Tests on a model composite bridge girder

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Tests on a Model Composite Bridge Girder

Essais sur une maquette de pont mixte

Modellversuche an Verbundträgern

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Introduction

The testing of the composite girder described in this paper was part of a programme of model studies and tests carried out in the design stages of the Tay Road Bridge project. The model studies were concerned with the effects of the bridge piers on the movement of river bed material, the evaluation of wind forces on the superstructure and the structural behaviour of the composite box girders.

The main test in the third part of these studies was carried out on a quarter scale model 45 feet span and is discussed in this paper.

Description of Project

The bridge is at present under construction and will provide a dual carriageway 1.4 miles long across the Firth of Tay in Scotland. An artist's impression of the bridge is shown in Fig. 1. Each span consists of a pair of welded steel box girders spaced 20 feet apart and braced together at the ends and 1/3rd points. On top of each girder is cast a reinforced concrete slab to carry a two-lane carriageway, connected to the box by stud welded shear connectors. The superstructure is carried on reinforced concrete columns which rise to a maximum of 104 feet above the water level.

The bridge has 42 spans, 31 of which are 180 feet long so that this span was chosen as the basis for model tests and is shown in Fig. 2. The girder is of composite construction, the connection of the slab to the steel box being made

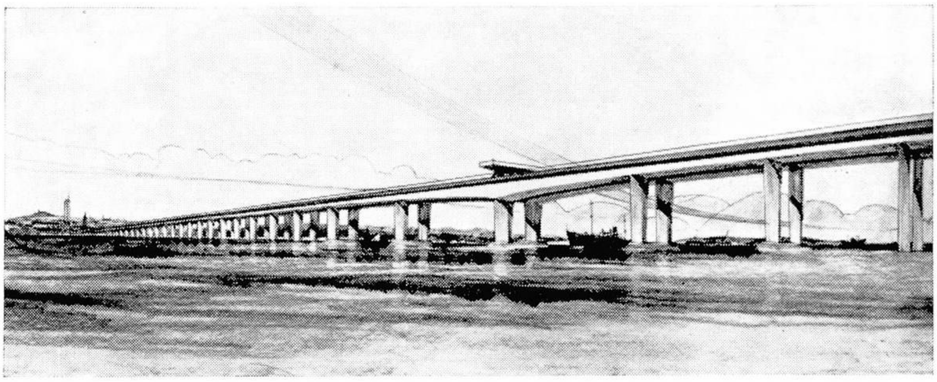

Fig. 1.

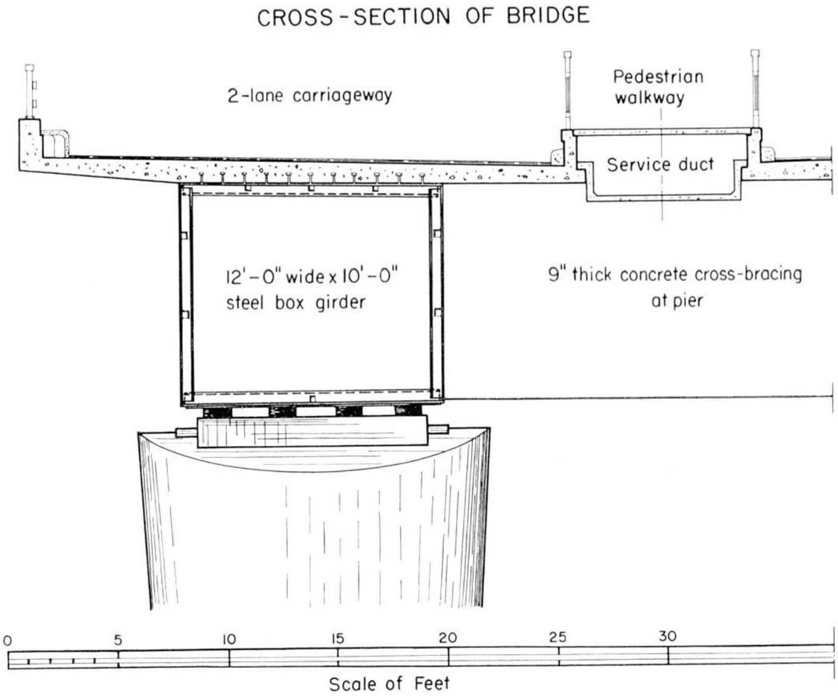

Fig. 2.

by 1300 stud welded $^3/_4$ in. dia. shear connectors. The tension flange is of high yield stress steel to B.S. 968 (1962) and all other parts are of normal structural mild steel. The steel end plates of each girder are set back from the extreme ends to allow the casting of a $10^1/_2$ inch thick reinforced concrete end wall in the space between the projecting ends of the webs and the lower flange. This detail is used to provide an economic form of stiffening over the points of support and at the same time to avoid the painting of closely spaced girder end plates. The cross bracings between the girders at the ends are 9 inch reinforced concrete walls, while those at the third points are 2 ft. 6 in. wide by 9 ft. deep steel box sections attached by high strength friction grip bolts. All stiffening of the box girder plating is internal using an orthogonal system of toe welded angles which are proportioned to the requirements of B.S. 153 Part 4 (1958). Before erection an 8 inch reinforced concrete slab is cast over the full width of the flange plate, this reduced composite section therefore carrying the full dead load.

Model Girder (Fig. 3)

The scale selected was 1 : 4 which produced the largest span (45 feet) which could be accommodated on the test bed. A smaller scale would have increased the difficulties of welding thin plates and casting thin concrete slabs.

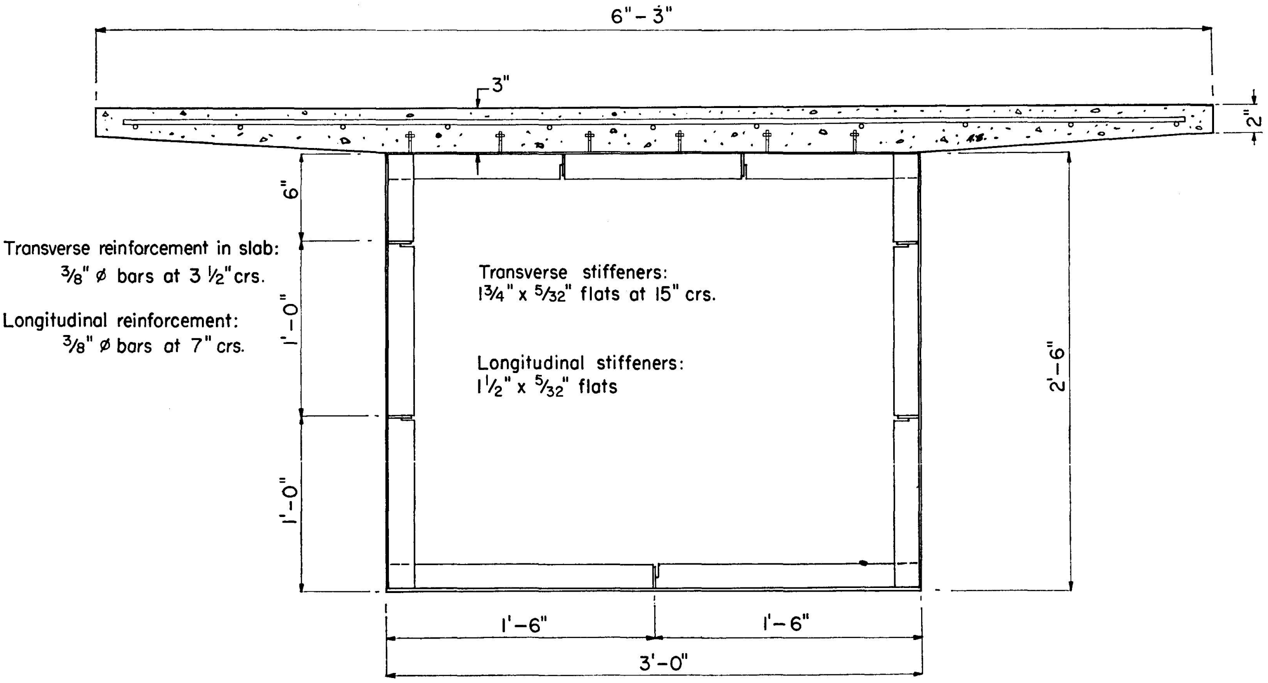

Fig. 3.

