a reinforced concrete slab deck, 3 in. thick, supported on standard 8 in. x 4 in. longitudinal beams. Transverse stiffening joists were bolted to the beams at mid-span and over

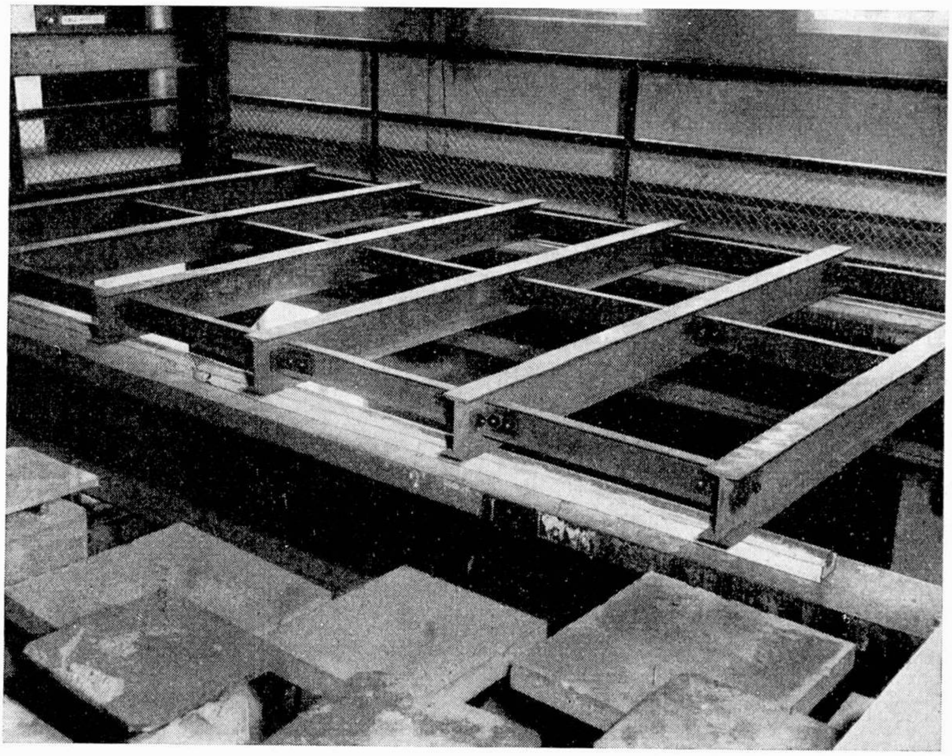

Fig. 1.

each support. The main beams and transverse stiffeners were first assembled on the steel testing frame over a span of 9 ft. (see Figure 1). 2 in. diameter steel bars partly embedded in concrete were used as bearers on the main supports.

The six main beams were spaced at 3 ft. centres, the end beams being at the ends of the concrete slab which was 15 ft. 4 in. wide.

The reinforced concrete slab was cast independent of the steel joist system, using a concrete with a strength of about 5000 lb. per sq. in. (4 in. cube) at 28 days. At an age of 10 days from casting, the slab was lifted into position and bedded down on mortar on the steel beams. In this way, the help given by the constructional method towards the structural interaction between the slab and the beams when under load was the minimum likely to be obtained in practice.

Testing Arrangements. Load was applied to the model by means of a hydraulic jack bearing against the main framework as shown in Figure 2 and was measured by means of a 75 ton steel proving ring, reading to an accuracy of about 70 lb. The load was transferred to the slab through a 3 in. diameter steel disc 1 in. thick bedded to the slab with plaster of Paris. A special disc with a groove along a diameter was used when it was necessary to measure strains in the concrete directly under the load; the disc was thereby kept clear of the strain gauge.

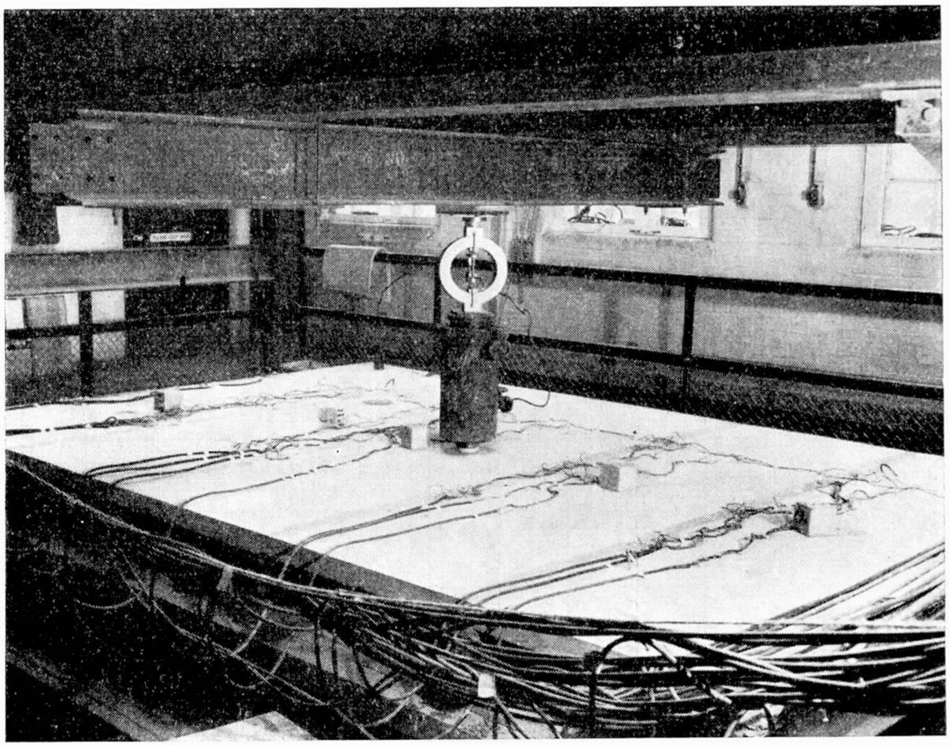

Fig. 2.

