Messrs A. J. S. McLaren, M.C., A.M.I.C.E., J. G. Probin, B.Sc., A.M.I.C.E., I. B. Petrie, B.Sc., and W. J. S. Goodman. The Authors wish to record their appreciation of the assistance rendered in the preparation of the Paper by Mr W. A. Wilkins, B.Sc., A.M.I.C.E.; and finally their thanks to the Directors of Messrs Holloway Bros. (London) Ltd for defraying the cost of investigation work and allowing much of the contents of the Paper to be published.

References

1. A. E. Reid and F. W. Sully, The Construction of the King Feisal Bridge and the King Ghazi Bridge over the River Tigris at Baghdad. Instn Civ. Engrs, Works Construction Paper No. 4, 1946.
2. M. G. Ionides, " The Regime of the Rivers Euphrates and Tigris." Spon, 1937.
3. W. Storey Wilson and F. W. Sully, " Compressed Air Caisson Foundations." Instn Civ. Engrs. Works Construction Paper No. 13, 1949. (*Vide* Figs 2 and 3, Plate 1, and *Fig. 14*, facing p. 15.)
4. A. M. Ward and E. Bateson, " The New Howrah Bridge, Calcutta; Design of the Structure, Foundations, and Approaches." J. Instn Civ. Engrs, vol. 28, p. 167, (May 1947).

The Paper is accompanied by fifteen photographs and forty-three sheets of tracings, from which the half-tone page plates, the folding Plates, and the Figures in the text have been prepared.

Discussion

Mr R. D. Gwyther said that in 1936 his firm had been asked by the Iraq Railways to furnish estimates of the costs of different kinds of bridges, including a single-line railway bridge, a combined railway and road bridge, and a bridge with a road and railway each having its own track. He approved the choice of the last-named type.

The ground contained sulphates to an extent that was injurious to ordinary Portland cement concrete, and so the use of vibro-piles had had to be rejected in favour of reinforced-concrete piles. At that time, preference would have been given to the use of a cement, produced in Egypt, called " seawater " cement which, because of its low tricalcium aluminate content, withstood the action of sulphates. The next best proposition was the " oilwell " cement used by the oil companies for lining the tubes of their oilwells. That seemed to possess characteristics which, although not quite of the same nature, were similar to those of Egyptian seawater cement, and so its use was specified and adopted. In addition to using special cement for making the piles, the precaution was taken of having them dipped in a bath of sodium silicate in order to harden the surface.

In order to obtain some information about the behaviour of concrete in such soil, a number of cubes were made and buried in the ground; they were as follows:—

Two of normal Portland cement
Two of Portland cement, hardened
Two of Oilwell cement, unhardened
Two of Oilwell cement, hardened
Two of Portland cement, coated with bitumen
Two of each type of cement, cast in the ground

Those cubes had been buried for 2 years in a bad situation. The sulphate-ion content of the ground was of the order of 8·57 per cent. and the groundwater had a *p*H value of 8·9. Theoretically, both of those factors were liable to have a very serious effect on mass Portland cement concrete. Mr Gwyther hoped that in 10 years' time somebody would take those cubes out of the ground and give a report on their condition.

In connexion with the use of the vibro-pile frame, Mr Gwyther had been disappointed when it was found that the vibro-pile frame could not be used, for it was an excellent machine. However, it had been used freely over the site to drive a shoe down for test purposes. The tube had been withdrawn and the hole filled with sand. Very useful information had been obtained, and he suggested that anybody with an ingenious turn of mind might not find any difficulty in using a vibro-pile frame to drive a shoe into the ground, and then plant a reinforced-concrete column on the shoe, grout it up, and have, as a result, something which was far better than a driven pile, knowing nothing of what had happened after it was hammered into the ground.

Mr Gwyther corroborated the Author's satisfaction with welded caissons and said that he would have no hesitation in accepting a welded caisson, including the shoe.

