Bending moments are calculated in accordance with Fig. 7, p. 608 (cross-girders not staggered).

A sample calculation for Loading No. 1 is given in diagram form in Fig. 22.

Fig. 16, *Plate* 2: *Stress diagrams: live load tests*

Diagram A. Initial stresses due to prestressing. The prestressing force is provided by nine 1-in.-dia. steel bars, which are initially stressed to 42 tons/sq. in. The force in each bar is therefore $42 \times 2{,}240 \times 0{\cdot}7854 = 73{,}900$ lb.

For the stresses at mid-span, the axes of the nine bars are $1\frac{3}{4}$ in. below the neutral axis of the slab. The total hogging moment is therefore

$$9 \times 73{,}900 \times 1{\cdot}75 = 1{,}164{,}000 \text{ lb.-in.}$$

Surface stresses due to prestressing moment $= \pm \dfrac{1{,}164{,}000}{1{,}200} = \pm$ 970 lb/sq. in.

Direct average compressive stress due to prestressing force $= - \dfrac{9 \times 73{,}900}{72 \times 10} = -$ 924 lb/sq. in.

Hence compressive stress at bottom surface $= -$ 1,894 lb/sq. in.
tensile stress at top surface $= +$ 46 lb/sq. in.

Diagram B. Stresses prior to application of live load. For the stresses at mid-span :—

		at bottom surface of slab : lb/sq. in.	at top surface of slab : lb/sq. in.
Residual stresses due to prestressing (assumed to be 16 per cent below initial stresses)	=	− 1,590	+ 39
Stresses due to self-weight of slab (on span equal to length of slab, 116 in.) $= \dfrac{6{,}750 \text{ lb.} \times 116}{8 \times 1{,}200}$	=	+ 82	− 82
Stresses due to weight of permanent way and ballast (load taken as evenly distributed over floor-units considered as continuous beam on three supports) $= \pm \dfrac{5{,}400 \text{ lb.} \times 141}{16 \times 1{,}200}$	=	+ 40	− 40
Net total stresses		− 1,468	− 83

Discussion

The Authors, introducing the Paper, contrasted the recorded lateral oscillations of the top flanges of the outer main girders of three double-track spans, similar in length but having different conditions of fixity between the decking and the main girders (see Fig. 23). In the bridge near Henwick where the shear-plate connexion shown in Figs 18 and 19 had been adopted, the lateral oscillation was less than one-eighth that recorded at the Athelney bridge, where the troughing rested eccentrically on the bottom flange angles and the riveted connexions were too weak to develop the end-fixing moment. In the bridge near Harrow-on-the-Hill, where the cross-girders rested on shear plates on the flanges of the stiffeners on the main girders, and the bending stresses due to the end fixing moment were carried by high-strength bolts (Fig. 12), the lateral oscillation was also small.

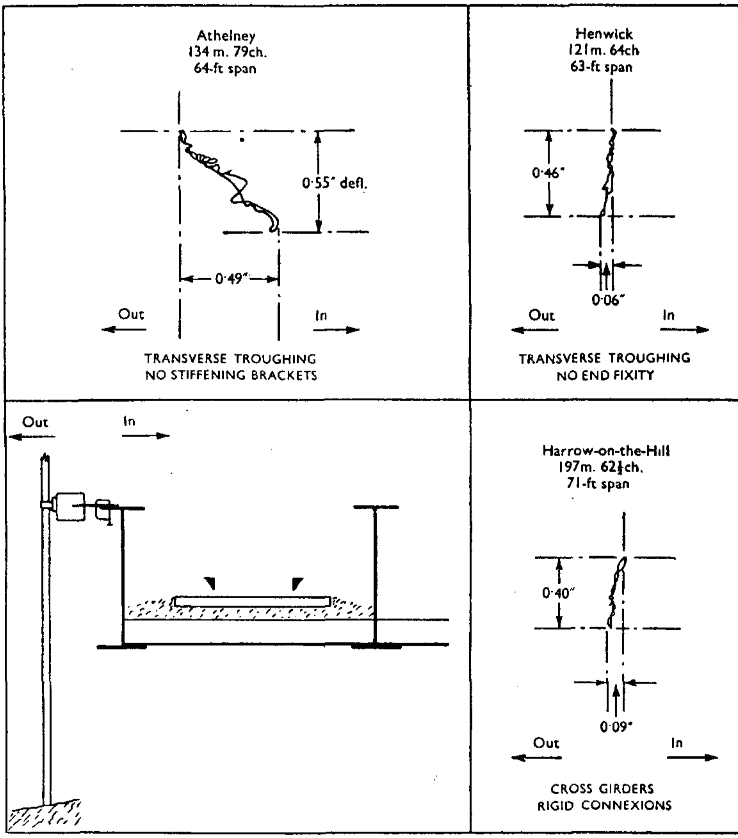

FIG. 23.—RECORDED LATERAL OSCILLATIONS OF MAIN GIRDERS

Referring to the use of high-strength bolts, they observed that the application of the technique to railway structural engineering was first being tried out in the United States in 1948, since when, extensive tests both in the laboratory and in the field had shown that high-strength bolts were as reliable as rivets and in some ways were better than rivets. The structural use of high-strength bolts in America had developed rapidly and an informative report on their use in steel railway bridges had recently been published by the American Railway Engineering Association.[7] The Western Region of British Railways had, since 1951, used high-tensile bolts and mild-steel nuts to British Standard specifications, otherwise following American practice more or less closely.

The torque values given in Table 1 had been calculated from the formula $Q = KTd$, where Q denoted the torque, T the tension in the bolt at 85 per cent of the yield stress, d the bolt diameter, and K was a non-dimensional coefficient whose value depended on the friction of the screw threads and of the nut face. The value 0·2 had been chosen for K as representing a fair average for bolts of British manufacture, and that was also the value used in American practice; but for generated threads (i.e. threads produced by rolling between cylindrical dies or threads rolled between flat dies to give an equivalent and consistent surface finish), a series of tests undertaken by the Research Department of British Railways had shown a remarkably consistent value of about 0·16 for the torque coefficient. That meant that, for such bolts, the torque values shown in Table 1 might be reduced by 20%.

Mr G. B. Godfrey (British Iron and Steel Federation) stated that he sought further information on high-strength bolting.