Strains in the beams, the concrete and the reinforcing bars were measured at a large number of positions throughout the system, using electrical wire resistance gauges. The deflections of beams and slab were recorded by means of dial indicators registering to 0,0001 in.

A complete set of strain and deflection readings was taken with the load at the mid-span and quarter-span points of each beam, and with the load at mid-span and quarter-span on the centre line of each panel. Owing to the symmetry of the model, it was, however, necessary to arrange the loading positions over only one quarter of the slab. In this paper, the results given apply in all cases to loading at mid-span.

The tests were made in three stages:

a) Load applied to the slab in its uncracked condition;
b) Load applied to the slab after it had been systematically cracked;
c) Final loading to destruction.

Test on uncracked model. The first loadings were restricted so that visible cracking of the slab did not occur.

Figure 3 shows the strains at mid-span in the bottom flanges of the beams plotted as influence lines for various positions, along the transverse centre-line, of a concentrated load of 10,000 lb. On the assumption that the bending moments in the beams are proportional to the strains in the lower flanges, the percentage distributions of bending moment between the beams for various loading positions have been calculated, and are given in Table 1.

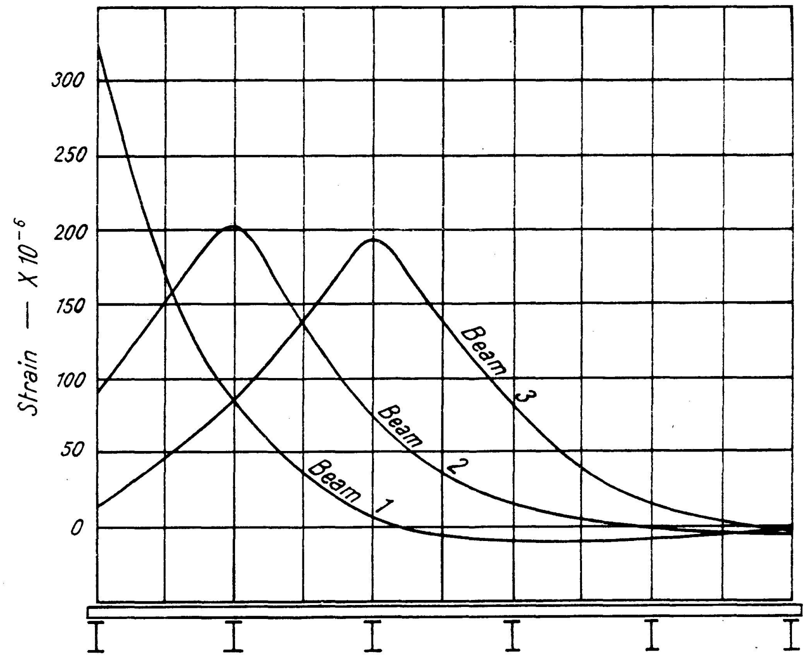

Fig. 3.
Influence Lines for Strains at Mid-Span in Bottom Flanges. Load 10,000 Lb.

Table I.
Percentage Distribution of Bending Moment between Beams for Load Along Transverse Centre Line of Model a) (uncracked slab)

Position of Load*)	Percentage of total bending moment resisted by Beam No.:					
	1	2	3	4	5	6
Beam 1	78	21	3,5	–1	–1,5	0
Panel 1 (between beams 1 and 2)	46	42	12,5	1,5	–1	–1
Beam 2	22	53,5	22,5	4,5	–1	–1,5
Panel 2 (between beams 2 and 3)	10,5	39	40	11,5	1	–2
Beam 3	2	21	53	22	4	–2
Panel 3 (between beams 3 and 4)	–1	11	40	41	11	–2

*) The beams are numbered 1—6 from one side to the other of the model.

It will be seen from Table 1 that for positions of the load other than on the side panels or edge beams:

I. With the load central on a panel, the adjacent beams each take 40 per cent. of the total bending moment, and the next adjacent beams each take 10 per cent;

II. With the load over a beam, that beam takes just over 50 per cent. of the total bending moment, and the adjacent beams each take just over 20 per cent.

When the load is applied nearer the sides of the model, the local strains are increased and in the worst position, with the load over the edge beam, that beam takes 75—80 per cent. of the total bending moment. Conditions as severe as these are not likely to arise in practice since the roadway does not usually extend to the edge beams and also these beams are commonly stiffened by the parapet.

These percentages do not refer to the actual proportions of loads carried by the beams, but only the distribution of moments between them. The further a beam is from the point of application of the load, the greater is the length of beam over which the proportion of load carried by the beam is effectively distributed; hence the maximum bending moment produced in a beam for a given proportion of load carried by that beam gets less as we move away from the applied load.

From a comparative study of the strains measured in the upper and lower flanges of the beams it was found that some partial composite action existed between the beams and the slab. The effect was more pronounced for beams near to the load, suggesting that what composite action occurred was primarily due to friction between the slab and the flanges of the beams.

Although the extent of structural interaction between slab and beams was far from complete, it was found nevertheless that the experimental values for the maximum strains in the interior beams agreed approximately with the strains calculated on the assumptions I. that there was complete composite action between each beam and the full panel width of slab (i.e. the width between beams) and II. that the beam and the full panel width have the same curvature. Actually, there was additional local longitudinal bending of the slab in the vicinity of the load and it appears that the consequent error in the assumption II. compensates roughly for the over-allowance for composite action in assumption I.

The transverse strains in the slab at the centres of the panels are shown as influence lines in Figure 4. Influence lines of similar shape were obtained for the transverse strains over the beams but the maximum values under the load were less than half those at the centres of the panels. With the load in a panel, the hogging moments in the slab over an adjacent beam were only about one-tenth of the sagging moment under the load.