Since the correct thickness is not rolled, plates nominally 0.099 in. were used instead of the correct $^3/_{32}$ in. and all other thicknesses were taken from the nearest "Birmingham Sheet Gauge" sizes. There was considerable variation in plate thickness within each plate of the same order as the difference between the correct scale thickness and that actually used. To simulate angle stiffeners to scale flats were used having the same outstand and second moment of area about their junction with the web. The shear connection was by standard screwed studs $^3/_{16}$ in. dia. fitted with nuts after welding. The concrete deck slab and area of reinforcement were reduced accurately to scale with the exception that the edge kerbs were omitted. The maximum aggregate size used was $^3/_8$ in.

Difficulty was experienced in the welding of the thin plates into the box section and several trials were necessary before a satisfactory procedure was developed. Welds were inspected visually, but the studs were tested in pairs after welding to a stress of 9 T/in.^{2} in shear.

The first stage of the concreting of the full-size girder will be done with the girder supported at its third points so that the concrete will lock in a prestress induced by the dead weight of the steel box. This was accomplished in the model by anchoring it to the floor at the ends and applying upward jack loads at the $^1/_3$ points. The jack loads were adjusted to produce a maximum stress of 5 T/in.^{2} in compression in the bottom flange. The concrete was poured in alternate bays approx. 6 ft. long, but the end diaphragms and 2 ft. of the slab at each end were left unconcreted until the internal strain gauging was complete. Immediately after casting, the eight loading beams were bedded on the wet concrete in order to ensure perfect contact when under load.

The concrete mix was 1:2.2:3.3 with a water/cement ratio of 0.6 giving an average cube strength at 28 days of 5,350 lb./in.^{2}. To minimise shrinkage the top surface of the wet concrete was protected with a plastic curing membrane. In order to obtain a fairly uniform strength at the time of test the end diaphragms were of 1:1.8:2.7 mix with a water/cement ratio of 0.53 giving a 28 day cube strength of 6,350 lb./in.^{2}.

The girder was set on its bearings 32 days after the first concrete was cast, the strain gauges then showing a tensile stress in the lower flange of 1.3 T/in.^{2} under the dead load of the complete girder and the eight loading beams.

The strains during the various stages of the prestressing operation are shown in Fig. 4.

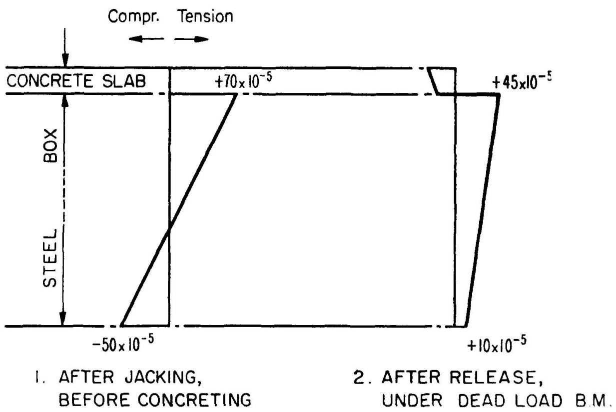

Fig. 4.

The moment of inertia of the mid-span cross-section for various values of the modular ratio and assuming complete interaction of the concrete and the steel box are as follows:

Modular Ratio m	Neutral Axis Depth (Measured from the top of the steel) d_n (in.)	Moment of Inertia of the equivalent steel section I (in.^{4})
6	5.20	7,730
8	6.47	7,260
10	7.52	6,872
12	8.35	6,560

The modular ratio for instantaneous loading of concrete with the cube strengths obtained would be expected to be less than 7.

Scaling

The model was constructed to a scale of 1 : 4 using the same materials as the prototype. Thus for the same stresses the bending moment scale is 1 : 64 and the shear force scale 1 : 16, i. e. the concentrated load scale is 1 : 16 and for distributed loads 1 : 4 for the same unit length.

The required model dead weight is then 35.5 Tons whereas the actual weight of the model was only 8 Tons (including 8 loading beams). However, since the model production sequence was not the same as for the full scale and there was no time to follow the long term effects of dead load this scale error was not corrected and allowance has to be made when interpreting the jack loads. For example, the eight point loading produced 91.2% of the bending moment due to the same total load uniformly distributed so that a jack load of 30.1 Tons corresponds to the correct scale dead load bending moment and for dead load shear the jack load is 27.5 Tons.

These differences from the prototype mean that care has to be exercised when interpreting jack loads at failure in terms of load factors.

Loading Arrangements

Fig. 5 shows the general arrangement of the loading system to represent U.D.L., the detail dimensions being dictated by the details of the test floor in the laboratory. A clear space of 10 feet was left in the centre of the span to accommodate the local concentrated load. It will be noticed that the eight load points are not equally loaded but the shear and moment diagrams shown in Fig. 6 are not significantly affected in shape. Load was supplied by two "Losenhausen" hydraulic jacks of the packless type driven from a hydraulic pump system which supplied both static and pulsating loads.

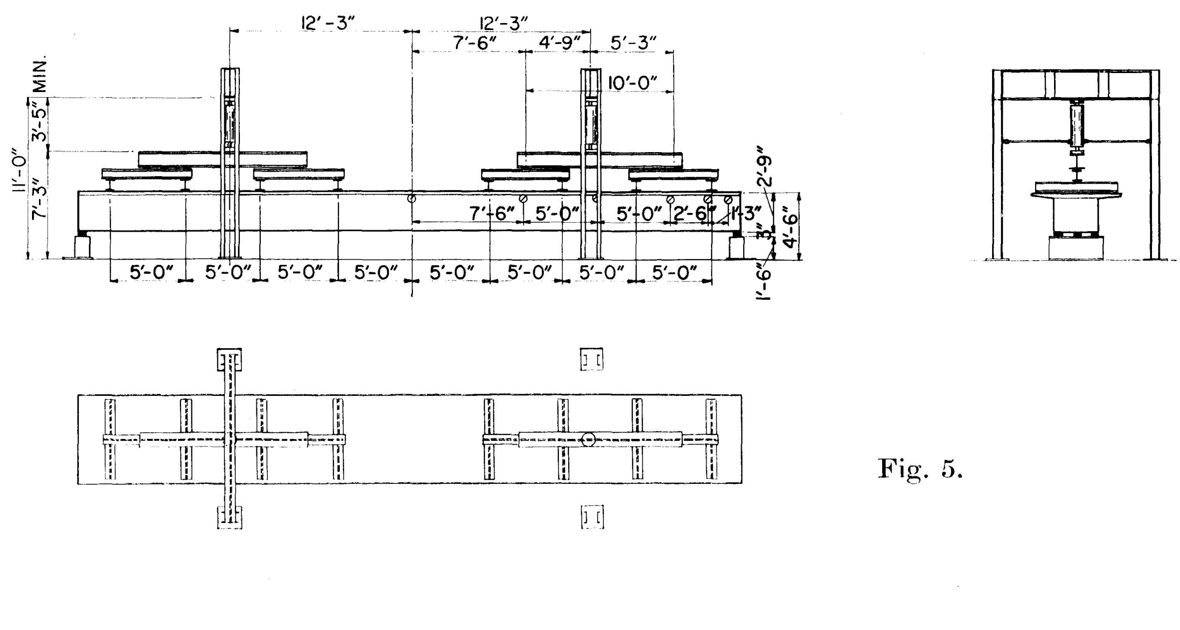

Fig. 5.