From the remarks made in their introduction and from the Paper itself the Authors seemed to have used only manual torque wrenches to tighten high-strength bolts. Why had they not used power wrenches driven by compressed-air or electricity? Such tools, although weighing about 30 lb. each, required less physical effort by the operatives. For casual or limited use, they were probably more expensive than manual wrenches, but time would be saved, an advantage of great importance in railway work.

Mr Godfrey showed some lantern slides in which the cross-beams and girders of an American bridge had been joined with high-strength bolts. He demonstrated that, in order to prevent slip, it was the American practice not to paint the contact surfaces or the areas beneath the washers. In the examples illustrated in the Paper, the Authors had provided shear plates which prevented slip. Had they painted the contact surfaces?

Fig. 24 showed the American Standard high-strength bolt. It would be observed that the threads were coarse but that a deep nut was used. He believed that the Authors' specification required bolts with an ultimate tensile strength of 45 tons/sq. in. and nuts with an ultimate tensile strength of 28 tons/sq. in., the figures being comparable with American practice. However, British nuts were shallower than American. Did the Authors use fine threads or had there ever been any stripping during tightening operations?

Mr Godfrey also showed some lantern slides of a truss bridge in Germany, the first in which high-strength bolts were used for site

[7] Report of Committee 15 on "Use of high-strength structural bolts in steel railway bridges." A.R.E.A., vol. 56, No. 520, pp. 592–634 (1955).

connexions. All the contact surfaces had been flame-cleaned and left unpainted.

Since the first American specification for assembly of structural joints using high-tensile-steel bolts was issued in January 1951, there had been a phenomenal increase in the use of bolts for bridges and structures. There was no British Standard for high-strength bolts, although there was nothing which precluded their use. Until an addendum was provided to B.S. 153 or B.S. 449 or an entirely new British Standard was issued, it

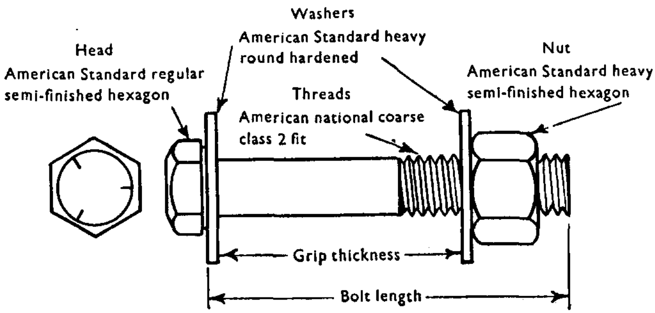

FIG. 24

would be helpful if the Authors could publish their own specification, with any other relevant information or instructions which they normally issued.

Mr T. Baldwin (Superintendent, Engineering Division, British Railways Research Department) discussed the question of welding connexions to tension flanges. On p. 605 it was suggested that a longitudinal weld connecting the stiffener to the flange was probably no worse than a rivet hole, but he had looked up a Paper—now almost a classic—by Wilson [8] which gave some figures on that point.

Where there were rivet holes through the tension flange of a rolled I-section, the limiting stress for an endurance of 2 million cycles was 0–6½ tons/sq. in. Where there was a small plate attached by a longitudinal fillet-weld the limiting stress was 0–6·1 tons/sq. in. Where the small plate was attached by transverse fillet-welds the limiting stress was 0–5·5 tons/sq. in. It therefore seemed that rivet holes caused rather less weakening than a longitudinal fillet-weld and that transverse fillet-welds were worse than either.

Those figures should be considered in relation to the fatigue strength of a fabricated girder. A small fabricated girder stood 0–7½ tons/sq. in. for the same endurance. When stiffeners were attached to the tension flange

[8] W. M. Wilson, "Flexural Fatigue Strength of Steel Beams." Univ. Illinois Engg Expt Stn Bull., 377 (1947–8).

of a rolled I-section by transverse fillet-welds, the endurance range was higher. The conclusion was reached, based on those few tests only, that there was no theoretical objection from the fatigue standpoint to welding stiffeners direct to the tension flange of a fabricated girder. That might not apply to the heavier welding on full-size bridges, but, unfortunately, no further information was available on large-scale girders. More tests were required, although he did not know how much testing work was available during the formulation of B.S. 153.

It seemed better not to weld to a rolled I-section, for welded attachments to the tension flanges of such a section reduced the endurance considerably, but with a fabricated girder, where strength was apparently already lost by the longitudinal weld connecting the web to the tension flange, the fillet-welds to the stiffener did not seem to be objectionable.

On the question of fillet-welds, it seemed almost worse welding unstressed material to the stress flange than welding the web to the flange, because the whole of the force necessary to make the stress in the attachment equal to that in the flange had to be transmitted by the welds, if the attached piece had appreciable length. If it was short some advantage would possibly be gained by deflexion of the welds, which might explain why welding stiffeners was not quite so bad as attaching a plate 4 in. long by transverse fillet-welds.

In regard to the mechanical properties of the heavy sections used for thick flanges, they might not always be up to expectations. Difficulties arose as to whether to test them near the surface or near the centre. There was a specification (B.S. 15 : 1948) stating what should be done with sections exceeding 3 in. square, but it did not clearly cover flats with one side less than 3 in. and the other exceeding 3 in. If such sections were made from strips cut from a big plate, the mechanical properties were probably inferior to those from universally rolled plate. Mr Baldwin would like to see universally rolled plate used for those jobs wherever possible.

He asked for comments on the relative advantages of the Authors' method of construction and the method using two cover plates attached by high-tensile bolts to the stiffeners, connecting them to similar transverse plates on the prefabricated deck units. Whilst that method had its own difficulties, it was a little more flexible in the tolerances necessary during manufacture.

Dr R. Weck (University Lecturer in Engineering, Cambridge) said that, in considering the problem of fatigue in railway bridges—and in other types of structure—it should be remembered that the attitude to fatigue in structures had undergone a gradual but revolutionary change within the past 20 years. Fatigue strength had been considered a mechanical property possessed by a material, in the same way that it possessed a yield point, a tensile strength, and an impact value. It appeared correct on that basis to reduce the permissible stresses in a structure subject to fatigue in proportion to the reduction of that fatigue strength of the material.

