7151 DISCUSSION

New Scotswood Bridge

D. W. SMITH

Mr T. J. Upstone, Chief Engineer, Redpath Dorman Long

The design of an arch bridge is always interesting and presents a good deal of scope. In the present case interest has been increased by departing from the normal design of tied arches. Co-operation between steel stiffening girders and the concrete deck has been introduced and the prestressing of the steel stiffening girder. Furthermore, prestressing strands have been used in the concrete deck. These devices are aimed at obtaining the fullest co-operation between the steel and concrete deck and at producing an economic design.

92. The interaction required between steel and concrete complicated the erection scheme as a considerable amount of deck concrete had to be placed at various stages of the steel erection. The profile of the structure was adjusted from stage to stage by jacking on temporary trestles. It is doubtful whether the inclusion of the concrete deck was really economic particularly in view of the very low working stresses quoted for the top flange of the stiffening girder. The maximum stress given is 4·57 ton/sq. in. under HA load compared with a permissible stress, also quoted in the Paper, of 13·2 ton/sq. in. Could not this flange have been reduced from 1¼ in. at least to the 1 in. thickness of the lower flange?

93. It is noted that the calculations were made by hand. This labour could have been saved by using a computer. Considerable benefit would have accrued as the complicated erection procedures could also have been worked out using the same computer program. The Appendix gives the well-known method for solving the singly redundant tied arch. In this the arch is treated as pinned at each connexion and subjected only to axial loads. The table of stresses, quoted in the Paper, shows bending stresses in the arch rib and it would be interesting to learn on what basis the arch rib moments were calculated. A computer program would have calculated these arch moments in the normal course of its work.

94. With regard to erection, the Contractor's first ideas were to float in the stiffening girders in one piece. It was only the difficulty of navigation up the River Tyne from a suitable building yard carrying the main stiffening girder 332 ft long which ruled out this method. The Contractor then turned to his second choice which was to launch the girders as had been described in the Paper.

95. The Author has introduced two stiff hangers at the ends of the arch and these are in a position where they will receive the greatest secondary stress. Rigid hangers could have been used for all the others without any serious secondary bending stress being introduced into the arch or the hangers. The use of stiff hangers would have made the arch erection easier and avoided the difficulties mentioned in the Paper of adjusting the wire rope hangers.

96. The unfortunate accident to the upstream stiffening girders only delayed the arch erection for five working days and was not responsible for the 'rather protracted erection' in § 71.

Mr J. A. Williams, Consulting Engineer, Sandford Fawcett, Wilton & Bell

I would appreciate some further information about the foundation cofferdams because we are currently designing a similar bridge foundation of roughly the same area as one of the main Scotswood piers, and with approximately the same number of H piles, all for similar underwater construction. We are thinking in terms of a plug about

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8 ft or 9 ft thick, and comparing this with Fig. 13 it would seem that, allowing for a tide somewhat above mean high water, there was a total uplift on the Scotswood plug of about 2000 tons. The 5 ft plug gave only 600 tons counterweight, and I assume that the balance was allowed to be taken by tension in the H piles, ignoring any help from the sheet piling. This means that with something in hand for a factor of safety, the Author was relying on some 30 tons anchorage from each H pile in the temporary condition, when the plug had just been cast and the cofferdam dewatered. Was the 5 ft plug designed in that way? Secondly, was it shown in detail on the contract drawings or was something different shown and the final design evolved with the Contractor after the contract was awarded?

Professor A. L. L. Baker

What would the Author consider to be the worst distribution of live load on the deck in relation to stresses in the hangers and the tie, and what sort of deflexion in the tie would there be for that position? I imagine that with the deck fully loaded the stress would mainly be increased in the arch because that is the right distribution of live load for the shape of the arch. With some sort of partial live load, say, concentrated over the middle half or concentrated in about one-third on one side, there would be considerable bending in the tie and considerable deflexion.

99. It is very interesting to note that the Dutch were using these designs some 15 or 20 years ago in building this type of bridge, and drivers of trains used to experience the wave in the deck that went ahead of the train, but the bridges worked out very satisfactorily, and I am sure this design will.