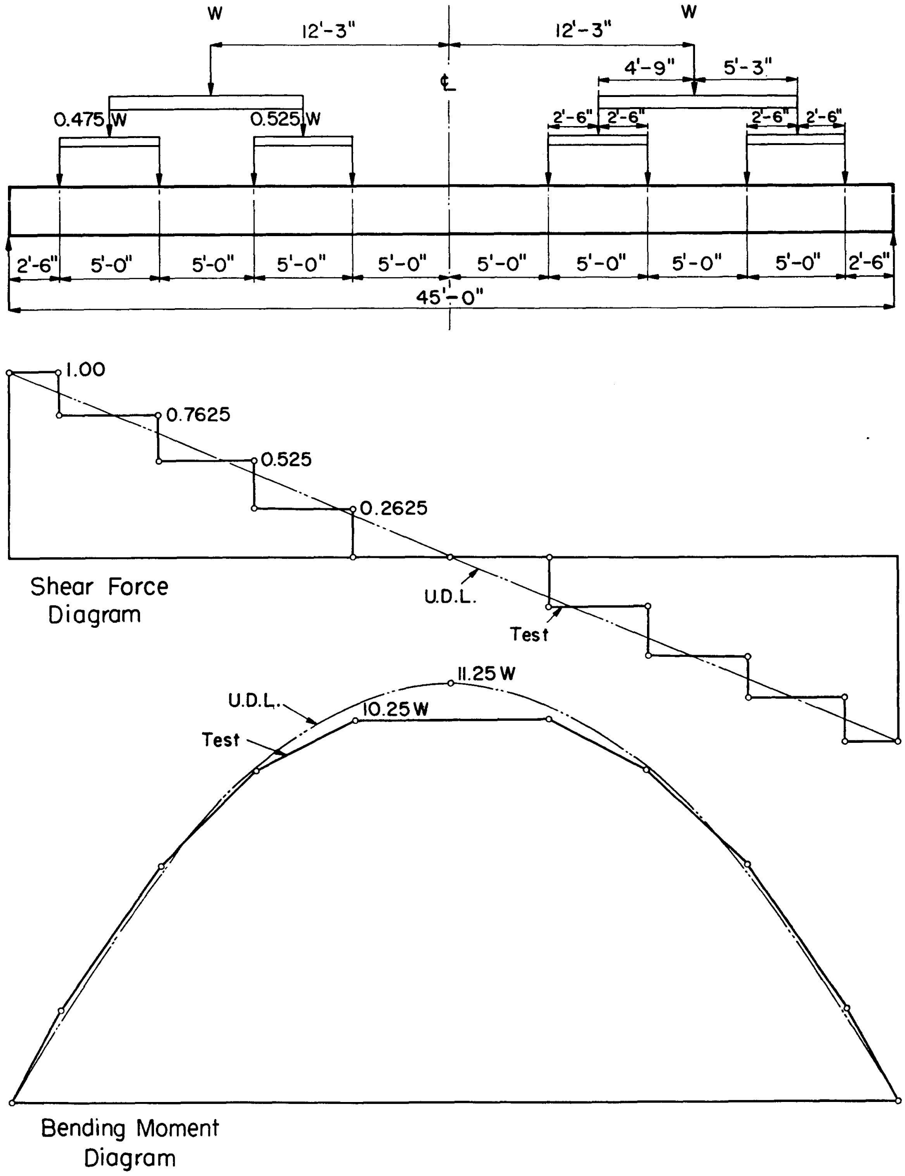

Fig. 6.

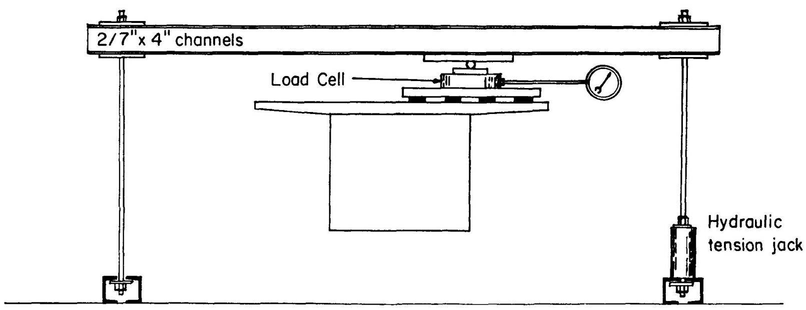

Fig. 7.

A general arrangement of the concentrated loading (H.B. type to B.S. 153) is shown in Fig. 7 the wheel contact areas being simulated by $^1/_2$ in. thick rubber pads cut to scale size and loaded through a 1 in. thick steel plate. For the "H.B." loadings the load was measured directly by a resistance gauge load cell.

Measurements

Vertical deflections and twist were measured by $^{1}/_{1000}$ in. dial gauges and by a surveyors' level. Strains were measured by foil type resistance gauges of 1 in. gauge length on the steel and 2 in. on the concrete. These gauges were attached to the steel by Eastman 910 adhesive and to the concrete by Araldite 951 and all gauges were finally protected by a layer of Araldite. The output from the strain gauges was recorded automatically at a rate of about 200 channels a minute on a typewriter and on punched tape.

The positions of the strain gauges at the centre cross-section discussed in this paper are shown in Fig. 8.

Relative movement, or slip, between the concrete deck and the steel box was checked by 26 $^{1}/_{1000}$ in., dial gauges positioned as in Fig. 5 and held in position as in Fig. 9.

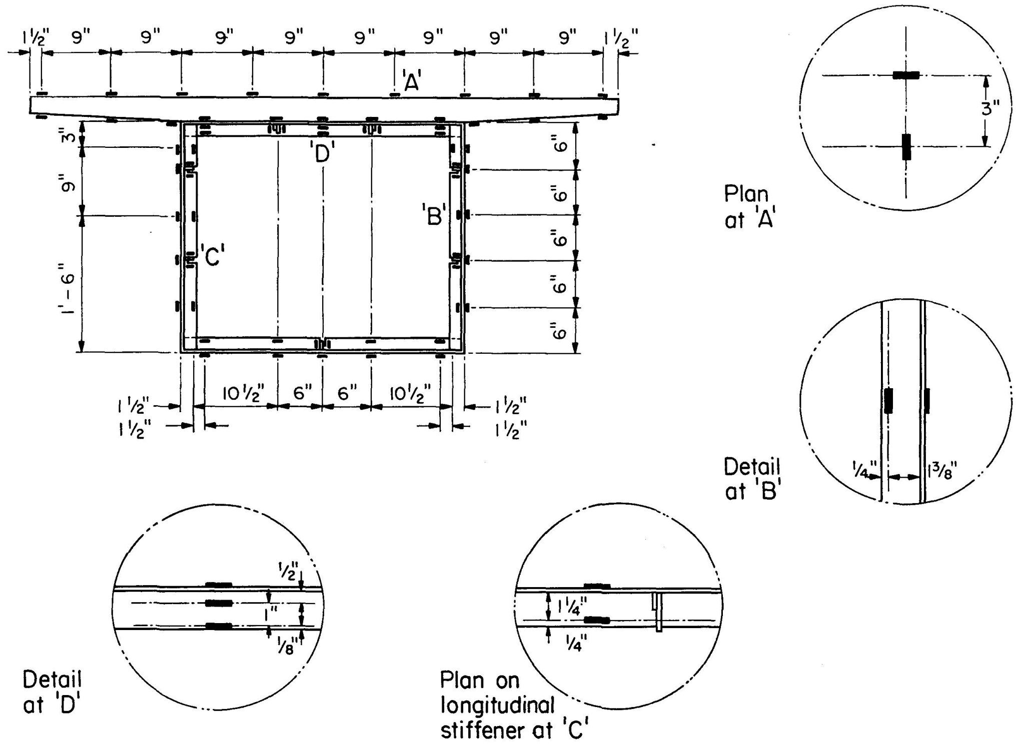

Fig. 8.