It was now realized that fatigue strength was not a property of the material, but of the structure, and therein lay a very important difference. Consideration of the fatigue strength of a structure involved a problem very similar in its complexity to the instability problem in anything but pin-ended struts. There was no such thing as the buckling strength of the material as such, but only the buckling resistance of the structure or a structural component of a definite shape, length, and method of construction. In the same way the fatigue strength of a structural component had to be considered.

Materials possessed nothing that could be called buckling strength, nor anything that could be called fatigue strength, which remained constant irrespective of form, design, and character of the structure. Just as, to solve buckling problems, first principles and experiment had to be used, so fatigue problems had to be treated in the same way.

That change of emphasis in looking at the fatigue problem was clearly expressed by the way the pioneers had tried to deal with it in railway bridges 50 years ago compared with the present-day methods. Fifty years ago, as the Authors had shown, fatigue was dealt with by the Launhardt–Weyrauch formula, which consisted merely in reducing the permissible stresses, irrespective of design detail, as a function of the ratio of maximum to minimum stress as if that was the only important factor influencing fatigue strength.

Nowadays, that had come to be recognized as complete nonsense. It was possible that two separate parts of slightly differing design made from exactly the same material would have vastly different fatigue resistance, although the ratio of minimum/maximum stress was the same.

B.S. 153 in use pre-war had not even gone that far. The Authors in saying that bridges in the past had been designed on the Launhardt–Weyrauch formula, could not have included bridges designed to B.S. 153. At the bottom of Fig. 3 the Authors showed attenuation of stress also for positive ratios of maximum and minimum stress, whereas clause 19 of B.S. 153 compelled the designer to make the allowance only for reversal of stress —if the stress changed from compression to tension in a member—as if changes between purely tensile limits had no effect.

In the new B.S. specification it was partially acknowledged that effective strength of a part depended on detailed design by making different allowances for different methods of construction. In his view, however, that had not gone nearly far enough and he doubted if it was possible to go far enough by the method of stress penalties which admittedly was the only practical method in a specification.

Specifications in general did not attempt to deal with more complex buckling problems by the method of stress penalties and left it to the skilled designer to ensure stability by constructive measures. It would be more rational to deal with the fatigue problem in the same manner, that was, not to make reductions in permissible stress but to leave it to the designer to pay the greatest attention to detailed design and to use only such details as had been proved either in experiment or in practice to withstand the repeated applications of stress.

He was glad to see one Paper clearly stating that fatigue failures in early bridges had occurred, but regretted that the Authors had not given more pictures and examples, because it was only by looking at failures that such mistakes in the design of details could be avoided.

Brigadier C. A. Langley (Inspecting Officer of Railways, Ministry of Transport and Civil Aviation) said the Authors had stated that many old bridges were still standing up to modern loading, and it seemed to him that a tribute was due to the pioneers who, with their little experience, designed such bridges. A report by one of his predecessors on the Saltash Bridge before it was opened in April 1859, showed how prudent and far-sighted Mr Brunel had been in design. The main elliptical tubes, of 450-ft span, were tested before erection with a static loading of $2\frac{3}{4}$ tons/ft run—higher than the loading produced by the Bridge Committee's formula now standard on British Railways.

He was glad that the Authors had specially mentioned the possible dangers of corrosion on members subject to fatigue loading. But were they fully satisfied that a waterproof joint between concrete flooring poured in situ and the main girders could be obtained? Would not cracks arise owing to the flexing of the webs which would allow moisture in? How could that be spotted by the bridge examiners, and how could it be overcome? With prestressed concrete deck units there appeared to be quite a space between the flooring and the main girders but he wondered whether enough had been allowed at the bottom where dirt could accumulate which might be difficult to remove completely. Would it be possible to lift the deck units a little to provide slightly more room underneath? He thought that headway clearance might be one of the snags.

The Authors mentioned brittle fractures and that steel in the tension flanges was based on Lloyd's P.403 rule, but at the top of p. 612 they suggested that flange plates up to 3 in. thick could be used. He thought there might be some difficulty in obtaining plates of that thickness in notch ductile steel, because he understood that the rolling mills were not prepared to roll plates thicker than 2 in.

Would high-strength bolts remain tight for the life of a bridge? Had the Authors enough data to confirm that? If not, it seemed they might fail very quickly through shear.

Mr A. M. Sims (formerly Chief Engineer, North Western Railway, Lahore, India, Indian Railway Service of Engineers (retired)) wished to draw attention to three special points. Fig. 3, on p. 606, included the modified Launhardt–Weyrauch formula, which had been criticized by Dr E. H. Salmon. In place of the "f ultimate" given in that formula, apparently 9·5 tons/sq. in. had been substituted. The two lines in Fig. 3 should apply as fatigue limits to a material for which the ultimate strength figure was 9·5 tons/sq. in. The application of a factor of safety of about $2\frac{3}{4}$ raised again the question whether legitimate use had been made of the formula. Perhaps the Authors would give their reasons for using it in Fig. 3.

Fig. 6 on p. 607 indicated that there was a lateral deformation of the top flange which would give rise to an additional secondary stress in that flange. Could the Authors give any indication of the form of that lateral deflexion of the top flange and say if any measurements had been made to ascertain the amount of that secondary bending stress? Some research work had been done at Bristol University but Mr Sims had not yet seen the results.

He was glad that the calculation of secondary stresses in general had been brought forward so strongly in the Paper. From p. 607 it appeared that there was an effective secondary stress 19% in excess of presumably the primary stress. That excess might be acceptable under the Indian Steel Bridge Code but it was doubtful whether it would pass B.S. 153 or the Institution's Code.

It was important to remember that methods of calculation of secondary stresses usually employed and those mentioned in text books in general were not sufficiently accurate in their results, yet all the codes assumed that they could be calculated accurately and correctly. For instance, at the foot of p. 603 it was stated: "Floor members and their connexions in through-type spans have a lower endurance limit."

An unpublished Paper by Professor Philpot on secondary stresses in verticals gave the proper theory and showed for a particular case that the text-book line-frame method gave a secondary stress in the vertical member of 1·75 ton/sq. in. at the level of the top of the cross-girder, but if the dimensions of the cross-girder and the vertical and sway bracing were correctly taken into account, the secondary stress amounted to 2·96 tons/sq. in. The primary stress was $6\frac{1}{2}$ tons/sq. in. in the vertical. According to text-book methods, the secondary stress was 27% of the primary stress, but by correct calculation it was $45\frac{1}{2}$%. In that respect the codes might lay down more definitely and accurately the methods of calculation for determination of the various kinds of secondary stresses.