Mr D. W. Manton, Chief Bridge Designer, Redpath Dorman Long

Could Mr Smith say whether he would use the composite action of the concrete with the stiffening girder in a future job? Considering the complications it caused in erection I am not sure that the benefit is really worth the trouble.

101. There also appears to be a braking girder at each end of the bridge, which would seem to be superfluous when there is a solid concrete deck firmly bonded to both stiffening girders. These things do involve a large number of small members which complicate both fabrication and erection. Did Mr Smith have any ideas at any stage of using a steel deck which would to a large extent get rid of the complications due to shrink and creep?

102. The Paper mentions the rope hangers as reducing secondary stresses but they always appear to be too small and too many. Surely two ropes, in each group, of a larger diameter would have been much simpler to handle than the four as used?

103. The adjustment of the ropes is mentioned as having caused some difficulty. Physically this was quite a difficult job. We designed jacking gear to go inside the stiffening girder to strain four ropes at a time. It seemed to be the only way to get the gear in the space available. The erectors had to move these 2–3 cwt pieces inside the stiffening girder from one suspender point to the next. This is inevitable when dealing with fairly heavy loads but another form of suspender might have simplified the gear. The arch rib was prestressed relative to the stiffening girder, in that in its unstressed condition the arch rib would not fit to the stiffening girder. It was not possible for anyone to check the length between the suspender point on the arch rib and the corresponding point on the stiffening girder accurately. I am sure that this had some bearing on the difficulties of adjustment of suspenders.

Mr L. G. Deuce, Ministry of Transport

I was interested to see that the whole of the concrete deck was taken as acting compositely with the stiffening girders. I would have thought a lesser amount of deck would have been appropriate in some stages of the calculations.

105. The stressing of the stiffening girders is a most interesting solution. I would

like to ask if any difficulties were experienced with friction at the comparatively simple saddles supporting the strands as there seems to be quite a camber in the stiffening girders and the simple saddles might have been responsible for quite high friction forces.

106. As fatigue has played quite an important part in the design, judging by the details adopted, I was interested to compare the fatigue spectra in Appendix 1 with those now contained in the Ministry's recently published Technical Memorandum BE 16.[8] It is not easy to make a direct comparison but as far as the HA loading is concerned Mr Smith's spectrum is probably a little less severe than BE 16, whilst his HB spectrum appears to be much more severe. Overall there probably is not much difference between the two although if anything BE 16 is probably the less onerous.

Mr R. F. Bell, Resident Engineer, Mott, Hay & Anderson

There has been quite a lot of criticism in the Newcastle area concerning the appearance of the bridge. In particular it is suggested that the stiffening girders look very much too deep. Mr Smith has given us considerable details of how the depth was related to the method of erection. I would like to point out rather more definitely than stated in the Paper how valuable they are in hiding the clutter of pipes and ducts running across the bridge underneath the deck. Some of these, the 36 in. water main and the 30 in. sewer, will take up almost as much depth as the girders themselves but are quite hidden, and the clean soffit line is preserved.

108. In § 7 Mr Smith describes the ingenious road layout on the north side as incorporating a low-level roundabout. In fact, I think his road layout is so ingenious that he has slightly misled himself. It is not a roundabout at all but a very short length of dual carriageway. It does not carry traffic around its western quadrant apart from the occasional buses turning on the termination of their route at that point.

109. In § 41 Mr Smith refers to cracking in the piers as being due to uneven settlement of the piers under load. In my opinion this may be the cause, though I feel that there is no really conclusive evidence to support it, and I would suggest that it is possibly a combination of this and pure shrinkage of the side walls of the piers. The solid blocks at the end of the piers are very rigid and can resist inward pull due to shrinkage stresses, and the result is cracking of the 12 in. side walls.

110. Finally, in Appendix 1 Mr Smith states that the bridge has a design life of 150 years. In my experience it is not often that this figure is quoted for a structure. How and why was this chosen? Was it selected by the designers? Was it selected by the clients?

Professor L. F. Stephens, University College, Dublin

The calculations for the main arch span were made by adopting the simplifying assumptions listed in Appendix 2, which reduce the number of redundancies in the structure to 1, taken as the horizontal tie force.