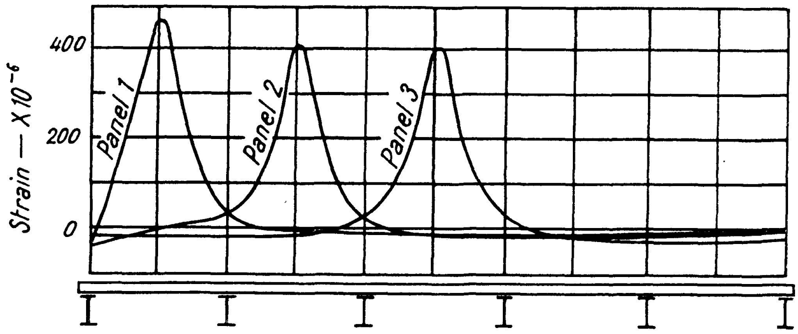

Fig. 4.
Influence Lines for Transverse Strains in Concrete at Panel Centres (Load 10,000 Lb.).

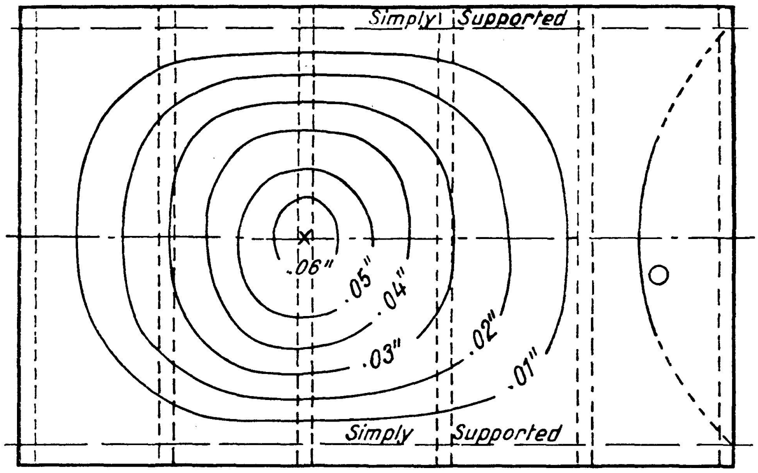

Fig. 5.
Deflection Contours for Load of 10,000 Lb. at Centre of No. 3 Beam.

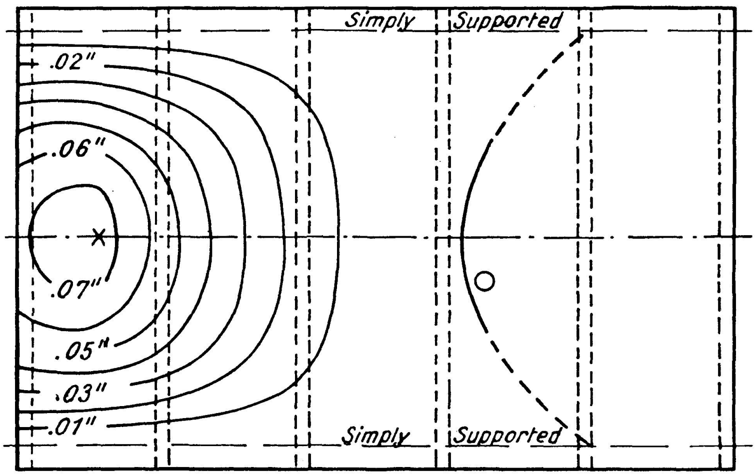

Fig. 6.
Deflection Contours for Load of 10,000 Lb. at Centre of No. 1. Panel

A fairly complete picture of the deflected shape of the model was obtained for all positions of the load. Figures 5 and 6 are typical deflection contour diagrams. It was found that if the deflections at mid-span of the beams were assumed to be proportional to the bending moments in the beams, the distribution of moment amongst the beams estimated from the deflections agreed extremely well with that deduced from the strain measurements on the bottom flanges of the beams.

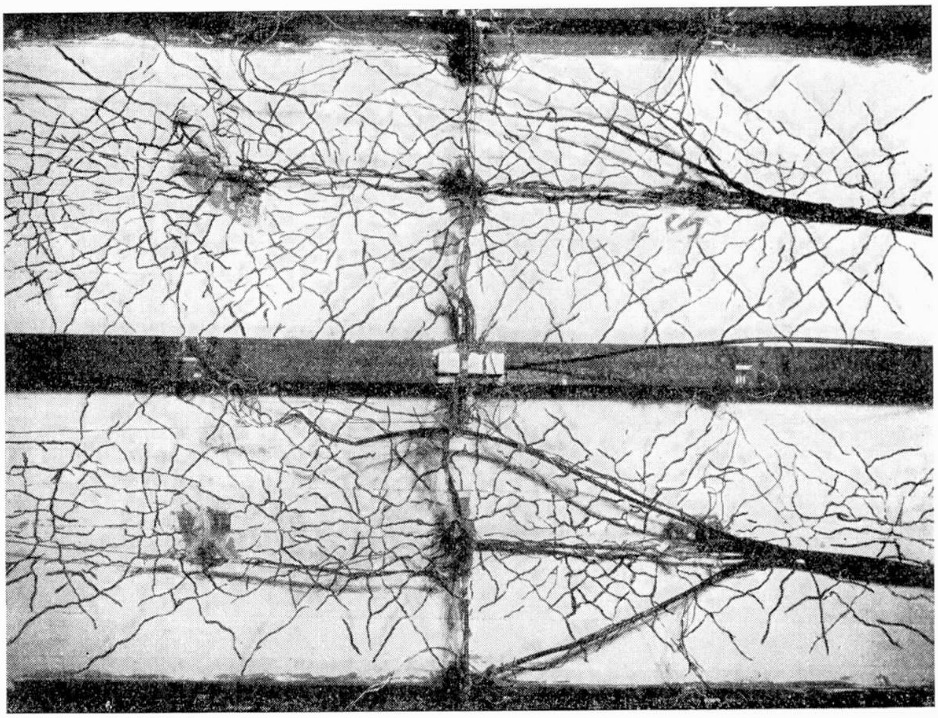

Fig. 7.