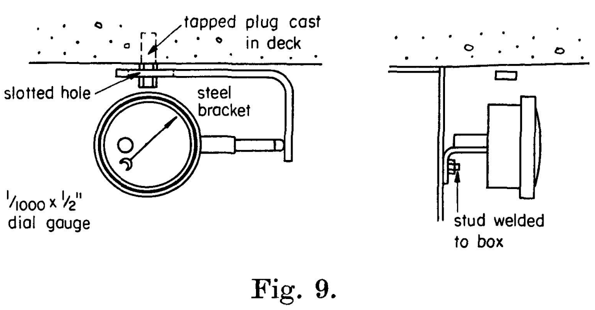

Fig. 9.

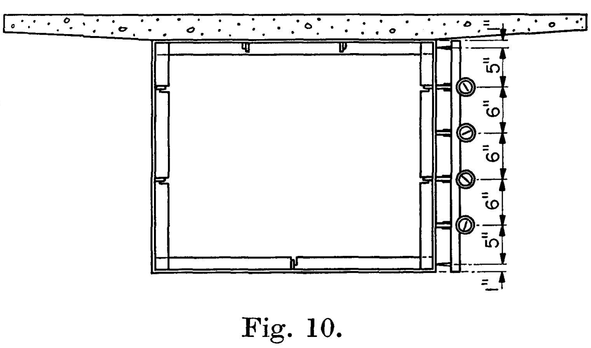

Fig. 10.

Deformations of the web were measured by dial gauges clamped to a stiff bar, the bar being located by three "feet" and held by hand in punch marks, Fig. 10.

Loading Sequence

After initial "settling in" runs up to 12 t the girder was loaded in increments to the scale dead load plus full live load (type H.A. to B.S. 153) of 48 t. During this stage no inelastic behaviour was observed but some of the "slip" gauges showed recoverable movements of $^1/_{1000}$ in. Web plate movements were insignificant.

A load corresponding to one bogie of the H.B. load was then applied first in the centre of the girder and then in the slow lane at mid span.

The next stage of the test was a sustained load of 48 t on the eight point system for 71 hours in an attempt to assess the creep rate.

Tests were then carried out to full live load (H.A. type) with the girder connected by diaphragms at the third points to flexible steel beams designed to simulate the support which will be obtained from the parallel unloaded girder. H.B. loads were also applied with the diaphragms attached.

All the diaphragms were then disconnected and pulsating loads of from 30 t to 42 t, i.e. approximately equivalent to the range from dead load to full live load, were applied. This loading was carried out at 50 cycles per minute for 2000 cycles to simulate the number of such loads which might be expected in 700 years. No permanent deformations were observed.

After these tests the girder was loaded to failure. This took place by a weld failure in the tension flange at a load of 102 t. The rest of the girder was undamaged and a successful repair was carried out allowing a second failure run to be made when failure occurred by web buckling over one support at a load of 117 t. The girder was again repaired and tested to obtain a different failure mode.

Bending Behaviour

a) Deflections

Deflections of the centre of the span during the loading to 117 t are shown in Fig. 11. There is no significant twist and the behaviour is linear up to 90 t total jack load indicating an effective moment of inertia of about 7,050 in.^{4} (steel units). It is shown below that all stresses are well below the yield stress of the steel at this stage and as the slip gauges show negligible loss of bond this departure from linearity is probably due to the effects of web deformations. The web finally failed at the corner which would show a larger movement of deflection gauge 4 as in Fig. 11.

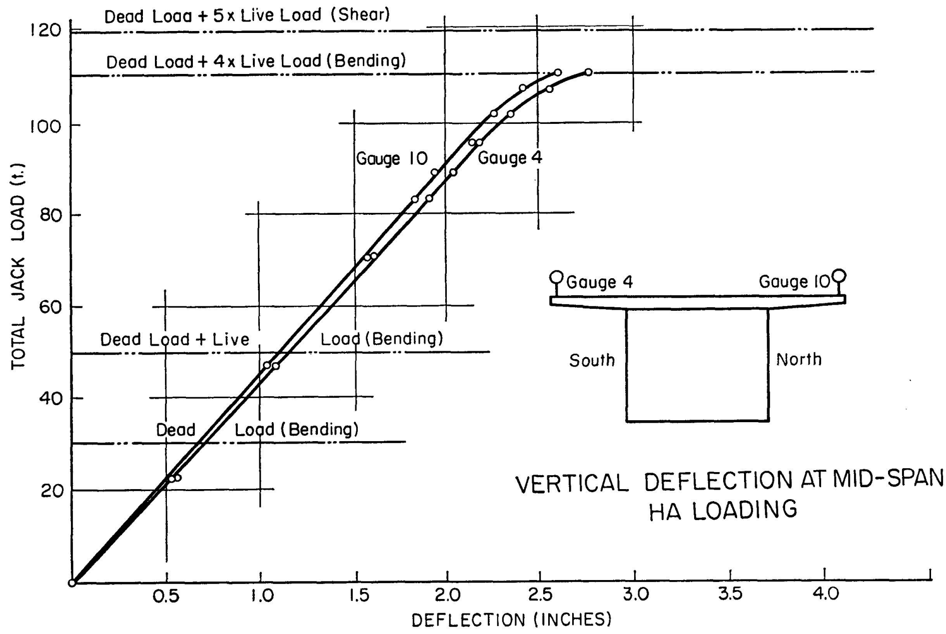

Fig. 11.

Calculation of the probable deflection due to the shear stresses in the web shows that shear accounts for about 8% of the total deflection and this has been allowed for in the above calculation of I.

Deflections with the concentrated "H.B." load at mid span give an effective "I" value of 7,700 in.^{4}.

b) *Pulsating Loads*

Under the action of the pulsating loads of 30 to 42 t (i.e. 36 ± 6 t) there were no residual deflections and the "slip" gauges indicated only elastic movements of about $\frac{1}{2} \cdot 10^{-3}$ in. at the ends. This amount of elastic relative movement is the same as for the static load cases.

c) *Creep Loading*

The creep of the central deflection during a 71 hour period is shown in Fig. 12, the loading being approximately the full dead plus maximum live load (i.e. 48 t instead of 50 t). Though the curve is not as regular as could be wished there is a marked change in the deflection rate after 5 hours and after 70 hours the deflection is still increasing slowly. At 70 hours a creep deflection reduces the apparent "I" value to 6,200 in.^{4} from 7,050 in.^{4} for the short time loading.

d) *Stresses*

The strain distribution at the central cross-section, Fig. 13, is well defined. From these strains and the measured neutral axis depth and assuming complete interaction between concrete and steel I is calculated to be 7,330 in.^{4}

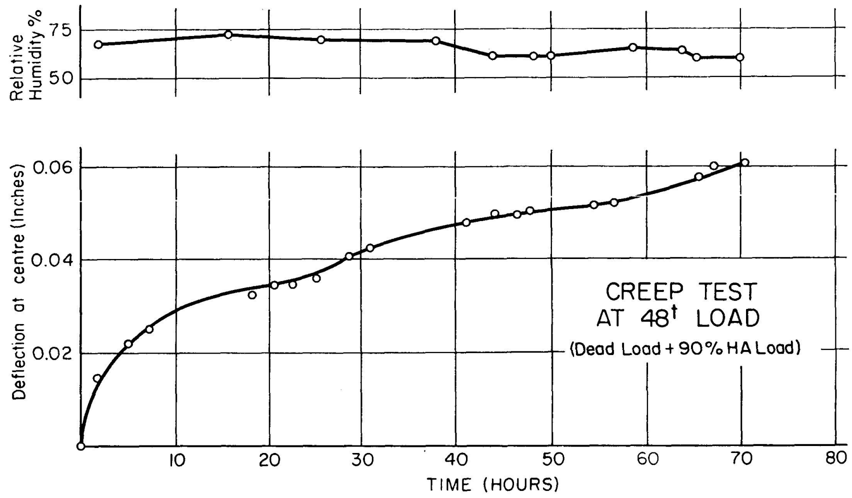

Fig. 12.