112. Using a general computer program for the analysis of a plane frame it is not now necessary to make such a series of simplifying assumptions. Using such a program some connexions can be assumed to be pinned, and some rigid, as required; axial shortening of the rib, hanger extensions and changes of arch shape may be taken into account. This means that long and tedious separate calculations of secondary effects, as in the paper by Chandrangsu and Sparkes,[7] are no longer necessary. The computer program does not require that analysis be artificially divided into primary and secondary stages.

113. Using such a program the bowstring arch investigated by Chandrangsu and Sparkes has been analysed, and influence lines for all required moments and direct forces have been obtained. In Fig. 21 the values of the moments in the rib and stiffening girder for one load case are compared with the values found by Chandrangsu and Sparkes.

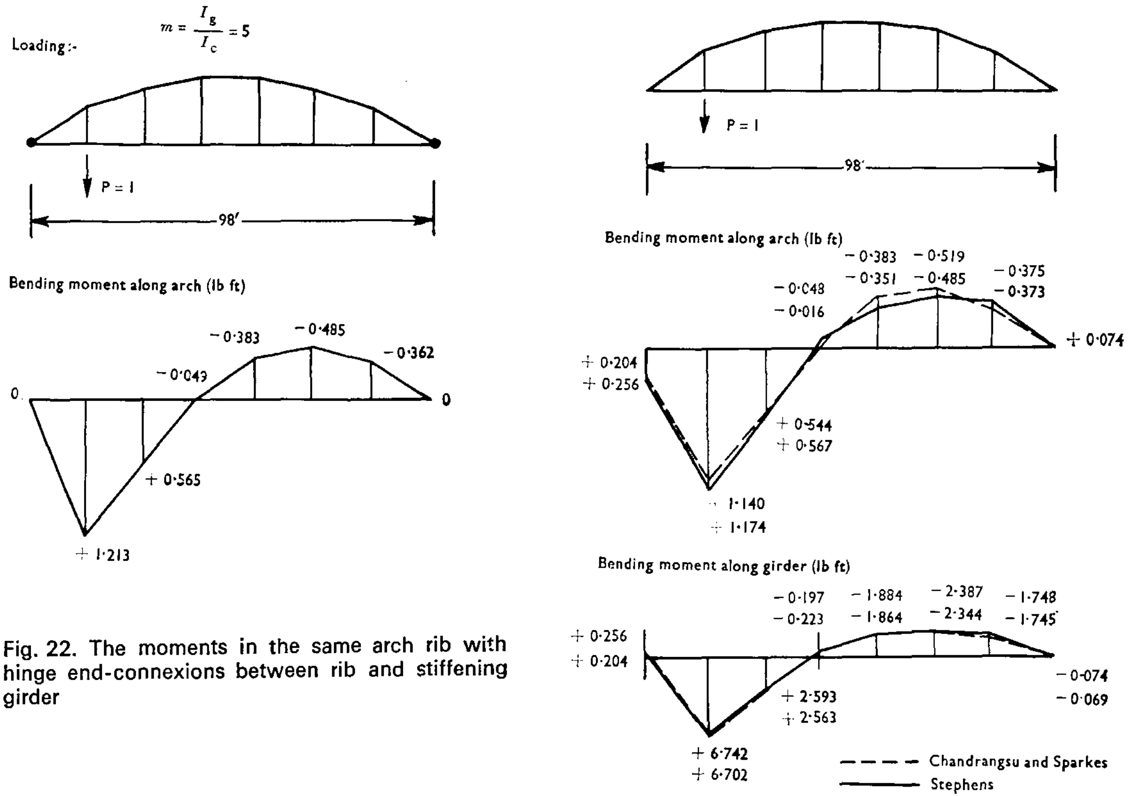

Fig. 22. The moments in the same arch rib with hinge end-connexions between rib and stiffening girder

114. The effect of hinged end-connexions between the rib and stiffening girder, as used in Scotswood Bridge, may be easily taken into account, and the moments in the same arch rib with this type of connexion are shown in Fig. 22. The effect of different ratios of rib to girder stiffness may also be easily investigated.