He was not so confident regarding pre-stressing. In the first place, great accuracy was required in fabricating and drilling; and on the site still greater care was required in erection, since the deformation which was produced by pulling structures into shape and upon which the whole theory depended was small—in some cases, no more than the usual tolerances and clearances normally allowed. An alternative to pre-stressing would be to calculate the deformation stresses and allow for the increased material in the scantlings; with modern methods of calculation that was not too laborious and, under the latest specification and code of practice, even less so. Mr Gwyther was not yet sure whether, in a bridge of that nature, pre-stressing was worth while.

The Paper had described tests which were carried out to determine the dynamic stresses, but it did not go so far as to give a clear statement of the objects of the tests or of the conclusions reached. In *Figs 40, 41*, and *42*, the deflexions at selected points produced by a train at speed had been superimposed on those produced by the same train at rest, and it was seen that so far as the main girders were concerned, the impact effects were very small and well within the allowances made in the design. The same was true for the cross-girders in *Figs 43*, although not quite to the same extent. In the case of the stringers the vibrations appeared to have reached a maximum of nearly 0·30, but it had been concluded that those records were unreliable owing to the method of attachment of the vibrograph.

From *Figs 34* it would be seen that the observed static load deflexions approximated very closely to those calculated, when both trusses were fairly equally loaded. On the other hand, *Figs 40*, *41*, and *42* showed that the measured crawl deflexion of the railway truss with railway loading only was 75 per cent. of that calculated. That meant that about one-quarter of the load calculated in accordance with BS 153 was transferred to the roadway truss by a torsional resistance of the whole structure. That was remarkable in view of the small number of sway frames and indicated the efficiency of the relatively deep overhead lateral bracing and the cross-girder connexions in transmitting those torsional effects. Since it was very unlikely in service that both road and railway girders would be fully loaded at the same time, the effect of either load acting separately would be a more equal distribution of loading between the trusses than the calculations indicated.

Mr W. Storey Wilson referred to several points in the design of the bridge which he considered to be worthy of special note.

In the first place, he liked the design of the piers. They looked like piers, and they looked as if they were there to do their job. In some designs, it was sometimes difficult to know whether certain parts of the structure were actually bridge piers or not.

Secondly, the main bearings on the piers were models of neatness and simplicity. They were quite distinct from the old type of bearing which was a large heavy cast-steel base bearing on rollers. The same neatness of design was noticeable in the rail expansion joint.

Thirdly, the design of the viaduct was particularly suitable for the very poor ground, which had a high sulphate-content.

The whole of the constructional work had been carried out without any mishap whatever. It was to a large extent a repetition of the same methods as were used for the King Feisal Bridge and the King Ghazi Bridge which were built before the war. Mr Storey Wilson felt that the Paper could be appreciated fully only by making extensive reference to the Authors' previous Paper.[1]

One of the main troubles encountered was the difficulty experienced by those in England of keeping the site supplied with materials in the order and with the speed with which it was required, and the result was

[1] See reference 1, p. 479.

considerable dislocation of the phasing of the work in Baghdad. For example, during the erection of span SS2, which was to land on stage C, work had had to be stopped because the flood was due and the staging might have been scoured out and collapsed. In that connexion it was rather interesting that a cable which had been dispatched from London recommending that work should stop at that time had crossed a cable from Baghdad making the same recommendation.

During the discussion on the Authors' previous Paper,[1] Mr Storey Wilson had commented on the fact that although the River Tigris carried some 20,000,000 tons of silt down to the sea every year, permission was not granted to a put a few thousand cubic yards excavated out of the caissons into the river. In spite of his remarks on that occasion, it still appeared that a few thousand yards must not be mixed with the millions of tons! It was difficult to understand why that was so, and the only reason he could think of was that when material was excavated out of a caisson it was called "muck," and when material was carried down the river it was called "silt." Apparently it would have been a mistake to have mixed the muck with the silt!

Could the Authors state the degree of accuracy to which the caissons had been sunk?