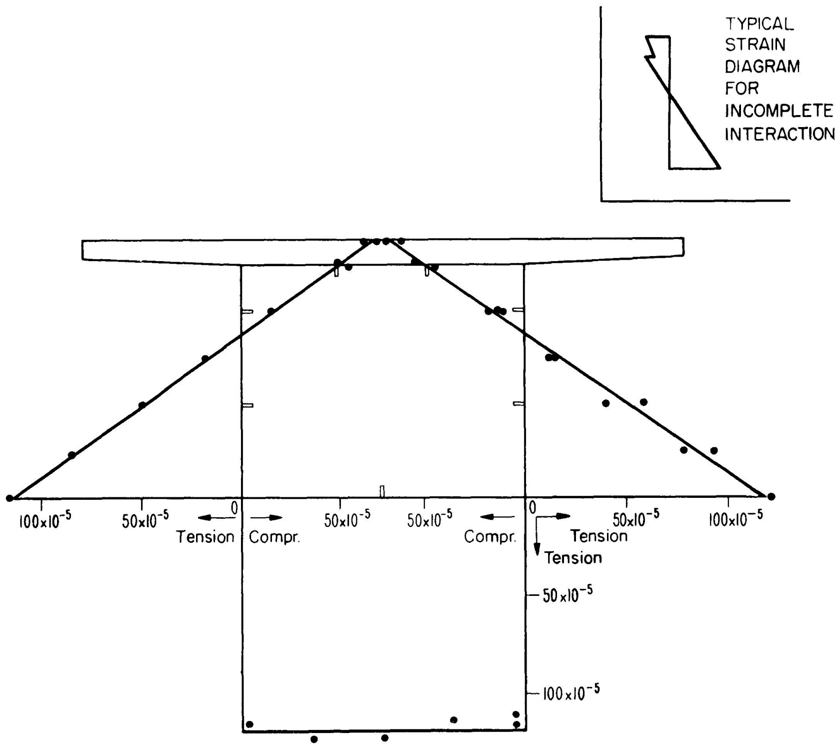

Fig. 13.

showing a modular ratio of 8. However, this value of the modular ratio gives a neutral axis depth of 6.5 in. whereas the measured depth is 9 in. below the top of the steel. Hence it is apparent that full interaction is not realised although the relative movement of slab and box was not measurable with the $^{1}/_{1000}$ in. slip gauges.

The observed neutral axis depth with $I = 7{,}330\,\text{in.}^4$ corresponds to a reduction of about 10% in the axial compression in the deck slab and the strain distribution must have been of the form shown in the inset in Fig. 13.

The final strain readings at a jack load of 117 t indicate a maximum stress of 21.1 t/in.^{2} to which must be added the initial stress of 1.3 t/in.^{2} giving a maximum stress in the H.T. steel flange of 22.6 t/in.^{2}. This is below the yield stress of this steel but the strain is in the yield range for the mild steel webs. There was no evidence of any yielding in the webs even after the final collapse.

There is one inconsistency in the results the source of which cannot be detected and whichever order is used to compare results the numerical values already quoted are not significantly affected though the inconsistency will show up in different ways. Following the present sequence of discussion the bending moment calculated from the measured strains and neutral axis position should check that obtained from the jack loads. The strains give a value 91% of that calculated from the loads.

Repeat Test to Failure

After the model had failed in the second main test by web buckling the first 8 feet of the girder was cut off and a new steel bulkhead welded into the open end, Fig. 14. In order to prevent a second failure by buckling diagonal angle stiffeners were welded to the outside of all four end panels.

The model was set up again but loaded with two equal point loads applied to the slab through 12 in. × 12 in. B.F.Bs. across the full width of the deck slab. Due to the test floor anchorages this loading could not be symmetrical. Deflection and slip gauge readings for this repeat loading are shown in Figs. 15 and 16.

No slip was recorded until the total load exceeded 48 t and final failure took place at 78 t by shearing of all the studs between the load point and the

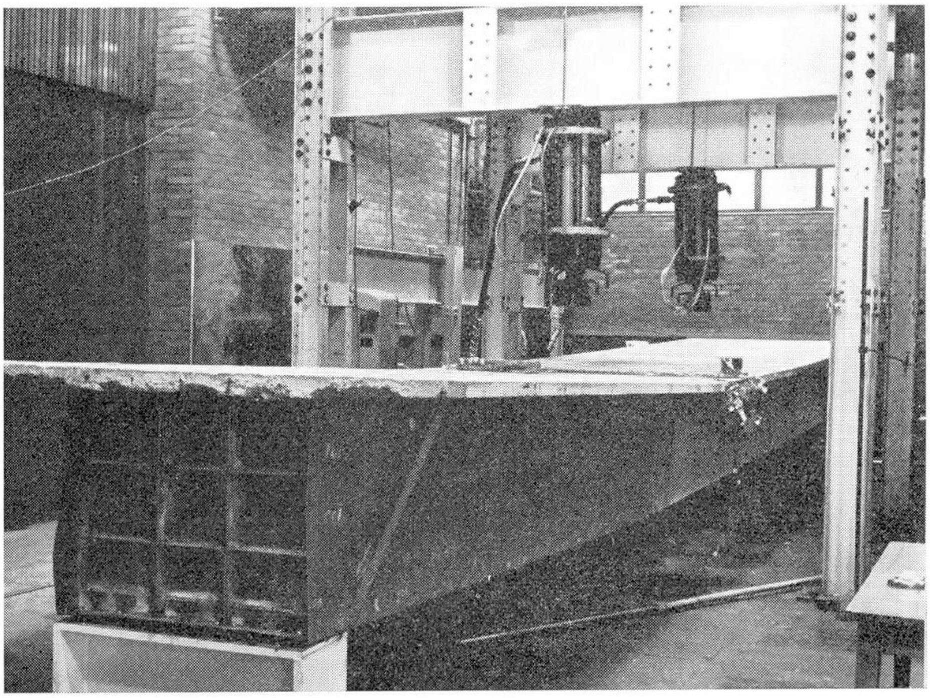

Fig. 14.

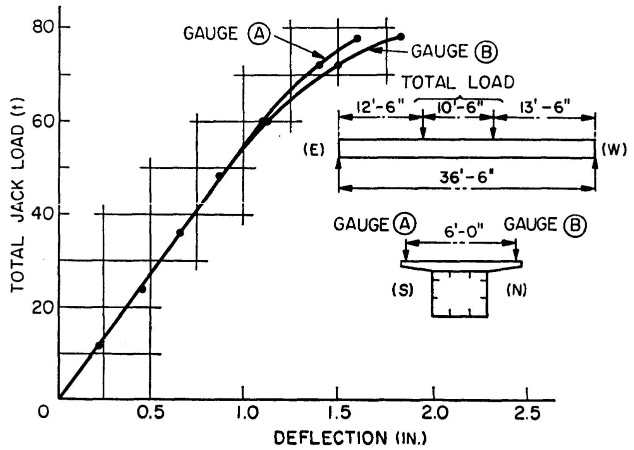

Fig. 15.

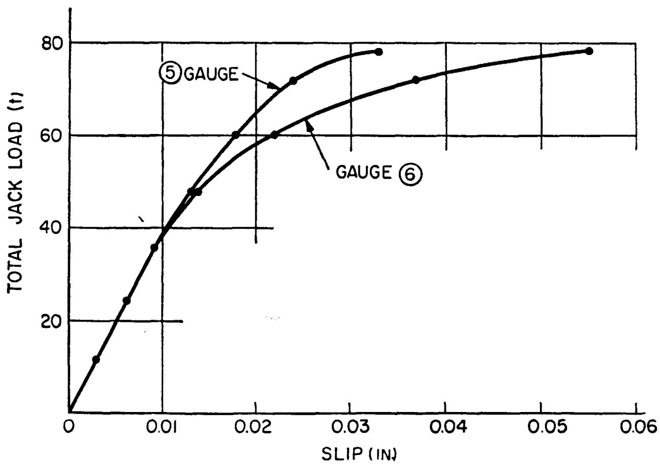

Fig. 16.