It appeared from p. 607 that the end-fixing moment had been based on results from one particular span with cross-girders at 6-ft centres. Would the Authors recommend the results given on p. 608 for application to all half-through type girders or could they add an Appendix showing a method of calculating the secondary stresses of general application to all half-through type spans?

If, in addition, they could include a sample calculation showing the method used for any of the points on the calculated stress curves in Plates 1 and 2, it would be valuable to all engineers responsible for design, maintenance, and administration.

Mr O. A. Kerensky (Principal Bridge Designer, Freeman, Fox & Partners) said the Paper rightly emphasized that attention should be paid to interaction between bridge elements. That applied to all designs, not only to those for half-through plate girder bridges. With greater intensity of permitted stresses, the neglect of the effects of interaction between deck, laterals, and main girders could no longer be allowed.

In all bridges the arrangement of stringers, cross-girders, and laterals should be such that either the interaction between them and the main girders could not take place or, on the contrary, all the elements were made to act together and the connexions designed accordingly.

Fatigue was no new problem; Benjamin Baker made allowances for fatigue in the Forth Bridge. But the problem had been forgotten and there remained only an inadequate stress reversal clause in B.S. 153. The introduction of welding re-emphasized the danger, not because welding was bad for fatigue but because the new process was examined by scientists from every angle—and the problem was re-discovered.

Before the 1939-45 war the Germans had some fatigue rules and in 1947 the American Welding Society produced a specification for welded highway and railway bridges, with rather elaborate allowances for fatigue. That specification and results of further research formed the basis for the proposed fatigue rules in draft B.S. 153, which relied mainly on reducing allowable stresses by varying amounts for different numbers of stress cycles, types of construction, and qualities of steel. But, however much the working stress was reduced, bad detail could raise a local stress several times above the average working stress, so that no rules for fatigue would hold, and the designers should bear in mind that simply reducing the stress was no safe answer to the fatigue problem. He believed that the so-called fatigue failures in bridges were more often caused by over-stressing through neglect in design than by genuine fatigue, because a critical number of stress cycles due to a maximum or near-maximum combination of design loads would seldom be achieved during the life of an average bridge.

The only answer to corrosion fatigue was protection of the steelwork, both in the detail produced in the drawing office and in subsequent painting and maintenance. It was not possible to make allowance for corrosion fatigue in the design of the members. The redistribution of loads between the main girders by the continuous cross-girders should be taken into account in design and the figures given on p. 607 were very interesting. If possible, such data should be obtained for a wider range of half-through and other bridges. The familiar coefficient of 0·8 for continuity on a fairly rigid intermediate support seemed to be confirmed by the field tests.

He had been interested in the forms of decking shown on p. 609. Most of them appeared to be specially conceived for quick renewal of relatively small bridges on existing lines. Prefabrication had been used to its maximum, which in the British Isles was undoubtedly the right procedure, but what about work abroad? What was the Authors' opinion about cast-in-situ reinforced concrete decks? Good vibrated concrete could now be made anywhere in the world. A reinforced concrete deck slab properly connected to steel stringers would also act as a compression flange of the stringers, producing a considerable economy in the weight of steel. Copper-strip bitumen-filled expansion joints could be provided as necessary. Had the Authors any experience of such decks?

On p. 611 the use of high-tensile steel for main girders was mentioned. High-tensile steel working at higher stresses suffered relatively greater fatigue than mild steel, and before considering what savings could be made by using high-tensile steel it would be necessary to agree on the conditions in which the fatigue effects could be ignored. For example in long-span or multiple-line bridges, where the main girders were designed to carry several trains at the same time high-tensile steel would be used in the heavier type girders where the greatest economy would result therefrom. On p. 611 the Authors stated that the compression flange of a half-through plate girder was assumed to have an effective length of not more than 3 times the distance between stiffeners. It was not clear why the effective length should be assumed as it could be found. It had been shown that in many cases it would be only the distance between the cross-girders.

The information about saving in costs of detailing when using welded construction was very interesting. Would the Authors say what detail they included on the drawings? Did they specify the sizes of welds, type of joint, welding procedure, and sizes of electrodes, or did they leave some of those decisions to the works? They had also completely eliminated site welding, but that had been used to a considerable extent on the Continent and perhaps the attitude in Britain should be reconsidered. Welding imperfections would affect the fatigue strength of the weld but not the static strength to any marked degree. Fatigue weakening of the weld with a small fault was no greater, and probably smaller, than some of the other fatigue sources present in structures; therefore perhaps site welding, at any rate in England should be permitted if it showed saving. Would the Authors agree or had they special reasons for prohibiting it?

In regard to high-strength bolts it was essential that there should be no paint or grease between the surfaces used in friction. What precautions did they take to ensure that that was so? Had the Authors any experience with long flange splices? What friction did they allow in determining the the strength of the joint?

On p. 619 the Authors stated that it was difficult to apply torques in excess of 600 lb.-ft. Pneumatic wrenches existed which would easily produce a larger torque. The Authors advocated mild-steel nuts, but if higher torques were used it seemed to him that high-quality nuts might be better. What precautions were taken in checking tension in the bolt? Did they rely on the torque formula or did they measure the extension of the bolt, which seemed a more positive way of obtaining the tension in it?

Mr E. K. Bridge (Assistant to Chief Civil Engineer (Bridges), British Railways, Southern Region) showed slides of two bridges which were completely prefabricated in the shops. The first was of Stewarts Road Bridge, Battersea, in which each of the girders was Z-shaped. The top flange turned outwards and the bottom flange inwards, and the spans were placed side by side with the girder stiffeners bolted together. The matching pairs of girders were built on jigs to ensure their similarity. Maintenance, inspection, and painting could be carried out completely from below and the upper surface was very clean with nothing to catch trailing waggon sheets. The spans were 47 ft 6 in. long $\times$ 11 ft 2 in. wide and they were brought to within half-a-mile of the site by road and thence, as an out-of-gauge load, to their final position. The bridge carried eleven tracks and the adoption of that form of construction effected a considerable saving in the length of time that speed restrictions had to be imposed.