Mr John Palmer prefaced his remarks by saying that he had had nothing to do with the job which was being discussed, but he had been sent to Baghdad a few months previously with the rather unusual job of reporting on the economic justification for a certain number of fairly large capital works which had been put in hand in that country in recent years. As one of the items in his Report he had stated that the capital expenditure on the bridge under discussion was entirely justified. The bridge filled a long felt need in the railway system of Iraq. As far back as 1927, it had been recommended as necessary by Brigadier-General F. D. Hammond, C.B.E., D.S.O. As an entirely independent opinion Mr Palmer would state that without doubt a first-class job had been produced, which was not an easy matter in present times.

Mr Palmer was interested in the cost of the steelwork. He had reckoned that it worked out at about £100 per ton, but that was by estimating the weight of the steelwork not in the superstructure. He had then tried to take the calculation a stage further and separate the main bridge from the viaduct, but the result came the wrong way round—the main bridge at £80 a ton and the viaduct £120 a ton. That was difficult to believe.

When in Baghdad he had been told that the bridge had been designed for two lines of railway eventually. It was certainly wide enough for a second railway line alongside the present track. It would be necessary to re-space the stringers, but there would be only 7 feet 6 inches of roadway if the second railway were put alongside.

On the question of steel viaducts, with a total length of 1 mile, the area of steelwork was about 130,000 square feet. It would have to be painted and repainted time and time again for the next 100 years. The maximum load on the foundations was given as about 90 tons, and the increase of load under a reinforced-concrete trestle would have been only 3 tons. It was reasonable to assume that the cost of a reinforced-concrete trestle would have been about half the cost of a steel trestle.

Mr O. A. Kerensky referred to the test loading described on p. 470. That loading represented 44 per cent. of the full design load. Further comment was hardly necessary, and the bridge would appear to be safe from overloading.

In connexion with the provision for stress reversals, continuous-span and cantilever bridges—particularly railway bridges—were severely penalized by the reversal-of-stress clause in the current British Standard specification. The rule that when the reversal of stress occurred half the smaller stress should be added to the larger, that was to say, the working stress should be reduced, was an attempt to deal with the effects of fatigue. That complicated phenomenon, however, could not be dealt with in such a simplified manner, and when the rule was applied to cantilever bridges it resulted in considerable waste of metal. The reversal of stress in a member was damaging only if it occurred a very large number of times. If about 750,000 were taken as a critical number for high-tensile steel, then, in order to affect the strength of the bridge, maximum-stress reversals would have to occur twenty times a day for a period of 100 years. Since maximum stresses were caused by a most unlikely combination and position of exceptionally heavy highway and railway loads, and impact on both, it was unlikely that severe stresses would occur very often and that there could be any fatigue effects on the steel. Therefore, no reduction of working stress was justified. It was to be hoped that the new British Standard now in the course of preparation would make more up-to-date provisions for the effects of fatigue and enable cantilever and continuous-span bridges to be designed more economically.

With regard to pre-stressing, although that appeared to work quite satisfactorily and cause little trouble on the site, it did produce complications in the fabricating shop, since trusses could not be properly shop erected, but had to be checked by a lot of exact measurements. It was to be doubted whether its adoption was really necessary in medium-size bridges of the cantilever type.

The latest American and Canadian specifications and the Code of Practice produced by the Institution permitted a secondary stress of 2 to 3 tons per square inch above the normal allowable working stresses, and if those specifications had been available at the time the Baghdad bridges were designed, it would probably have been found that the unavoidable secondary stresses exceeded the values only by very small amounts, if at all, and no pre-stressing would have been included in the design.

A comprehensive series of tests on deflexions had been carried out, and apart from giving a feeling of natural gratification and pleasant surprise at seeing a structure behave more or less as predicted, the tests should yield a store of valuable information on the dynamic behaviour of the various elements of the structure.

It appeared that the main trusses were subjected to very small impact effects, but very alarming oscillations of rail-bearers had been recorded. Would the Authors comment on that!