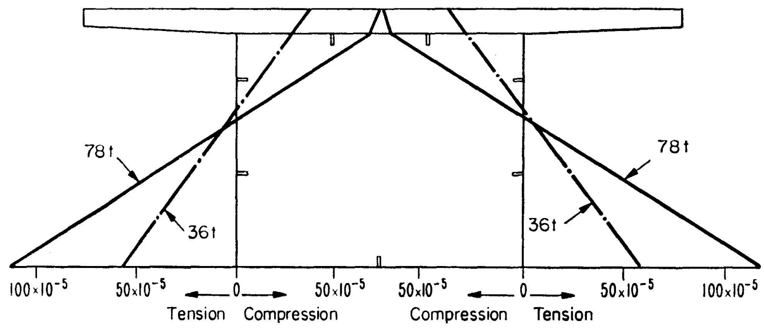

Fig. 17.

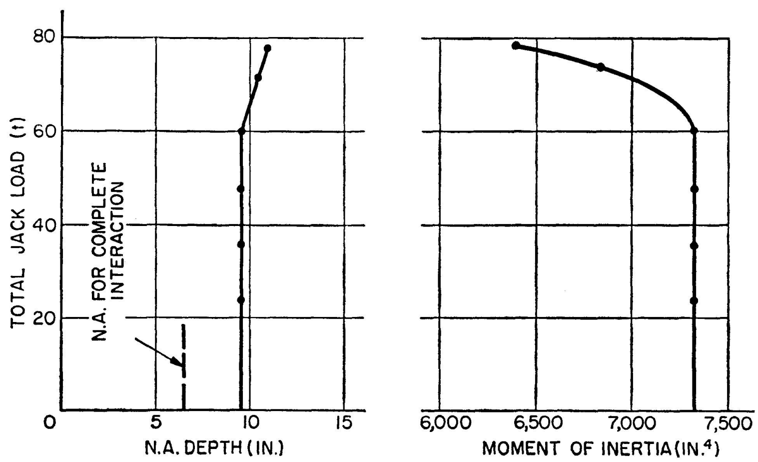

Fig. 18.

nearer support. At failure the strain gauges on the bottom flange showed no definite departure from linearity and indicated a stress of 18 t/in.². The total force transmitted through the studs at failure was 110 t giving a shear stress on the 220 studs of 18.2 t/in.² The shear strength of the studs when tested independently was 20 t/in.².

Strain measurements at loads below 60 t showed that the neutral axis depth was 9.5 in. from the top of the steel box, which corresponds to about 90% interaction. At loads above 60 t the neutral axis depth increased until just before failure at 78 t it reached 10.8 in. indicating a further reduction in the interaction. The values of I calculated from the measured strains altered from 7330 in.⁴ at 60 t to 6400 in.⁴ at 78 t and from deflection readings was 6550 in.⁴. Fig. 17 shows the strain distributions at 36 t and at 78 t and Fig. 18 shows the variation of neutral axis depth and I value with the load.

Web Buckling

A design criterion based on the development of a tension field in the web is not yet acceptable in girder bridges since there is an aesthetic limit to distortion. The design of the web stiffeners was based on BLEICH's [1] recommendations and conforms to the requirements of B.S. 153 (1958) for intermediate stiffeners.

A survey of the plate thicknesses before fabrication showed reductions of thickness at the edges of from 11% to 8%. While the thickness at the middle of the plate could not be measured variations of this order could represent a reduction of around 45% in the buckling strength of a web panel.

A preliminary survey of the web profile is shown in Fig. 19 which indicates

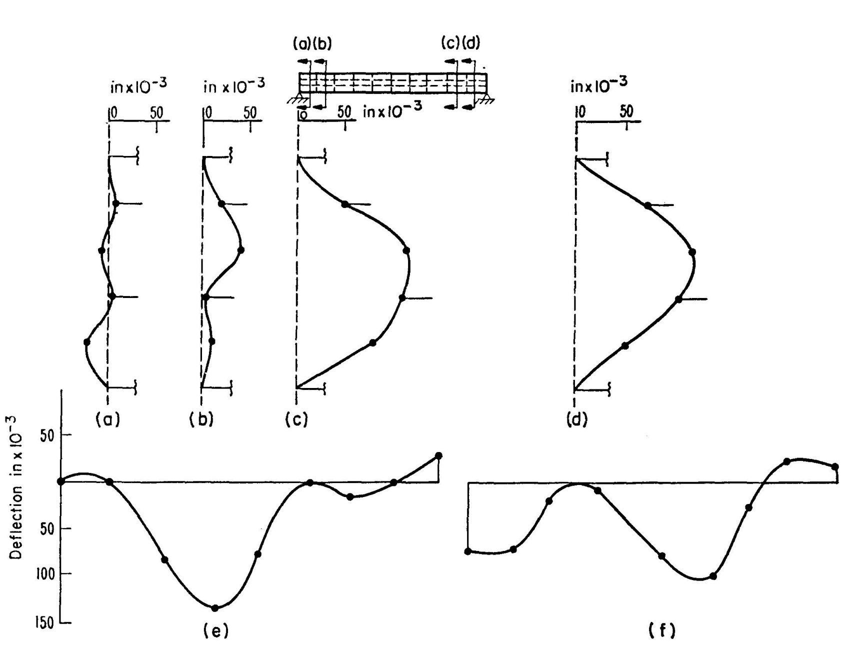

Fig. 19.

in (a), (b), (c) and (d) the vertical profiles near the centre of the end two panels at opposite ends of the girder. Fig. 19 (e) and (f) give the same data for horizontal profiles near the centre of the web depth. There was no convenient way of establishing a base for these horizontal profiles to show any initial bend of the stiffener but in the vertical direction at the east end, Figs. 19 (c) and (d), the marked bulge of the complete web and stiffener assembly is obvious. Failure occurred at this end preceded by large overall bulging as well as deflections of the web plate within the stiffener panels.

The plate bulges are of the order of 0.60 to 1.35 times the plate thickness. Additional deflections of the web were recorded during loading, Fig. 20, but

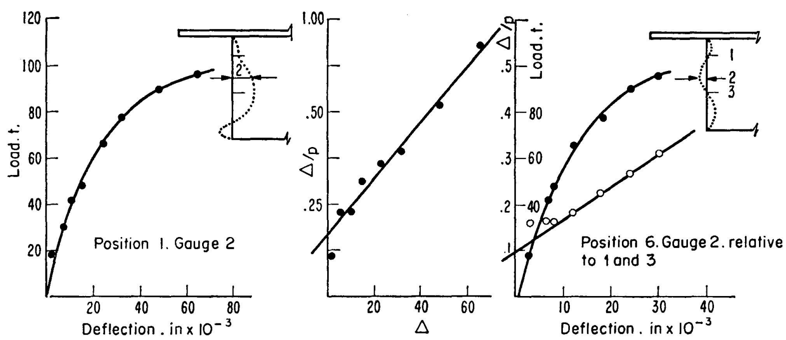

Fig. 20.

are so small compared to the initial values that SOUTHWELL's [2] method for estimating the buckling load was not reliable. However the results selected and shown in Fig. 20 do indicate a critical load to be of the order of 140 t. It is to be noted that the curves plotted for point 2, in Fig. 20 include the overall deflection of the stiffener assembly. This compares with the theoretical value for a simply supported panel buckling in shear of 167 t.

The critical features of the test are clearly shown by the movements of the gauges in Fig. 21 which show the increase in overall bulging and of deflection at gauge 4 up to a load of 96 t.

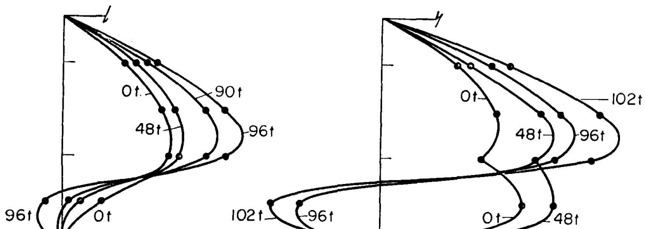

Fig. 21.