115. Many steel tied arch designs have incorporated an eccentric connexion at the junction of the rib and stiffening girder, the intersection of the arch rib axis with a vertical line through the support being placed some distance above the centre-line of the stiffening girder. Such a detail was adopted in the case of the navigation spans of Storstrøm Bridge in Denmark[9] the purpose being 'to assist in equalizing the stresses on the girder flanges'. Was an investigation of a connexion of this type made in the case of Scotswood Bridge?

Mr T. M. Young, Deputy Burgh Surveyor, Airdrie (formerly Resident Engineer, New Scotswood Bridge)

As Resident Engineer from the inception of the scheme until 3 months after the initial bridge opening, I experienced all the site problems pertaining to the bridge and its approaches in difficult ground conditions.

117. I formed the opinion that the bridge design was a logical follow-up to the excellent bridges which cross the Tyne and Wear, and which were built in the 1930s. This modern arch design follows the same basic shape of its predecessors, while incorporating new materials and ideas to achieve a lighter and more streamlined effect.

118. The following points which came to light during site construction might be worthy of mention. The main box girders could have been made a little heavier to increase torsional stiffness, and alleviate problems of the lining up and fitting in of arch rib end joints. This would also have helped to prevent the buckling of the girder which took place over the South West temporary river trestle. I should add to Mr Smith's explanation for this occurrence that the necessary unsymmetrical placing of the temporary trestles seemed to have a lot to do with the problem. At the other three temporary river trestles support points, the main bridge girders had internal web stiffeners near to the support, while at the South West trestle, the equally spaced stiffeners were virtually equidistant around the point of support. While the on-shore piers above the foundations were of hollow box reinforced concrete construction, it might have been advantageous to fill the small hollow entries of these units with dry light fill (fly-ash or the like). This would have eliminated the need for the manhole openings in the heavy reinforced concrete tops of the piers. It was noted that any hair cracks which appeared on the pier walls, started at the edge of a top slab manhole opening. I should also mention that on the immediate bridge approaches, the composite deck construction caused a few site problems, in respect of the differential deflexions during concrete pours. The rather intricate and slow sequence of concrete pours, which was necessary to minimize constructional stresses in the composite deck structure, would seem to cancel out some of the advantages of this type of design. However, in overcoming this problem, as in all other site difficulties, which arose from time to time, my staff and I had excellent co-operation from Mr R. J. Clements, BA, MICE, the Contractors' agent, and his staff.

119. The ready co-operation received from representatives of the local authorities and the statutory undertakers did much to simplify the phasing-in of the project. This allowed traffic to move at all times through the area on both banks, as well as across the river at this site by means of the Old Scotswood Bridge.

Mr A. E. Temple, Engineer, F. R. Bullen & Partners

I should like to ask whether in the course of time that has elapsed since this type of bridge was first decided upon you would consider putting in a prestressed concrete tied arch. I know the loads involved would be very much greater so that the difficulties with the foundations would be increased, but in spite of this, the cost might well be less under present-day conditions. There could also be eliminated some of the difficulties which other people seem to envisage of prestressing a reinforced concrete deck resting on a steel structure. Had you not prestressed the deck would you have been involved in all sorts of contraptions such as joints in various places to overcome the inclination of the deck to act with the steel structure unless you used such joints? As the protection of steel is now so much more easily and economically done, would you consider putting in a steel deck instead of reinforced concrete, assuming the bridge were otherwise a steel structure?

Mr R. E. Landau, Project Engineer, R. Travers Morgan & Partners

Concerning Fig. 7 showing the grip bolted connexion in the arch rib, presumably a hand hole was necessary for access. What system was used for applying torque to the grip bolts? Also, what protective treatment was adopted where the hanger rod passes through the rib? The same question applies at the lower end of the rods where they enter the stiffening girder.

122. Mr Smith states in § 23 that the use of prestressing saved 40 tons of steel. If this is priced at £75 per ton (mainly cost of material) the saving is £3000. On the other hand there was the additional cost of prestressing strands and anchorages plus the actual prestressing—what was the cost of these? However, the reduction in plate thickness may have been worthwhile in itself. Might this have been achieved alternatively by adopting a deeper stiffening girder, rising above footpath level?