Mr H. Q. Golder suggested that much could be learned from the difficulties of carrying out a soil investigation on a job such as the one under discussion. Naturally it was desirable to visit the site, but that was not always possible, particularly when the jobs were at some distance. It involved the cost of shipping plant to distant parts and the cost of flying an engineer out there, so everything possible had to be done without visiting the site. If the soil engineer could not visit the site it was very important to have some intelligent person there; he did not require to be trained in the testing of soils, so long as he was intelligent and honest. Soil sampling tools could be flown out to the site together with instructions for their use. If it were not possible to obtain good undisturbed samples of soils, then it was much more helpful if the man on the site would say so rather than send back something which purported to be an undisturbed sample and was not.

In the case of the Baghdad Railway and Road Bridge disturbed samples of soils were received, but they were reputedly so. It was, fortunately, possible to check the water content on the site so it was known that the soils had not dried out during transit. The information which it was possible to get from that type of sample was obtained by careful visual description and from the liquid limit and mechanical analysis. If the water content were known it was possible to get the relation between that and the liquid limit, which gave the liquidity index, from which in turn it was possible to make a reasonable guess at the strength which the soil would have in the ground and to obtain some estimate of allowable bearing pressure. It was also possible to use some curves which Professor Skempton had published to make estimates of consolidation settlements, but that was not done in the present case. The results showed that a test giving 4·8 tons per square foot on a 1-foot-diameter plate was unsafe, since it gave high stress only in the dry crust; on a larger footing such a high load could not be carried. *Figs 8* showed very clearly that the zone of over-stress under the 1-foot plate loaded at 4·8 tons per square foot was much the same as under a footing 8 feet 6 inches wide when loaded to 0·6 tons per square foot. *Figs 8* also showed the value of that type of quantitative analysis even when nothing was known about soil properties. The arrangement in *Fig. 8* (*c*) was clearly superior to that in *Fig. 8* (*b*). That, he suggested, was the answer to those engineers who maintained that soil mechanics was a highly theoretical science, and who objected to terms such as " elastic isotropic homogeneous semi-infinite solid." Without such terms it would not be possible to carry out the analysis which the Authors had carried out. The defence of the engineer against such terms was to adopt the attitude of Perronet. Dr Hamilton, writing on the life of the eighteenth-century French engineer Perronet, said that his work was broadly based on shrewd intuition backed by a capacity for applying theory when he found it helpful, without being intimidated by it when it conflicted with his judgement! That summed up the attitude of the Authors.

With regard to the piles under the viaduct foundations, Mr Golder's firm had not been asked about them and he wondered what the answer would have been if asked whether or not it would be advisable to drive them, because there were two conflicting possibilities. Piles would reduce the stresses in the soft material immediately under the dry crust and transfer them lower down where it might be imagined that the soil would be mechanically slightly stronger. On the other hand, driving piles in those silts often resulted in considerable disturbance, and there might be a considerable drop in strength after drying. However, some of that strength would be regained with time. There was also the possiblity that the piles might pass through or into a bed of sand. On the whole, piles were probably a good idea and helped considerably in the bridging over of weak spots and increased lateral stability and prevented tilting.

Had the Authors any information relating the driving resistance of the piles to the actual static load which they would carry? In soils of the type dealt with, driving resistance was often not a very good guide except as a relation of one pile to another.

Mr Golder was not presenting an apology for soil mechanics without soils, because, speaking personally, he never allowed a report to be written by a man who had not seen the site, but the technique which the Authors had described could be very useful if it were impossible to visit the site for reasons of expense or time.

Mr Rolt Hammond commented on the very practical manner in which the information contained in the Paper had been presented.

He was particularly interested in the application of welding to bridges, and he wondered whether any comparison had been made between the cost of riveting and welding. In present times the welding of high-tensile steel did not present the difficulty that it had done in the past. A great deal had been learnt during the recent war on the welding of homogeneous hard armour and the evolution of suitable electrodes.