Test on cracked model. The slab was cracked by the application of loads of approximately 20,000 lb. through a $4\frac{1}{2}$ in. diameter steel disc, at 12 in. centres along the longitudinal centre line of each panel. The cracking was marked on the underside of the slab and Figure 7 shows two typical panels after cracking. The cracks were painted in for the photograph; they were actually hair cracks, nowhere more than 0,01 in. wide.

Figure 8 shows the strains at mid-span in the bottom flanges of the beams plotted as influence lines for various positions, along the transverse centre-line, of a concentrated load of 10,000 lb. Comparing these with the influence lines for the uncracked slab given in Figure 3, it will be observed that the beam strains were increased considerably as a result of the reduced stiffness of the slab. The distributions of bending moment between the beams are set out in Table 2. There is clearly less lateral distribution than with the uncracked slab, the characteristic distribution figures for the two cases of

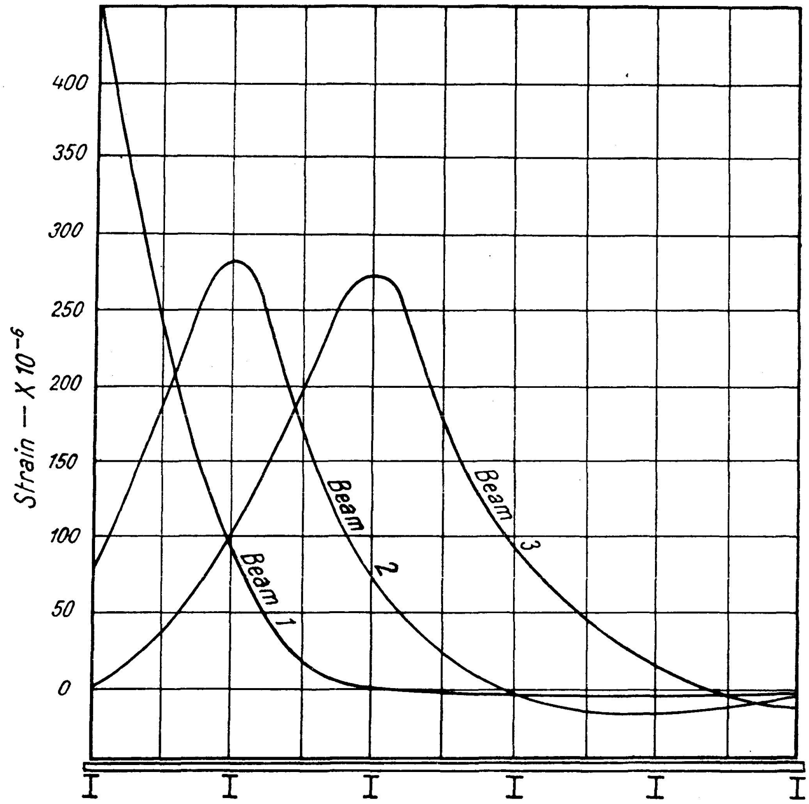

Fig. 8.
Influence Lines for Strains at Mid-Span in the Bottom Flanges of Beams. Load 10,000 Lb. on Cracked Model.

Table 2.
Percentage Distribution of Bending Moment between Beams for Load along Transverse Centre Line of Model a) (cracked slab)

Position of Load	Percentage of total bending moment resisted by Beam No.:					
	1	2	3	4	5	6
Panel 3 (between beams 3 and 4)	0	7	45	42	6	0
Beam 4		–1	20	64	17	0
Panel 4 (between beams 4 and 5)	0	–3	11	47	42	3
Beam 5	0	–3	3	21	61	18
Panel 5 (between beams 5 and 6)	–1	–2	0	8	43	52
Beam 6	–1	–1	–2	0	18	86

beam loading and panel loading being 19–62–19 and 5–45–45–5 compared with 22–53–22 and 10–40–40–10 for the uncracked slab. With the load over an outside beam, that beam takes 86 per cent. of the total bending moment in all the beams, compared with 78 per cent. before cracking.

A study of the strain measurements indicated that there was slightly less composite action between slab and beam as a result of cracking. The strains in the concrete were in all cases considerably increased, by 50—100 per cent. The deflections of the model were similarly affected; typical deflection contour diagrams are given in Figures 9 and 10. The distribution of bending moment

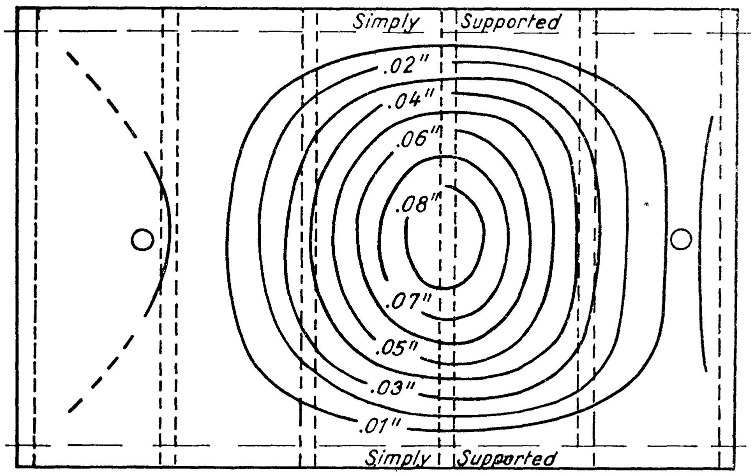

Fig. 9.
Deflection Contours for Load of 10,000 Lb. on Cracked Model at Centre of Beam No. 4.