123. From §§ 20 and 27 it is apparent that the stress level at the holes in cross girder webs was a source of some concern, and the close spacing of cross girders is explained by reference to these holes. The thickest web plate appears to be $\frac{5}{8}$ in. (Fig. 11); was a locally thicker plate considered? Does the load factor against web yield quoted in § 27 refer to initial yield? (If it were ultimate there could be a fatigue problem.)

124. The sliding joint of stringers over cross girders is described in § 26. Could the Author further describe the function of this joint in conjunction with a continuous deck?

125. In § 27 concerning peening, what was the cost of peening to the contractor? What inspection technique is necessary to ensure that the necessary standard of peening is achieved?

126. Were the tests of roller bearings on epoxy pitch (§ 75) 'static', or carried out with longitudinal movement under load?

127. Is the due proportion of General Items included in the unit costs given in § 80? What were the costs per ton of steel for (a) fabrication and delivery, (b) erection, (c) protective treatment?

Mr C. A. Miller

In § 21 and Fig. 7 the Author indicates that the box section arch ribs were connected by bolts. I would be grateful if he could indicate the method by which the designers intended these bolts to be fixed, and whether the contractors had any difficulty in making these connexions.

129. The Author does not mention maintenance. The arch ribs and bracings, the hanger top fittings, and the steelwork below the concrete deck are all areas where routine inspection and painting will be expensive by reason of the difficulty of access. Perhaps the Author could describe briefly any provision for inspection and maintenance and, beneath the bridge deck, the installation, alteration and repair to the various suspended services.

D. W. Smith

Professor Baker asked about the worst distribution of live load. There are some influence lines in Fig. 21 from which it will be seen that the worst bending moments at the quarter points take place in the sagging direction when slightly less than half the bridge is loaded, and a slightly smaller maximum in the other direction (hogging) when a little more than half the bridge is loaded.

131. We did gain some differential prestressing by the fact of erecting the stiffening girder over the temporary supports at about the quarter points as maximum bending moments occurred at these points. The stiffening girders were erected over the temporary supports and held down at the main piers, and the bending thus induced was not released until after the concrete deck was hard. By this means the prestressing in the concrete deck was maximized at the point of maximum bending moment.

132. The maximum stiffening girder bending moment occurs at the quarter point when 142 ft of the span is loaded from one end. The maximum deflexion is assumed to be sufficiently closely approximated by the mid-span deflexion, which is as follows for the widened dual three lane bridge:—

(*a*) full span loaded	HA	1·31 in.
	HB	2·33 in.
(*b*) 142 ft loaded from one end	HA	0·37 in.
	HB	0·59 in.
(*c*) 130 ft loaded in centre (case for maximum mid-span deflexion)	HA	1·85 in.
	HB	2·62 in.

133. An actual heavy load on the dual two lane bridge in January 1969 caused mid-span deflexions as follows:—

Load at quarter point	0·36 in. (calculated 0·72 in.)
Load at mid-span	1·32 in. (calculated 1·44 in.)

134. Contrary to what I implied in the discussion, the maximum hanger load occurs when the horizontal thrust in the arch is a maximum, since, apart from secondary effects due to changes in shape of the arch, there is a simple triangle of forces at the top of each hanger. These maxima occur with 0·7 span loaded centrally, though with the whole span loaded at the reduced intensity of BS loading applicable to this longer loaded length, the total hanger load is only about 1½% less than the maximum; (6% less based on the live load part only).

135. With regard to **Mr Upstone's** remarks on the interaction between the concrete and the steel causing complication of erection, as **Mr Temple** pointed out, if the reinforced concrete deck is not connected to the steelwork it has to be separate, and this means allowing it to move and to have a lot of joints. The provision for this movement and joints would in my view have been an even greater complication, but the fact of connecting them together did have the advantage of a differential prestressing which suited the form of the bending moment diagram.