After the weld failure at 102 t there was a large permanent buckle at gauge 4 which was hammered in after the girder had been repaired. The plate profile under the repeat loading shows the return of this bulge after 72 t. At 90 t the buckles in this panel were obvious to the eye and final collapse occurred at a load of 117 t.

The inside of the panel after collapse is shown in Fig. 22 and clearly shows

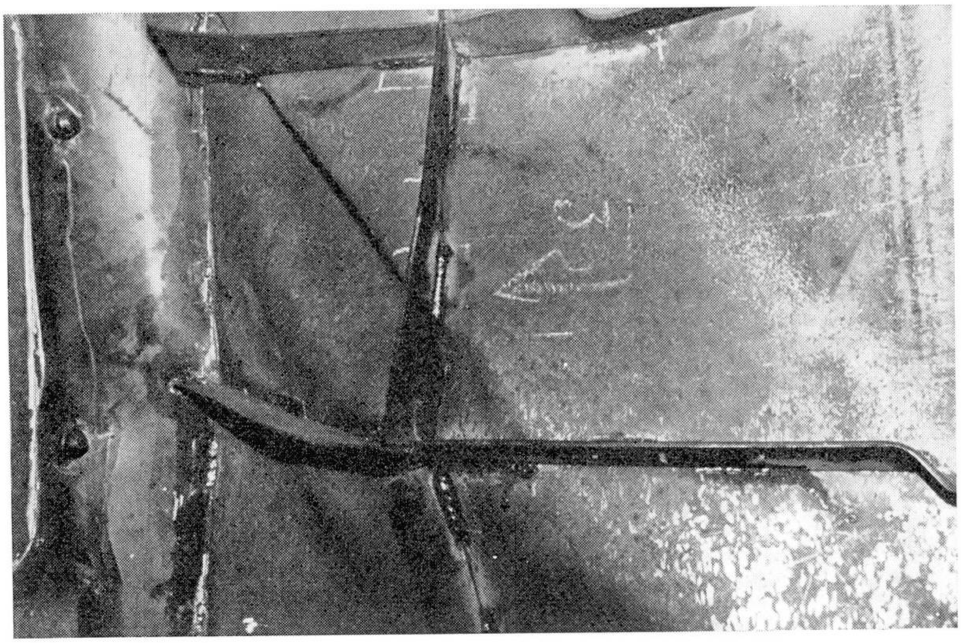

Fig. 22.

the overall bending and the tear of the joint in the vertical stiffener. The ultimate strength of the model girder depended entirely on the presence and magnification of this large initial bend of the complete web-stiffener assembly at one corner.

There are additional factors to be considered before this result can safely be applied to the full size girder viz:

1. The thickness tolerances and the flatness of the full size plates are much better than the model values.
2. Welding distortions of the full size girder are more easily controlled and the correct girder cross-section shape can be more accurately maintained.
3. Better initial flatness of the plating will give an increased failure load in itself but since the main feature appears to be the overall deformation of the web the indication is that as long as the web stiffeners are straight the local plate irregularities within the stiffened panels are relatively unimportant.

Torsional Effects

Torsion of the box section will be caused by off-centre loading and by differential movements of the parallel girders.

Tests were made on the off-centre load effects by simulating one bogie of the H.B. loading as shown already in Fig. 7. This loading was applied on the "slow" lane both at the centre of the span and at the third point. Differential movement of the two girders was simulated by providing elastic supports at the connecting diaphragms, Fig. 23 and applying the usual central loading to the girder.

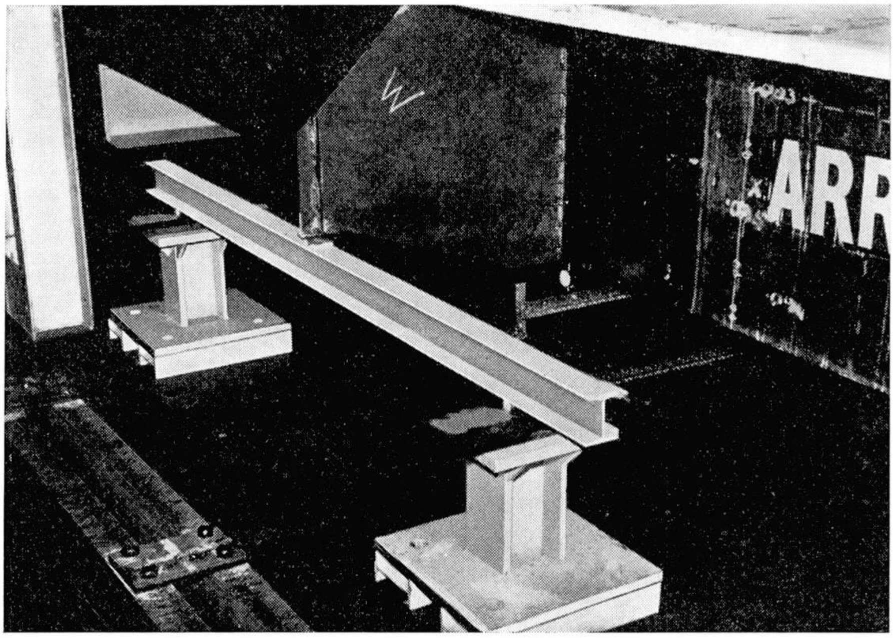

Fig. 23.

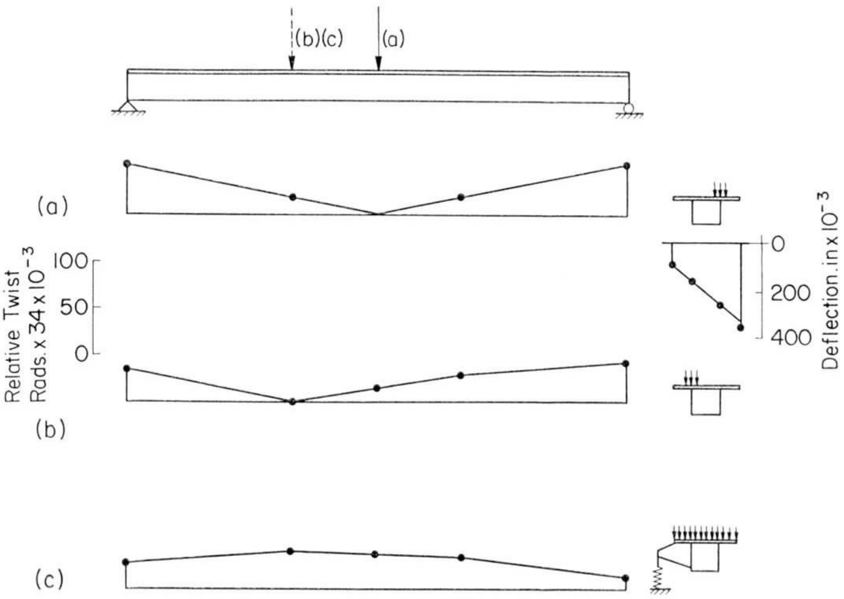

Fig. 24.

The differential deflection between the edges of the box section is a measure of the torsional twist and Fig. 24 shows the values from which the effective torsional stiffness is calculated as follows

Fig. 24 (a) and (b) off-centre loads: $J = 1600$ in.^{4};
Fig. 24 (c) differential movt.: $J = 5700$ in.^{4}, calculation $(m = 10)$: $J = 5220$ in.^{4}.

It is difficult to account analytically for the very low value obtained from the off-centre load case. Strain measurements in Fig. 25 show clearly a sway or racking near the concentrated load which would be interpreted as an appreciable reduction in the effective torsion stiffness.

Although an analytical estimate of this effect has not been possible it does not seem accurate enough to replace the real load in the orthodox manner with a central load and the equivalent torsional couple at the longitudinal axis of the girder.