On p. 463 the Authors had stated that on any future occasion they "would not hesitate to weld the caissons completely, now that welding technique has developed and that Iraqi workmen are rapidly learning this technique and are capable of turning out first-class work." That comment was confirmation of Mr Hammond's view that welding was going to lead to a new age of craftsmanship. It was fascinating work and there was much yet to learn about it.

Had consideration been given to the automatic welding of large components under factory conditions? Mr Hammond had recently seen crane-supporting stanchions, nearly 70 feet in height, being automatically welded for the whole length, and they were certainly very fine jobs. He was referring in particular to the processes known as "Fusarc" and "Unionmelt". The latter was the submerged-arc type of welding system which gave a fine joint and had been thoroughly proved in welded ship construction during the war.

The Authors had referred to the necessity for using correct welding procedure. It was always necessary when welding components to lay down the runs in such a manner that distortion was reduced to the minimum, that was to say, balanced out as far as possible. That was quite simple to check. Very often with large members it was necessary to put on a certain amount of camber before welding in order to allow for the distortion which would take place.

Mr Alastair Storrar said that the supply of ballast and sand from a source between 80 and 90 miles from Baghdad was a constant source of worry, particularly in low water where, in places, no more than 3 feet of water existed up the river. A quantity of the order of 60,000 tons had been washed and brought down; it had been transported by contractors and hired transport in a little over 2 years. Great credit was due to the foreman in charge of the plant. He was always willing to produce not only that little extra but sometimes the large extra which was being demanded of him.

Pre-stressing of the anchor-span members had been mentioned, but only those members between panel points 6 and 10 and between 14 and 18 (the two adjacent panels to the piers) were appreciably affected, and, in practice the pre-stressing was easily carried out once the technique (see *Fig. 31*) was fully developed and the tricks were known.

After the first drift had been inserted it was found that very slight adjustments in the tension of the straining wires enabled other holes to be lined up and drifts and bolts inserted where necessary. Drifts had been gauged before being issued to erectors, and any which were more than five-thousandths of an inch undersize were rejected. No elongation or deformation of the rivet holes had occurred.

Referring to the pump sections which were shown in *Figs 5*, Mr Storrar said that it had been found, on construction of the buttresses, which had had to be shuttered in one lift with consequent disturbance to the normal panel steel shuttering, that it was a very slow and laborious business, since working space was very limited in the cofferdam. Eventually the pump sections had been redesigned on the site, using the existing piping. Incidentally, the lower strainers which were fixed below low-water level were one of the last items to arrive on the site.

The Authors, in reply said that Mr Gwyther had referred to the use of piles, with particular reference to cement. On the general question of using cement in Iraq, without dwelling at all on the chemical constituents, experience had shown that the slower the setting time of the cement the better were the prospects of getting good concrete. It was almost true to say that all concrete cracked in the Middle East, and similarly all brickwork cracked, but with slow-setting cement the cracking seemed to be very much less than the cracking which occurred with rapid-hardening cement. That was the reason for venturing on the generalization in the Paper—that there was no future for rapid-hardening cement in Iraq. Cement which was rejected from the mills in Great Britain on account of its long setting-time but which was otherwise suitable, could be used with advantage in Iraq.

Mr Reid said that Mr Storey Wilson had sent cold shivers down his spine when he talked about his beautiful excavation as "muck." That was rather a terrible word, and Mr Storey Wilson might have used a noble expressive Scottish word, namely "glaur," instead. The word glaur had the advantage of meaning exactly what it seemed to mean.

Mr Sully noted that Mr Storey Wilson regretted that more had not

Fig. 45

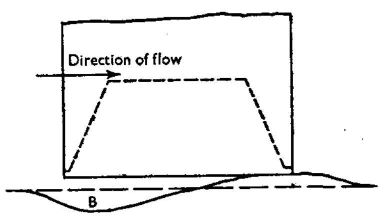
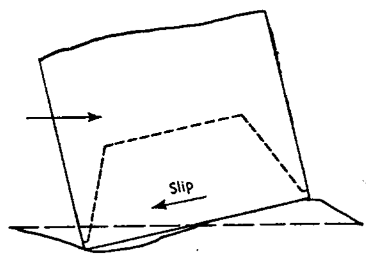


Fig. 46



been made of the caisson foundation in the Paper. Mr Storey Wilson and both of the Authors had already very nearly flogged that horse to death.