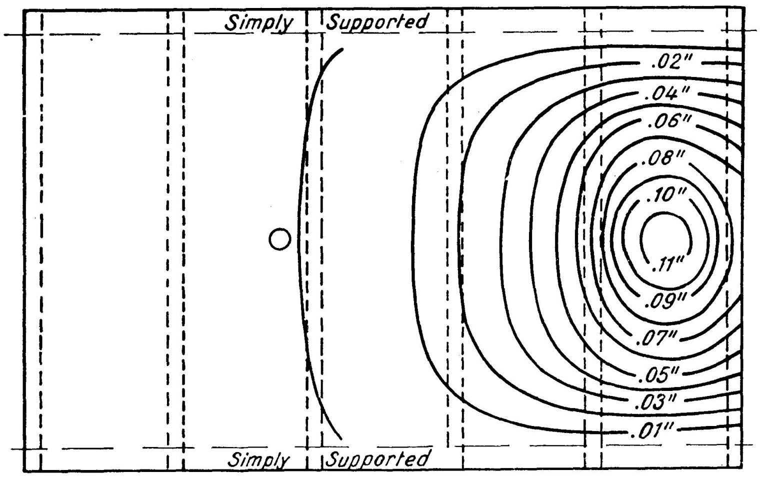

Fig. 10.
Deflection Contours for Load of 10,000 Lb. on Cracked Model at Centre of Panel No. 5.

amongst the beams estimated from the deflections again agreed fairly closely with that based on the strain measurements on the bottom flanges of the beams.

Test to destruction. Primary failure of the model was caused by the application of a load through a 3 in. diameter steel disc bedded down on plaster of Paris immediately over Beam 4 mid-span. At a load of about 40,000 lb., the strain in the flanges of this beam increased sharply beyond a value of $1500 . 10^{-6}$

indicating that the yield strength of the beam had been reached. At a load of 45,100 lb. the slab immediately under the load was deprived of any effective help from the beam and failed by punching. A view of the underside of the slab (see Figure 11) shows the form of failure. The volume punched through was approximately in the form of a truncated cone of 18 in. diameter at the bottom and 3 in. diameter on the top surface of the slab. No crushing of the slab was visible on top.

There was a tendency during the test for the proportion of bending moment carried by the beam immediately beneath the load to increase as failure was approached. At failure, the maximum stress in the adjacent beams (nos. 3

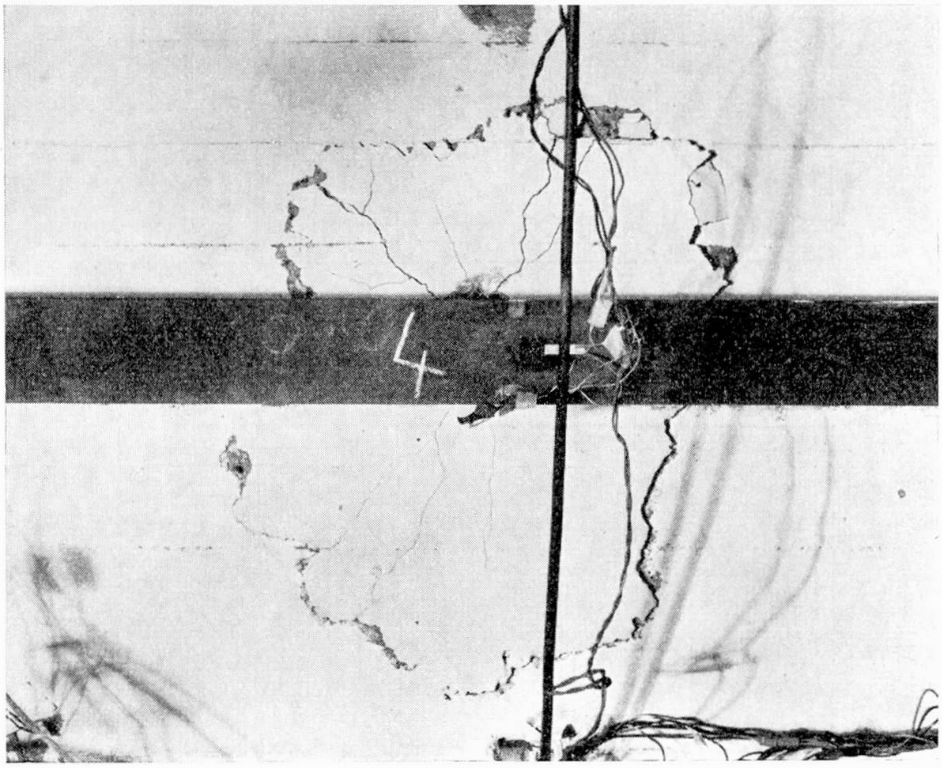

Fig. 11

and 5) was only about 6 tons per sq. in., and it is clear that for the conditions chosen, no appreciable gain in load resulted from the effect of plasticity of the steel, in redistributing the moments after yielding had started in the beam immediately under the load.

After the primary destruction test on the model, five punching failure tests were made in Panels 3 and 5. The load was applied through a 3 in. diameter disc as before. The volumes punched through were very similar to that in the earlier test when the load was applied over a beam. At a load of about 14,000 lb. yield occurred in the reinforcing bars, as indicated by strain measurements beneath the loaded area. Final failure occurred at about 21,000 lb.

Test on a model filler-joist system with the steel beams completely encased in the concrete slab

Construction of the model. The second series of tests was made on a half-scale model of a typical filler-joist bridge deck. It consisted of sixteen standard 6 in. x 3 in. beams completely embedded at 12 in. centres in a concrete slab, 9 in. thick, which was reinforced by light welded fabric above and below the beams. While casting the slab, the timber mould was suspended from the 6 in. x 3 in. beams which thereby carried the full dead weight of the concrete.