Other slides showed a curved bridge for the Twickenham fly-over, where the depth of construction had to be kept down to an absolute minimum, and the curved main girders permitted shorter cross-girders which could therefore be shallower. Each of the three spans was lifted into place in one piece. The largest was 75 ft long and 15 ft wide and weighed 34 tons. The spans were delivered by road and loaded on to wagons close to the site. The old bridge was taken out and the new spans were installed during one week-end.

Because so many connexions had failed on old bridges, there was a tendency to stipulate a robust cross-girder with a robust connexion to the main girder. On skew girders that could lead to very high stresses where cross-girders were supported at one end on a rigid abutment and at the other end on a main girder, at a point that could deflect considerably.

The Authors mentioned the stresses transferred to the stringers from the main girders through interaction of the deck, and the possibility of that extra stress shortening the fatigue life of the stringers—subject, as they were, to high frequency of loading—should not be lost sight of.

As one of the factors causing bridges to last longer than might have been expected, the Authors had said that plastic strain followed by lapse of time increased fatigue life, but he would like to know if that had been confirmed by more recent research.

Unfortunately, points which suffered many reversals were often connexions which were also points of greatest corrosion, and that emphasized the need for frequent patch painting of such connexions rather than painting the whole bridge at longer intervals.

Mr A. N. Butland (the Assistant Civil Engineer, British Railways, London Midland Region) said he was interested to read in the Paper of faults in design 20 or 30 years ago, when the cross-girder end had often not been designed satisfactorily, but were not the Authors taking too grim a view of those failures? They had chiefly been of cross-girder connexions, of the type illustrated so clearly, which was a much earlier variety; he still had great faith in the riveted structure.

He was not aware of any serious trouble developing in such structures, apart from minor points, such as lack of waterproofing at the ballast girder and lack of success in keeping out the water from the main girder web. Now that steel was becoming less difficult to obtain, how far would they be justified, in undertaking a very big bridge problem quickly, in returning to that type of construction?

He was interested in the predilection said to be shown for welding flanges to webs of main girders before welding the stiffeners. He had been told that if stiffeners were welded first to the web of a 60-ft plate girder there was a shortening of about $\frac{1}{4}$ in. so that presumably if the stiffeners were added later there was a lot of locked-up stress. Was that an accurate statement or an exaggeration?

The economy to be derived from welded work was largely dependent on the ability to use thick plate and so reduce the amount of welding to a reasonable quantity. Taking a 40–50-ft girder with top flange and web to B.S. 15 and bottom flange to Lloyd's P.403, how did the cost compare with straightforward riveted girders? He noted that the Authors recommended riveting for spans exceeding 90 ft.

He had been comforted by the assumption which could be drawn from the tests at Derby that it was safe to design cross-girders as if they were simply supported, for the main section. Turning to Fig. 14 on p. 615, did not the Authors think that the ballast boards illustrated on a slope against the web of the main girder would allow water to trickle down the vertical web and accumulate on the horizontal portion of the flange away from the drying action of the wind? Would it not be better to have a welded strip along the web turned outwards to throw the water off? Fig. 18 on p. 618 showed very ingenious connexions for troughing, but it seemed that there was a bolt in the middle of a troughing flute, which was to be filled with tarmac or concrete. There was one bolt per flute. Did the Authors expect any trouble with corrosion of that bolt? Would it not be troublesome to replace it? If the development of corrosion in the life of that bolt was not noticed, was it not possible that under vibration there might be a slight outward movement of the main girder relative to the cross-troughing, bearing in mind that there was a small landing only on a shear plate? Would not that produce a difficult condition? Possibly there was a complete through-tie between the main girders which was not illustrated.

On p. 619 dealing with high-strength bolts, it was stated that the force P could be increased only by elongation of the bolt, thereby separating the plates. Was it not true that as the load on the bolt increased so there was an equal reaction on the plate and a smaller indentation of the plates themselves, so that force P could be increased without separating the plates but with a decrease in the elastic compression of the plates being held together ?

He was pleased to notice the reduction in number of members to be handled at site. That was very important. Could more information be given about the precast jack arches ? Could not the Authors go further and have one jack arch which would fit between two cross-girders for, although there would be a certain tension developed in the concrete during the lifting, it would not be a big expense (and there would be a saving in the erection work) with the use of a couple of light rods to take care of such tension. Any reduction in the number of lifts was worthwhile.

In permissible stresses and fatigue, civil engineers seemed to be wavering between an estimation of strength based on ultimate strength, an estimation based on yield point, and an estimation based on fatigue life. If the life of old bridges was to be assessed on a fatigue basis, more work was necessary on the frequency of maximum loading. It would be interesting and useful to know with what frequency the maximum calculated loading was experienced, particularly with steam locomotives. In the admirable method shown, dealing with fixing moment at the end of the cross-girder, stresses could be calculated and checked by static loading and modern strain gauges, but it was necessary to go further and see how frequently among the many loadings on the bridge one approaching its ultimate design loading was obtained. A histogram would be very useful.

Economy in use of material could go too far and some reserve of strength was very desirable. He welcomed any tests or revision of design methods which would justify an increase in the life of old bridges but he hoped to see new bridges designed still with an ample reserve of material since a considerable reserve strength was obtainable at a much smaller percentage increase in overall cost.

Mr W. T. Everall (Consulting Engineer in private practice) referred to the use of high-strength bolts, mentioned on p. 619. During the past 3 years he had been concerned with the use of such bolts for connexions in a number of mild-steel railway bridges under development. In the last paragraph, under Table 1, the Authors wrote :—" For practical reasons it is usual to limit the size of bolts to $\frac{7}{8}$-inch diameter because of the difficulty of applying a torque in excess of 600 lb.-feet." For the bridges in which he was interested, all the bolts were of 1-in.-dia. high-tensile steel with unified fine threads—12 t.p.i.—and had a minimum ultimate strength of 51 tons/sq. in. In practice it had been found that clamping forces of 20–23 tons tension in a 1-in. bolt could be obtained by a torque load of 550–600 lb.-ft, the value of 20 tons representing a minimum of 90% of the yield point.

The additional 15% above the minimum provided in the range of bolt tensions covered the dispersion of the load which occurred in group connexions and the correction of small discrepancies in the making of the contact surfaces as well as the working tolerances in operating tools for tightening the bolts.