With regard to the degree of accuracy in the sinking of the caisson, using floating craft, a caisson could be pitched and held to within 2 or 3 inches of position in most cases. In one case, however, rather an odd accident had happened. Mr Reid had suggested to Mr Storrar that in the case of caisson No. 4 it might be wise to pitch the caisson about 4 inches downstream. The current had been running at about 3 to 4 feet per second, and as the caisson had approached the bottom, within a foot of touching, scour had taken place at B in the *Fig. 45* with a build-up at the back of the caisson. As the caisson had been lowered further it had tended to move forward against the stream, and finally had landed in some position such as that indicated in *Fig. 46*. When the men had entered and had begun to cut away it had slid forward a little more. Unfortunately, the young engineer whose duty it had been to pitch the caisson, had got mixed in his plus and minus signs and had pitched it 4 inches upstream instead of downstream. That caisson, therefore, had actually moved altogether about 9 inches out of position, necessitating careful plumbing in order to get the equipment installed in the caisson.

With regard to Mr Palmer's comments on steelwork costs, the figures quoted on p. 475 were all-in costs, from which it was impossible to abstract costs of individual items.

Referring to Mr Palmer's remarks on the viaduct design, the Authors concurred that piers in brick or concrete would have given a very pleasing effect, but steel had been chosen for lightness. In passing, however, it might be said that the climate of Iraq was very kind to steelwork, the amount of corrosion taking place over, say, 10 years being hardly measurable, and nothing like the protection required in Great Britain was necessary. The same could not be said for the Persian Gulf area. On the Baghdad road bridges, originally painted 13 or 14 years ago, there was no evidence of serious corrosion or pitting anywhere, and on repainting it was not found necessary to give the underside of the bridge-work a second coat of paint. It could, therefore, be said that the maintenance cost over 100 years would not be heavy.

Tenders had been invited for the work in accordance with the law of Iraq.

In reply to Mr. Palmer, the bridge would indeed carry two rail tracks, but not the simultaneous roadway loading on the remaining roadway width mentioned by Mr Palmer.

Turning to Mr Rolt Hammond's remarks in connexion with welding, there were no accurate records of the comparative costs of welding and riveting, but in a general way it could be said that welding was definitely cheaper. Welding for the caisson plating had been adopted chiefly on account of cleanliness rather than cost, although the cost had in fact been less than the cost of riveting. No attempt had been made to weld any of the superstructure steelwork. Iraqi workmen were learning slowly, but were not considered good enough to tackle really important load-carrying steel bridge-structures yet. In any case, the Authors were not in favour of site-welded joints in truss structures of the type under consideration. They were strongly in favour of shop-welded details wherever advantageous, but site joints of trusses should be riveted or should have pinned connexions.