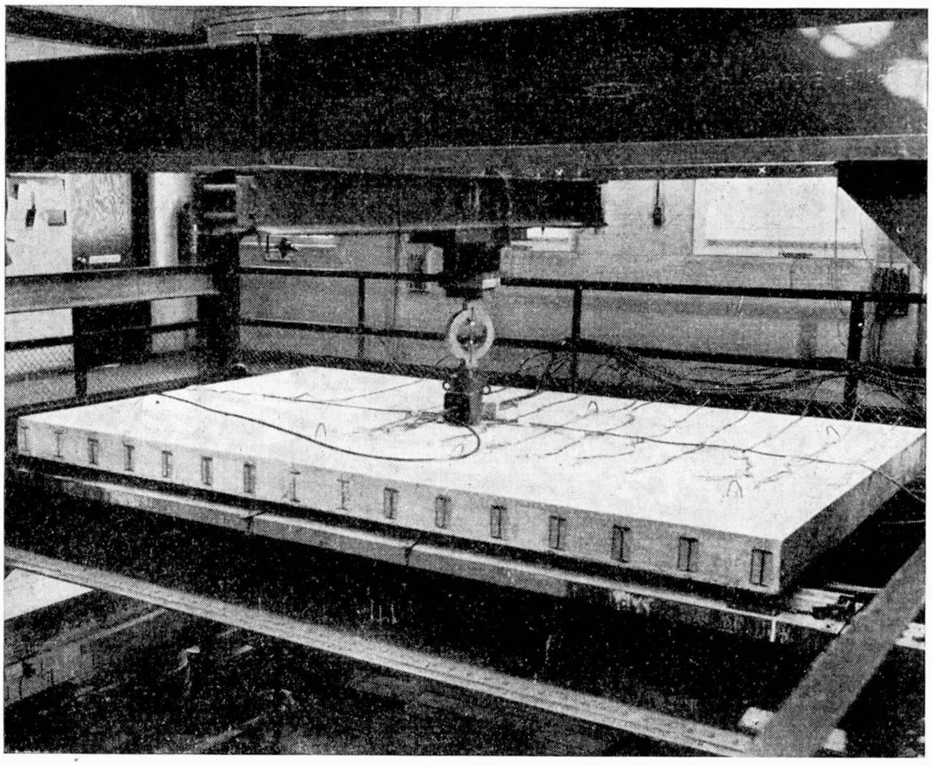

Fig. 12.

The general testing arrangements were the same as for the previous model (see Figure 12).

The results quoted here for the slab in both its uncracked and cracked conditions were obtained from eight positions of the load along the transverse centre-line of the model and are combined in one section for convenience in comparison.

Tests prior to destruction. The strains at mid-span in the bottom flanges of three of the beams are plotted as influence lines in Figures 13 and 14 for various positions, along the transverse centre line, of the concentrated load. Tables 3 and 4 show the percentage distribution of bending moment between the beams on the assumption that the bending moments in the beams are directly proportional to the mid-span strains in the bottom flanges.

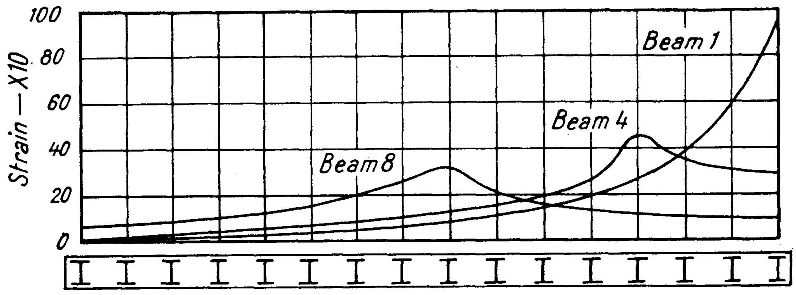

Fig. 13.
Influence Lines for Strains at Mid-Span in Bottom Flange of Beams for Load of 10,000 Lb. along Transverse Centre Line of Uncracked Slab.

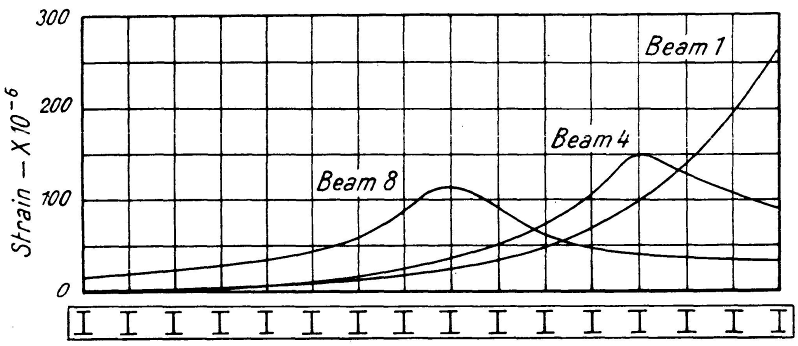

Fig. 14.
Influence Lines for Strains at Mid-Span in Bottom Flange of Beams for Load of 10,000 Lb. along Transverse Centre Line of Cracked Slab.

It appears from Tables 3 and 4 that the proportion of the total bending moment carried by the most highly stressed beam is fairly constant at a value between one sixth and one eighth with the load anywhere on the central half of the model and, as would be expected, increases as the load approaches the edge to a maximum value of one third with the load over the outside beam. There was little significant change in distribution of the bending moment on cracking; the close spacing of the beams and the high stiffness ratio of the slab to the beams as compared with model a) would account for this.

It should be noted here that the sum of the mid-span strains in the beams for any particular load position increases as the load approaches the edge of the model. As a result, the actual maximum recorded strain in the outside beam bears a greater ratio to the maximum recorded strain in an interior beam than would appear from Tables 3 and 4. The actual maximum recorded strains are shown in Figures 13 and 14.

The assumption of complete composite action between the concrete and the filler joists was found to lead to a reasonable (but slightly high) estimate of the sum of the strains in the beams at mid-span.

Tests to destruction. Five tests to destruction were made on the model, three with the load midway between two beams and two with the load directly over a beam.