The situation of the bolt groups permitted 90% to be tightened by pneumatic-impact wrenches and the remainder, where the space was confined, by manually operated mechanical wrenches.

A reduction in the torque was obtained by using unified fine threads and the application of graphite grease to the threads of the nut. The torque required was reduced to 575 lb.-ft compared with 700 lb.-ft required in Table 1.

Where the operating space for tightening the bolts was limited, a mechanical tool had been developed using a worm and worm-wheel geared wrench operated by a short lever arm from a reversible ratchet device with an automatic release when the correct load on the bolt was reached. The adjustment for torque calibration was by means of a test rig. The necessary anchorage was obtained by a prolonged extension of the thread portion of the bolt to which the gear wrench was connected.

Recently one of the bridges having several thousand high-tensile-steel friction bolts in its assembly was subjected to preliminary tests. The live load consisted of two locomotives of the 4–8–0–G–16 Tank type, representing about 90% of the full loading capacity of the bridge and providing a very severe impact effect, and running at various speeds. The preliminary trials were very encouraging. That railway bridge was one of the first in Britain to be subjected to full-scale tests in which all connexions had been made with high-tensile-steel friction bolts and with only a small number of drifts for location purposes.

In the introduction attention was drawn to the erroneous practice common in Britain up to recent times of designing cross-girders as simply supported beams and reference was made to the phenomena of interaction between floor and main girders which formed the subject of a Paper read before the *Institution* in 1923.

So far as former Indian Railway practice was concerned, during the 1920s and 1930s for three-girder bridges and spans carrying roadways outside main girders bracketed from the rail floor the cross-girders and connexions had usually been designed correctly for continuity. Special expansion joints were provided in longitudinal floor members where necessary.

The incidence of failures in members and connexions in well-designed structures was small in India and America during that period. Designers were probably influenced by Waddell [9] who stated :—

> " No one has yet shown that the repetition of stresses not exceeding the elastic limit has ever produced rupture and as the intensities of working stresses in bridge members rarely exceed half that limit, it is

[9] J. A. L. Waddell, " Bridge Engineering," vol. 1, 1st Edn, Chapman and Hall, London, 1916.

evident that in properly designed structures, the fatigue of metal is non-existent."

As a result of more recent carefully conducted experiment and because of the development of defects in service in many parts of the world, more ample provision was to be made for fatigue.

The substitution of high-strength bolts for site rivets both for shear and for tension connexions which was becoming increasingly popular appeared to have originated with the pioneer work of the late Professor C. Batho.[10]

*** **Mr R. G. Braithwaite** (Principal, Braithwaite and Co. Structural Ltd, West Bromwich) referred to the Authors' remarks regarding the difference in quality between welds laid by hand and those laid by machine. The present method of assessing the strength of fillet-welds had been established as a result of investigation of many hundreds of manually laid welds, and appropriate allowance for discontinuity of electrodes, fatigue of operator, and other deterrents had been made.

Those items were nullified by the use of automatic welding equipment and it would be pertinent to know if the Authors used higher working stresses in consequence.

Furthermore, the deeper penetration obtained by the use of automatically controlled welding feeds provided a much stronger and more consistent weld than that laid by hand.

It was unfortunate that since fillet-welds were normally accepted on visual examination of weld contour and surface appearance no account was normally taken of that factor.

It therefore appeared that the economical development of improved methods of welding was being penalized, and the Authors could help by indicating what stresses they would allow those welds to carry.

The Authors, in reply, agreed with Mr Godfrey, that with the increasing use of high-strength bolts in lieu of rivets for field joints, power-driven impact wrenches would inevitably follow, as indeed they had done in the United States; but in the somewhat limited application on the bridges described in the Paper, the numbers of bolts had been too small to justify taking an air-compressor to site. The bolts had been tightened initially with ordinary spanners and the final torque completed with hand-operated torque-limiting spanners. In the attachment of deck units to the main girder stiffeners there was not enough space in which to operate a power-driven impact wrench. The contact surfaces were not painted whether shear plates were used or not, since it was important not to have anything between the steel parts which, in the course of time, might reduce the tension in the bolts and possibly lead to some loosening.

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[10] Second Report of the Steel Structures Research Committee. H.M.S.O., 1934, pp. 176–177; and Third Report of the Steel Structures Research Committee. H.M.S.O., 1936, pp. 394–401.

*** This contribution was submitted in writing after the closure of the oral discussion.—SEC. I.C.E.

The Authors agreed that the British nuts they used were smaller than the American, being about 8% smaller across flats and 7% shallower ; but they had had no trouble with them.

The Western Region specification for high-strength bolts and the instructions issued for their use were given in Appendices I and II (pp. 621 and 622).

Referring to Mr Baldwin's statement that there seemed to be little or no objection from the fatigue standpoint to welding stiffeners direct to the tension flange of a fabricated girder, the Authors thought it depended on the form of the stiffener and whether the welds ran parallel with or at right angles to the lines of stress in the flange.

In the method of construction using high-strength bolts passing through flanges on the face of the stiffeners the bolts were in direct tension. Fewer bolts were needed to carry the bending moments at the ends of the cross-girders than if the joint were made by bolted cover plates attached to flat-plate stiffeners, since the bolt group would then be in shear and the modulus about the centroid would be less than for a group in tension, where the centroid would be near the outer end of the group. The fact that the bolts could be reckoned to have values for double shear would be offset by the cover plates having to be bolted on either side of the joint. The former method of construction had the practical advantage that, during erection, prefabricated deck units could be lowered down between the top flanges of the main girders ; the clearance holes for the bolts provided all the tolerance necessary for assembly. In the other method of construction the location of a deck unit having two cross-girders, each of which would have to be lined up simultaneously with a flat-plate stiffener, would not be so easy. The flanged stiffener was very desirable in transferring traction and braking forces from the deck to the main girders because without the flange such forces would bend a flat plate stiffener about the welds connecting it to the girder web.

The Authors were in agreement with Mr Baldwin's comments on the mechanical properties of plates used for thick flanges ; the doubts he expressed had been confirmed by tests. The provisions of B.S. 15 were in need of amendment to ensure that tensile and bend test pieces could be taken from any position in the section of the material.