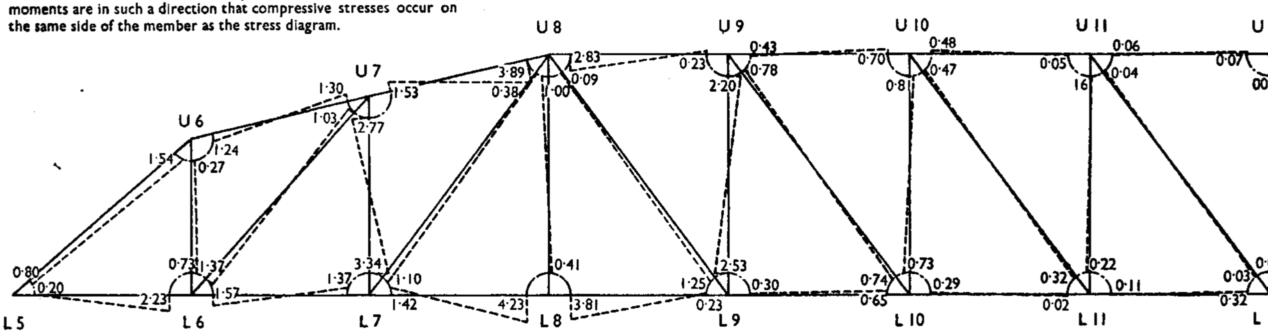
Mr Gwyther and Mr Kerensky had asked whether pre-stressing was worth while. In the Author's opinion that was debatable, but it was rather interesting to see that it was possible to get, despite small changes in dimensional and angular limits, practically 70 per cent. of the result aimed at, which was quite good.

The small impact factor as indicated by the deflexion curves was partly explained by the fact that American locomotives had been used for the tests; they were very well balanced for two-cylinder engines.

The Authors referred to the remarks of Mr Gwyther and Mr Kerensky on the large deflexion of the stringer indicated on *Fig. 44*. It should have

Fig. 47

Figures indicate stress intensities in tons per square inch at a distance of 3 feet from intersection points of members. The bending moments are in such a direction that compressive stresses occur on the same side of the member as the stress diagram.



Magnitude of Secondary Stresses which would be set up in a Non-Pre-stressed Bridge by the Loading Adopted for the Pre-stressing Technique been noted in the Paper that those were reproduced from the celluloid films, and for the reasons stated in connexion with the truss deflexion the rapid vibrations were superimposed on the normal deflexion and exaggerated at ten times the vertical scale. Similar remarks applied to the cross-girder diagram in *Fig.* 43. In the latter diagram, the end-fixing effect of the main bearing detail was shown since the measured deflexions were only about one-half of the deflexions calculated with simply supported ends. That fixity would not be realized to the same extent for the other cross-girders away from the piers.

The Authors agreed that for the main trusses a comparison between the mean dynamic deflexion and the measured crawl deflexion indicated a low impact factor, but that did not take into effect the very rapid oscillation which took place over only one or two panels as the load passed. Although the resulting reversible deflexion from that cause was small it was set up very locally and undoubtedly imposed considerable direct and bending stresses, in the adjacent members which had to be taken as fixed-ended. For that reason, although agreeing with Mr Kerensky on the probable wastage of material on large bridges of the type he had mentioned, with relatively massive members, when the stress reversal clause in BS 153 was adopted the Authors would hesitate to disregard it entirely for medium-span cantilever bridges with open-type track as used in that bridge.

Referring to the desirability of pre-stressing, mentioned by Mr Kerensky, *Fig. 47* indicated the magnitude of secondary stresses which would be set up on the non-pre-stressed bridge for the loading adopted for the pre-stressing technique. It would be noted that the permissible increase of allowable working stresses of 2 to 3 tons per square inch was very considerably exceeded in that case.

The Authors felt that pre-stressing was desirable and offered very little difficulty, provided that the first-class workmanship expected for bridge structures was put into the fabrication and that proper erection technique adopted. The attitude to the whole question in the past seemed to have been : " let us avoid the excessively laborious secondary-stress calculations (by the older methods) by pre-stressing which can be readily calculated and put the extra work on to the fabricating shops." The present attitude seemed to be : " having improved the mathematical approach by relaxation methods, why worry to pre-stress? Why not avoid the troublesome shop work, which made very little difference in any case ? " If that attitude were adopted, any fabrication errors which occurred in a contrary direction would increase secondary stresses, whereas with pre-stressing the effort was always in the right direction to reduce those stresses particularly where they were most important.

With regard to Mr Palmer's remarks about the cost of steelwork, the average price for the superstructure steelwork was about £85 per ton, and for the railway and roadway viaducts respectively, £80 and £70 per ton approximately.