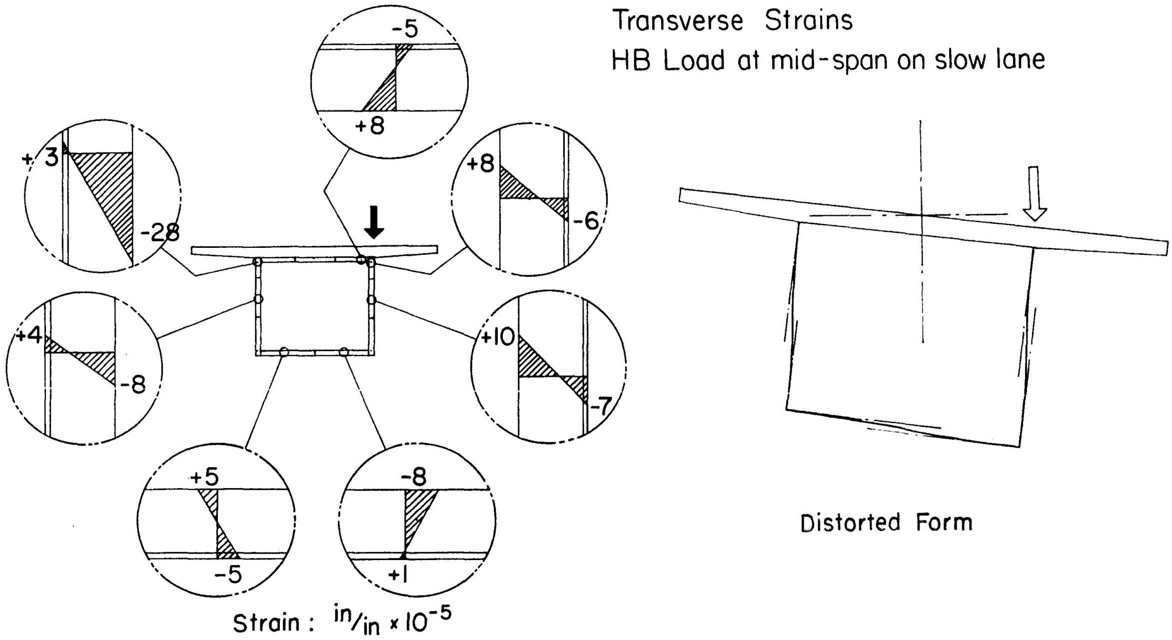

Fig. 25.

Torsion due to the differential movement of the girders does not produce this racking since the torque is applied by the connecting diaphragms in a manner more closely approximating to the orthodox torsional stress distribution on the cross section. While the technical significance of this experimental observation is obvious and the Authors are not aware of this aspect having been previously investigated, it is not important in the present case. The actual twist under two units of this abnormal load i.e., twice the test load, is only about $3 \cdot 10^{-3}$ radians (about 10 min.). It does affect the forces on the connections of the diaphragms but the alternative load condition of full distributed load on one girder is in fact much more severe.

Knowing the experimental flexural and torsional stiffnesses it is a simple matter to calculate the load transferred between the parallel girders but for interest's sake this was checked by simulating the effect of the second girder as already shown in Fig. 23. Due to the short time available these elastic supports were installed before the correct flexibilities of the girder had been determined and they were in fact too flexible. The actual flexibility of the girder as a support is $29 \cdot 10^{-6}$ in. per lb. but that of the test supports was

$38 \cdot 10^{-6}$ in. per lb. plus $9 \cdot 10^{-6}$ in. per lb. due to slip in the diaphragm connection i. e. a total of $47 \cdot 10^{-6}$ in. per lb.

This results in a calculated reduction of the load transferred when the loaded girder is carrying 21 t from 3.45 t per support to 1.85 t per support and agrees with the measured value of 1.83 t.

Discussion

In design and construction, speed is essential, so that an Engineer's tests must be completed in a very short and well defined time. The advancement of knowledge requires a lengthy period and cannot be done to a timetable. So that the two aspects of such tests are, from the beginning, not compatible.

Large tests are expensive and it is this cost factor which usually ties the two together.

The tests described in this paper fulfil their engineering function in that, although no new or unpredictable factors of significance emerged the Engineer has seen the complete life of the structure and has direct evidence of its safety and of the fitness of his calculations.

The failure load of 117 t in the first main test is equivalent on the prototype to the shear due to full dead load plus 4.96 times H. A. load and to the bending moment due to full dead load plus 4.37 times H. A. load.

The three most important general points emerging from this study are as follows:

a) The incomplete interaction between the concrete slab and the steel box.
b) The reduction of the torsional stiffness due to the concentrated load.
c) Overall bulging of the complete web-stiffener assembly overriding the buckling of the individual web plate panels.

In this respect the test does represent a necessary extension of laboratory research work.

The testing was done in a University teaching laboratory by a team of seven drawn from the departments of Civil Engineering, Mechanics and Mechanisms, Naval Architecture and Aeronautics and from the Consulting Engineer's staff, all of whom worked on this test in addition to their normal duties. A suggestion is sometimes made that such work is better done by a full time staff engaged permanently on such work. The Authors have given careful attention to this point of view and are firmly convinced that such centralisation and specialisation is not in the best interests of Civil Engineering or of the general wellbeing of structures research. They do, of course, realise that it is necessary to improve the efficiency of this type of work and that this can be achieved by a mutual understanding of the needs of both the Engineer and the Research Worker.

Acknowledgements

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Abbreviations

dia.	diameter
ft.	foot, feet
in.	inch, inches
lb.	pound, pounds
T	Ton (2,240 lb.)
t	tonne (1,000 kg)

References

1. Bleich: "Buckling Strength of Metal Structures". McGraw-Hill, 1952.
2. Southwell: "Theory of Elasticity". Oxford, 1941.

Summary

The paper reports on a series of tests carried out on a 1 to 4 scale model of one span of a composite box girder bridge. The girder consisted of a steel box stiffened orthogonally on the inside, with a concrete deck slab connected to it by stud welded shear connectors.

Analysis of the readings led to the following general conclusions:

1. During the elastic stage the share of the load carried by the concrete was approx. 90% of that calculated for full interaction.
2. Concentrated loads applied eccentrically over one of the webs produced sway distortion equivalent to a reduction of 70% in the torsional rigidity.
3. Buckling strength of the web-stiffener assembly overrides buckling of individual web-plate panels.

Résumé

Le présent mémoire contient les résultats d'essais effectués sur une maquette (échelle 1 à 4) d'une travée simple d'un pont mixte acier-béton. L'ouvrage est constitué d'une dalle de couverture en béton, liée par des goujons

de cisaillement soudés à une poutre-caisson métallique, raidie orthogonalement à l'intérieur.

L'interprétation des résultats a conduit aux conclusions suivantes:

1. Dans le domaine élastique, le béton reprend environ 90% de la valeur correspondant à une liaison acier-béton parfaite.
2. Des charges concentrées excentriques, agissant dans le plan d'une des deux âmes, provoquent une déformation du contour de la section transversale, ce qui correspond à une réduction de 70% de la rigidité torsionnelle.
3. La résistance au voilement de l'ensemble tôle-raidisseurs était inférieure à celle des panneaux élémentaires.

Zusammenfassung

Der Beitrag befaßt sich mit einer Versuchsreihe, die an einem Modell im Maßstab 1 : 4 eines in Verbundkonstruktion hergestellten Kastenträgers durchgeführt wurde. Der Verbundkasten war gebildet aus Stahlblechwänden, die auf der Innenseite orthogonal ausgesteift waren, und aus einer Deckplatte aus Eisenbeton, die mit der Stahlkonstruktion durch aufgeschweißte Scherbolzen verbunden war.

Die Auswertung der Versuchsdaten führte zu folgenden allgemeinen Ergebnissen:

1. Im elastischen Bereich war der Belastungsanteil, der vom Beton aufgenommen wurde, etwa 90% des Wertes, der für volles Zusammenwirken berechnet worden war.
2. Konzentrierte, exzentrische Belastung über einem der Stehbleche ergab eine Verformung, die einer 70%igen Verringerung der Verdrehfestigkeit entsprach.
3. Die Beulfestigkeit des ausgesteiften Stehbleches war kleiner als diejenige der einzelnen Teilfelder.