Table 3. Percentage Distribution of Bending Moment between Beams for Load along Transverse Centre Line of Model c) (uncracked slab)

Position of Load	Percentage of total bending moment resisted by Beam No.:															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Beam 1	30	20	13	10,5	7	5	4	3	2	1,5	1	1	0,5	0,5	0,5	0,5
Panel 1 (between Beams 1 and 2)	25,5	21,5	14	10,5	7,5	5	4	3,5	2,5	1,5	1,5	1	0,5	0,5	0,5	0,5
Beam 4	12	12,5	14	17,5	11,5	7	6	5	3,5	2,5	2	2	1,5	1	1	1
Panel 4 (between Beams 4 and 5)	10,5	11,5	12	16	14	8	6,5	5,5	4	3	2	2	1,5	1,5	1	1
Beam 7	5	5,5	5,5	7	8	10	15,5	12	8	5,5	4,5	4	3	2,5	2	2
Panel 7 (between Beams 7 and 8)	4	4,5	5	6	7	8,5	14,5	15	9	6	5	4	3,5	3	2,5	2,5
Beam 8	3,5	3,5	3,5	4,5	5,5	6,5	11,5	16	12,5	8	6,5	5	4	3,5	3	3
Panel 8 (between Beams 8 and 9)	3	3	3,5	4,5	5	6	9,5	12,5	15,5	9	7	6	5	4	3,5	3

Table 4. Percentage Distribution of Bending Moment between Beams for Load along Transverse Centre Line of Model c) (cracked slab)

Position of Load	Percentage of total bending moment resisted by Beam No.:															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Beam 1	32	19,5	12	9,5	6,5	5	4,5	4	2,5	1,5	1	0,5	0,5	0,5	0,5	
Panel 1 (between Beams 1 and 2)	27,5	21	13	11	7,5	5,5	4	3	2	2	1	0,5	0,5	0,5	0,5	0,5
Beam 4	10,5	11	12	19	11,5	7,5	6	4,5	4	3	2,5	2	2	1,5	1,5	1,5
Panel 4 (between Beams 4 and 5)	10	10	10,5	14	14,5	8,5	7,5	5,5	5	3,5	2,5	2	2	1,5	1,5	1,5
Beam 7	5	5	5,5	6,5	7,5	9,5	16	9	8,5	5,5	5	4	4	3,5	3	2,5
Panel 7 (between Beams 7 and 8)	44,5	4,5	5,5	5	6,5	8	13,5	12	10	6,5	5,5	4,5	4,5	3,5	3,5	3
Beam 8	4	3,5	4	4,5	5,5	6,5	9,5	14	12,5	7,5	6	5,5	5	4,5	4	3,5
Panel 8 (between Beams 8 and 9)	3	3,5	3,5	4	5	6	8	12,5	15	8,5	7	6,5	5,5	4,5	4	3,5

With the load between two beams, final failure occurred due to punching at an average load of 48 tons, after yield had been reached in two and possibly four of the adjacent beams.

With the load over a beam, that beam yielded at an average load of 39 tons. No movement of the beam relative to the slab occurred up to this point. When the beam yielded the concrete failed in diagonal tension along the planes joining the top flange of the loaded beam to the lower flanges of the adjacent beams and the concrete within these planes deflected subsequently with the loaded beam. After yield the load dropped back to 10—15 tons and a steady deflection of the beam and attached concrete took place (up to a maximum of 9 in.); at the same time the ends of the loaded beam pulled out, the movement relative to the edges of the slab reaching a maximum of 1 in.

Summary

The paper describes research work now in hand at the Building Research Station in connection with modern steel and concrete bridge deck systems. Three types are being studied:

a) A series of steel joists supporting a reinforced concrete slab;
b) As a) except that the joists are encased in concrete and concrete jack-arches span between the joists to support the slab; and
c) A filler joist system, with the steel joists completely encased in the concrete slab.

The first two types are likely to be used extensively in the future; the third type is likely still to be used for short spans, up to about 15 ft. The factors which are being investigated in all three cases are, first, the extent to which the system provides lateral distribution, whereby the effects of a concentrated load (such as a wheel load) are shared by a much greater width of the system than that over which the load is directly applied; and, second, the extent to which the steel joists and the concrete slab act together in structural combination in resisting the induced bending moments.

The work is divided into three general sections:

I. Tests on actual bridges;
II. Tests on bridge deck systems in the laboratory;
III. Mathematical analysis, supplemented by small-scale model analysis.

A few modern bridges have been tested in this programme and the most illuminating result has been the low values for the measured strains due to specially heavy traffic; so low, indeed, that further development of extra-sensitive strain gauges coupled with the use of very high test loads is necessary before an accurate picture can be obtained of the moment distributions in such bridges .It is at any rate clear that important economies may well result from a greater knowledge of the action of these bridges under load.

The laboratory tests are being made on scale models of bridge decks, the scale being about 1:3 or 1:4, except in the case of the filler joist system where the scale is about 1:2. The span adopted throughout for the models is 9 ft., the model being supported within a steel framework so that a concentrated load can be applied at any point of the slab. In each test, over one hundred electrical resistance strain gauges are used for measuring the strain distribution in the concrete and in the steel joists.

These tests have shown that a considerable degree of lateral distribution exists even when the concrete slab has cracked. Also, even when a slab is cast on top of uncased steel joists without mechanical anchorage to aid the interaction of the steel and concrete, friction is sufficient to induce appreciable composite action. However, some special shear connectors fixed to the joists appear to be desirable if full allowance for such action is to made be.

Zusammenfassung

Der Bericht beschreibt die gegenwärtigen Untersuchungen der Building Research Station an modernen Stahl- und Betonfahrbahnen. Es wurden drei Arten untersucht:

a) Eisenbetonplatte getragen von Stahlträgern;
b) wie a) aber mit einbetonierten Trägern und bogenförmigen Querträgern aus Eisenbeton, die die Platte tragen;
c) ein Trägerrost, wobei alle Stahlträger in die Betonplatte einbetoniert wurden.