The Authors were grateful to Dr Weck for his comments, with which they agreed They confirmed that, up to 1945, bridges on the Great Western Railway had been designed, not to B.S. 153 but to a range-of-stress formula of Launhardt–Weyrauch type similar to that given in Fig. 3. Notwithstanding the logical objections to so doing, expressed by Mr Sims, the results had been quite satisfactory.

Commenting on the figures relating to bolt tightening given by Mr Everall, the Authors thought the remarkably low torque values could be ascribed more to the graphite grease, or to the quality of finish on the threads, than to the use of fine threads. The means described for tightening the bolts were similar to those used by the Authors, except for the prolong on the bolt.

Replying to Brigadier Langley, the Authors agreed it was not generally possible to obtain a waterproof joint between concrete haunching and the web of a plate girder. For that reason, the space between the decking—whether it consisted of prefabricated prestressed concrete units or concrete jack arches—and the main girders was left open for painting and maintenance. At girders between tracks, the width of that space had been governed by the room available between the ends of the sleepers in the six-foot way. The construction depth was often a controlling factor which meant that the underside of the deck had to be as nearly as possible level with the bottom flanges of the girders. But even when that was done it was still possible to arrange for the whole of the exposed steelwork to be readily accessible. Where transverse troughing rested on shear plates on the web of a girder as shown in Fig. 18, the waterproofing had been taken right up to the web and the joint sealed with bituminous plastic compound whilsc steel drip-plates welded to the web prevented water from getting down behind the protective concrete haunching above the joint.

When the Paper was written, special steel based on Lloyd's P. 403 rule had been specified for the tension flanges of welded girders. That had been obtainable in thicknesses up to 3 in., the limit above which, in Britain, plates could not be straightened by rolling. Subsequently, with the introduction of N.D.1 steel, the thickness of plates had had to be limited to 2 in.

The experience of American railway engineers, confirmed by the examination [7] of bolted joints in fifteen bridges since 1948, showed that high-strength bolts were more reliable than rivets for remaining tight in service.

Replying further to Mr Sims, the Authors agreed that little was known about the secondary stress in the top flange of a main girder. Stresses recorded at the inner and outer edges of the top flange of an outer girder in a 40-ft span such as that illustrated in Fig. 13 showed that near the centre of the span, the increment of stress when the adjacent track was loaded, was 16% greater on the outside edge of the flange than on the inner edge. But towards the bearings the stress increment on the inner edge was the greater by nearly 22%. That showed that the tendency for the flange to bow inwards near the middle of the span was counteracted by the restraining influence of the reactions on the bedplates at the ends of the girder.

The stresses produced by the increased loading on the centre girder, mentioned on p. 607, were treated as primary stresses but the centre girder was not designed for more than 10% increase in the live load, for the reasons given.

A mathematical treatment of the distribution of loading between the main girders would have to take into account the flexibility of the stiffeners and the torsional rigidity of the main girders. The Authors were relying

[7] See reference 7, p. 627.

on the more direct guidance furnished by the tests they had made, the results of which were embodied in Fig. 7. They would agree that there was room for further exploration of the problem, both by experiment and analysis.

The sample calculation suggested by Mr Sims was given in Appendix III, p. 623, which had been added to the Paper since the printing of the advance copies.

Replying to Mr Kerensky, the Authors said they had been concerned principally with the reconstruction of existing bridges on British Railways where there was no time to build in-situ-concrete decks. It might be possible to make good vibrated concrete anywhere in the world but, in their opinion, concrete made under ideal conditions in a depot would be superior to and less costly than concrete cast in situ on a bridge deck where trains had to be allowed across before the concrete was fully cured.

The Authors agreed that the effective length of the compression flange could be accurately determined by analytical methods. In the type referred to, that length had worked out to more than twice the distance between cross-girders (or effective brackets) and so the length had been taken as three times that distance.

With regard to the detailing of welded girderwork, the information given on the drawings had included the sizes of welds, the types and arrangement of welded joints, and all other particulars needed for fabrication in the works, except that the sizes of electrodes to be used and other details of welding procedure were left for the contractor to submit for the engineer's approval. Site welding had been avoided because the quality of the work done in the field, often under adverse weather conditions, was unlikely to be as good as that done under cover in a works. Field welding on a railway bridge was likely to be hurried and subject to frequent interruptions, which would not be conducive to good workmanship.

To ensure that contact surfaces to be fastened by high-strength bolts would be kept clear of grease or paint, no further precaution had been found necessary than a note on the drawings and in the specification. In the design of joints, high-strength bolts of the same nominal diameter could be used as a straight substitute for rivets, relying on the tension in the bolts to clamp the bearing surfaces together so firmly that friction would prevent any slipping of the bolted parts. The safety of that assumption had been proved in practice. It was not necessary, in design, to consider what would be the value of the coefficient of friction involved, but it should be observed that a coefficient no greater than 0·3 would develop enough friction, under the high clamping force provided by the bolts, to equal the shear load allowed on rivets of the same size.

The difficulty in applying torques of more than 600 lb.-ft arose when high-strength bolts were used in situations where access was restricted by adjacent structure, but it had been overcome by the use of a torque-multiplying wrench (Fig. 25).

With regard to the suggestion to use high-quality nuts for high torques, mild-steel nuts were more suitable for bolts of 45/55-ton steel, because the softer steel would facilitate, by plastic deformation, the adjustment of the nut thread to the bolt thread, and so permit a more even distribution of the loading throughout the thickness of the nut under the tightening torque.