Die ersten zwei Typen werden in Zukunft viel angewendet werden, der dritte wird immer noch gebraucht für kleine Spannweiten bis ungefähr 15 Fuß. Die Faktoren, die in allen Fällen speziell untersucht wurden, sind:

1. die Lastverteilung in Querrichtung, wobei ein weit größerer Teil des Systems sich an der Aufnahme der Spannungen einer konzentrierten Last (z.B. eine Radlast) beteiligt, als nur der direkt darunterliegende Bereich; und
2. das Zusammenwirken der Stahlträger mit der Betonplatte bei Biegung.

Die Arbeit wurde in drei Sektionen unterteilt:

I. Messungen an bestehenden Brücken;
II. Messungen an Fahrbahnsystemen im Laboratorium;
III. Analytische Untersuchungen ergänzt durch Messungen an Modellen in kleinem Maßstab.

Das überraschendste Ergebnis der Messungen an einigen modernen Brücken waren die kleinen gemessenen Spannungen, selbst für schwersten Verkehr. Sie waren so klein, daß für die genauere Abklärung der Momentenverteilung in solchen Systemen besonders empfindliche Meßinstrumente und sehr hohe Versuchslasten angewendet werden müssen. Es kann aber jetzt schon gesagt

werden, daß die bessere Kenntnis des Verhaltens dieser Fahrbahnen unter Verkehrslasten wichtige Materialeinsparungen ermöglichen wird.

Die Laboratoriumsversuche für a) und b) wurden an Modellen im Maßstab 1:3 oder 1:4, für den Trägerrost im Maßstab 1:2 vorgenommen. Die Spannweite der Modelle war durchgehend 9 Fuß. Durch eine spezielle Stahlrahmenkonstruktion war es möglich, die Einzellasten an jeder beliebigen Stelle anzubringen. Für jeden Versuch wurden über hundert elektrische Spannungsmesser eingebaut, um die Spannungsverteilung im Beton und in den Stahlträgern zu bestimmen.

Die Messungen zeigten, daß eine beträchtliche Querverteilung vorhanden ist, auch wenn die Betonplatte gerissen ist. Sogar wenn die Platte auf Stahlträger ohne Dübel gegossen wird, ergibt die Reibung noch ein bestimmbares Zusammenwirken.

Trotzdem sind Dübel auf den Stahlträgern anzuordnen, wenn völliges Zusammenwirken erreicht werden soll.

Résumé

L'auteur expose les recherches qui ont été faites à la Building Research Station sur des tabliers modernes en acier et béton. Trois types ont été soumis à ces investigations:

a) Dalles en béton armé supportées par des poutres en acier.
b) Dalles en béton armé supportées par des poutres du même matérial, raidies par des éléments transversaux arqués en béton armé également.
c) Grille portante, constituée par des poutres en acier noyées dans une dalle de béton.

Les deux premiers types sont appelés à être largement employés dans l'avenir. Le troisième type sera toujours utilisé pour les petites portées jusqu'à l'ordre de 4 à 5 m.

Les points sur lesquels ont particulièrement porté les investigations sont les suivants:

1. Répartition de la charge en direction transversale.
2. Comportement combiné des poutrelles en acier et de la dalle de béton dans le cas de la flexion.

Les recherches ont été divisées en trois parties:

I. Mesures sur ponts existants.
II. Mesures effectuées en laboratoire sur diverses dispositions de tabliers.
III. Etudes analytiques complétées par des mesures sur modèles à échelle réduite.

Les mesures effectuées sur quelques ponts modernes ont fourni un résultat remarquable: les contraintes mesurées sont faibles même pour les conditions

de trafic les plus sévères. Elles sont si faibles que pour établir d'une manière précise la distribution des moments dans ces systèmes, il a fallu employer des instruments de mesure particulièrement sensibles et recourir à des charges d'essai très élevées. On peut toutefois d'ores et déjà indiquer que la connaissance plus approfondie du comportement de ces tabliers sous les charges roulantes permettra de réaliser de substantielles économies de matériaux.

Les essais de laboratoire concernant les dispositions a) et b) ont été faits sur des modèles à l'échelle de 1:3 ou de 1:4; pour les grilles c), on a adopté l'échelle 1:2. La portée de tous les modèles était de 2,70 m. Un dispositif spécial comportant un cadre en acier permettait d'appliquer les charges individuelles en tous points voulus. Dans chaque essai, on a utilisé plus de cent extensomètres électriques pour la détermination des contraintes dans le béton et dans les poutres en acier.

Ces mesures ont mis en évidence une très large distribution transversale, même en cas de fissuration de la dalle de béton. On constate encore un comportement combiné très net entre la dalle et les poutres en acier du fait de frottement, même lorsque la dalle n'est pas rendue solidaire des poutres par un goujonnage. Il y a néanmoins lieu de prévoir un goujonnage lorsque l'on veut réaliser la coopération intégrale des deux éléments béton et acier.

c) Problems of Impact and Fatigue and their Effect on Permissible Stresses in Cast Iron Girder Bridges

Stoß- und Ermüdungsprobleme und ihre Auswirkungen auf die zulässigen Spannungen in Balkenbrücken aus Gußeisen

Les questions de choc et de fatigue et leurs répercussions sur les contraintes admissibles dans les ponts à poutres en fonte

G. R. Mitchell B. Sc.

Introduction

The purpose of the following notes is to amplify certain aspects of the work on cast-iron bridges already referred to by Dr. Davey and to indicate recent trends in the method of approach to problems of impact, fatigue and permissible stresses in bridge girders.

Dynamic Stress Measurements

The Ministry of Transport Standard Loading for Highway Bridges includes an allowance for impact equivalent to a 50% increase in the nominal static load. A similar impact allowance has in the past been used in this country