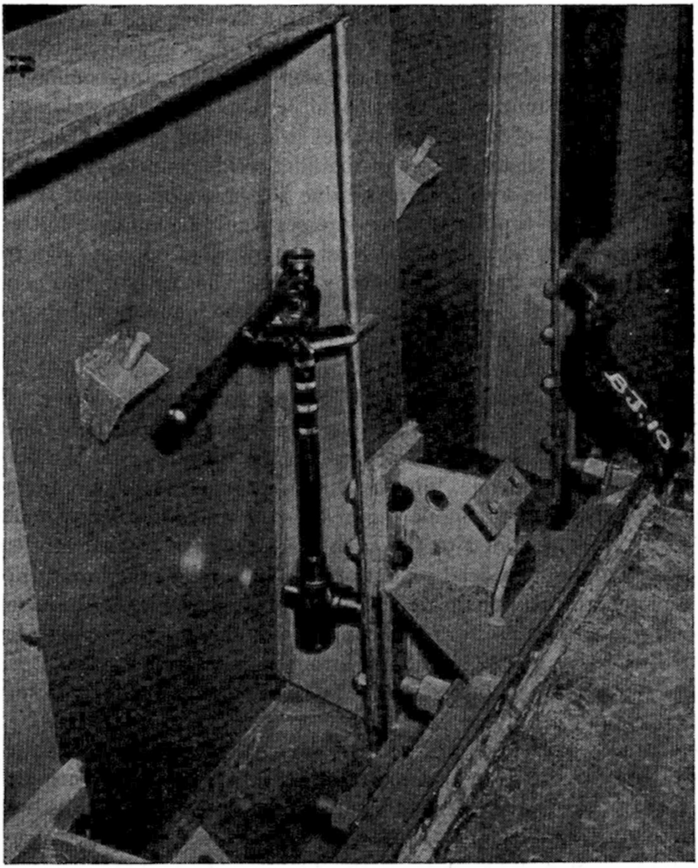

FIG. 25.—TORQUE-MULTIPLYING WRENCH WITH TORQUE-LIMITING SPANNER

No direct precaution was taken to check the tension in the bolts: the Authors relied on the torque formula. The fact that the tension could vary by about 20% above or below the value given by the formula, according to the quality of finish of the threads, would not detract from the performance of the bolt in service.

Replying to Mr Bridge, the Authors agreed that the deformation stresses in deck members caused by the unequal deflexions of main girders in skew spans should be allowed for in design. They had dealt with that problem in a manner similar to that of the design of the cross-girder connexions at the centre girder of a double-track bridge, where similar deformation took place when the span was loaded on one side of the centre girder.

Referring to Mr Butland's contribution to the discussion, the Authors said they did not depreciate the merits of riveted construction, and first-class riveted work was still obtainable. But riveting was a dying trade. Welding was now of recognized reliability. Advances in welding technique were steadily proceeding, and the time was inevitably approaching when whatever could be done by riveting could be done better by welding.

On the question of locked-up stresses in a welded girder, the Authors had considered the shear stresses in the fillet-welds connecting the flanges to the web when the web contracted due to the welding-on of stiffeners afterwards. At the top flange those stresses would be additive but at the bottom flange where they mattered most they were of opposite sign to the principal shear stress under the working load. The Authors had come to the conclusion that the locked-up stresses could be ignored, and they were not aware of any practical evidence to the contrary.

Comparison of the costs (May 1955) of 50-ft riveted and welded girders of equal strength showed a saving of 7·8% in favour of the latter. That was in spite of an extra charge of £8 per ton for 1·92 ton of N.D.1 steel in the bottom flange of the welded girder. In the riveted girder the division of costs was 31% and 69% for the materials and the labour, including overheads, respectively. Similar figures for the welded girder were 32·5% and 67·5%. To be added to the saving in favour of welded fabrication there was a reduction in dead weight of 11·5% as well as the lower drawing office and future maintenance charges. Spans exceeding 90 ft in length had been riveted because for such spans it was cheaper to use high-tensile steel to B.S. 548; so far, welding of either that steel or steel to B.S. 968 had not been generally adopted for the fabrication of girderwork for railway bridges.

Referring to the single row of high-strength bolts connecting the end plates of the troughing to the main girders in Fig. 18, the Authors considered that the risk of corrosion on the shanks of those bolts was no greater than it would be on the shanks of rivets because the steel plates were held in close contact by the clamping force of the bolts.

It was not possible to use a single jack-arch unit extending across the full width of a bridge because, under the deflexion of the cross girders, such a unit would be subject to bending stresses for which it could not be conveniently designed.

The Authors welcomed Mr Braithwaite's remarks on the merits of automatic welding but they could not agree there was a case for raising the permissible working stresses on welds, since for full-penetration welds the Codes already allowed working stresses not less than those permitted in the parent metal.

The Authors were glad Mr Everall had drawn attention to the pioneer work of Professor Batho. An early application of high-strength bolts had been in the Callender-Hamilton bridge.[11]

Correspondence

Mr E. M. Lewis (Director of Research, W. S. Atkins and Partners) stated that the combination of welded steelwork with high-strength bolted friction-grip joints, together with integral steel and precast concrete elements used by the Authors led to rigid structures. The older riveted and bolted forms of structure were " free to breathe " but the new forms were not. In many old bridges the demands made upon that lack of rigidity in the connexions had led in service to working rivets and loose and fatigued bolts and to fretted and corroded surfaces. Those troubles had called for considerable and continuing inspection and maintenance throughout the life of the structures. The Authors had done much research which should lead new bridges to be far more trouble free, particularly as a result of the steps they had taken to guard against fatigue and corrosion. However, there was one feature which might be common to all the bridges except those with precast jack arches which, on the face of the data given in the Paper alone, would seem to portend some trouble in the years to come. Mr Lewis wished to ask the Authors for details of precautions which they had probably taken but which were not clear from the text.

It was not clear from Fig. 14 if the bottom of the stiffener was welded to the flange or if it was free ; no weld was shown but such welds were referred to in the text. If the stiffener was free then since the bottom flange of the girder and the adjacent web stretched under load the distance between each pair of stiffeners would be increased and they would flex in double curvature as indicated in the diagram in Fig. 26. The secondary bending stresses f_s in the stiffener due to deflexion δ might be considerable ; they varied in proportion to the quantity Lt/l^2. As an indication of the order of such stresses if it was assumed that the stiffener deflected as a fully encastered member over the length l then taking $L = 50$ in. and $t = 0{\cdot}4$ in. and assuming a live load stress of 5 tons/sq. in. in the main girder web, then $f_s = 7\frac{1}{2}$ tons/sq. in.

That simplified computation indicated that the true value of the secondary stresses close to the welds attaching the stiffener to the web might be about 5 tons/sq. in., allowing for a degree of flexibility in the stiffener connexions to the web. Those secondary stresses added to the primary stresses arising from live load ; hence it seemed that every time a train passed over, the stress range locally might be about 10 tons/sq. in. Furthermore, if there was any continuity between adjacent deck

[11] A. M. Hamilton, " Bolted connections in structures." *Engineering,* vol. 165, p. 433 (7 May, 1948).