136. It is quite true that the calculations should have been done on a computer. This is the one decision which I wholeheartedly regret. If the bridge had been designed one year later it would most certainly have been designed on a computer. We did wonder whether to do so, and the decision was 51–49, and it was the wrong way. We did it by hand. As Mr Upstone implied, it is quite easy to calculate this once by hand, but there are a lot of intermediate conditions during erection, and allowance for creep and shrinkage, so that the calculations have to be repeated a number of times, and this repetition demands the use of a computer.

137. Mr Upstone asked about the basis of bending stresses in the arch rib. We simply calculated the change in curvature from the total stiffness, and from that curvature one gets the stress in the arch rib. The inaccuracy in doing this is extremely slight in view of the fact that the arch rib is many times more flexible than the stiffening girder.

138. It seems that a saving of about £7 10*s*/ft could have been achieved by reducing the stiffening girder top flanges to ¾ in. thickness, and the reduction in weight etc.,

by adopting a steel deck would have saved a further £12/ft in main structure steelwork and in reducing the number of foundation piles. As against this, a steel deck would have cost about £62/ft more than the reinforced concrete deck actually adopted. If these figures are correct, a reinforced concrete deck is the right choice for this span.

139. Mr Upstone said that stiff hangers would have been more economical, and a point was also made by **Mr Manton** that if we were to have ropes we should only have two and not four. However, it appeared to me that he was rather contradicting himself, for after asking for a smaller number of ropes, and therefore larger ones, he said it was difficult to hump around the gear inside, which would have been heavier gear if they had been larger ropes. In point of fact, the stiffening girders are seven feet high and 3 ft 6 in. wide, so that even allowing for some restriction of the width by prestressing strands, the humping round of gear inside should not have been as difficult as all that.

140. The evidence we had that secondary stresses are severe in a tied arch with stiff hangers was twofold. First of all, the failures on a number of such bridges in Belgium between the wars, where a considerable amount of research has been done to find a nice smooth flow of stress round the joints, but it seemed to me that it would be much better to cut through this difficulty altogether by having flexibility at the joints. (See § 24 of the Paper.)

141. The other evidence arose from an opportunity we had a few years ago of testing full scale 53 ft 4 in. span tied arches very much like this. We laid two of them down flat and jacked them towards one another, and measured the direct and secondary bending stresses. The direct stresses agreed pretty well with the calculations. The secondary stresses bore no relation to the calculations, and some were very large. It was these two pieces of experience that led to the conclusion that we should try to keep the secondary stresses down by making the hangers flexible. The end hangers are indeed rigid, as seen in the cross section of the bridge, but are only 9 in. wide as seen in the elevation. Therefore there may be fairly appreciable secondary stresses in the hangers themselves, but even these short end ones are far too flexible to transmit any appreciable bending stresses into the much stiffer sections of the arch rib and the stiffening girder.

142. Mr Upstone referred to an error in § 71. I am glad to be put right. I thought that the erection had been slow as a result of a slight accident which occurred, but apparently only five days were lost.

143. **Mr Deuce** mentioned friction at the saddles supporting the prestressing strands. I do not think it was at all serious. The amount of kink at any one saddle is very small indeed. The tension in a strand is 58 tons, and the normal reaction on a saddle is approximately 1 ton. There are only a relatively small number of them over the length of the bridge, and the amount by which the strands pulled out agreed fairly well with what it should have been, assuming the tension to be constant over the full length. The saddles were carefully smoothed and rounded so as to minimize this difficulty.

144. The whole width of concrete deck was taken as acting compositely with the stiffening girder, and in defence of this it can be said that:

(*a*) the stiffening girder flanges are constant throughout, and reach their maximum stress only about the quarter point of the span, therefore a shear lag extending, say 80 ft into the span (over a slab half-width of 36 ft 6 in.), can be accepted;

(*b*) the slab is well stiffened by cross girders and stringers;

(*c*) a heavy concentration of shear connectors is provided in the end bay, on the stringer 7 ft 8 in. from the stiffening girder, with a substantial steel diagonal connecting this stringer to the stiffening girder as shown in Fig. 12;

(*d*) the adverse effect of shrinkage is fully allowed for, as well as the beneficial effect of the increased resistance to bending resulting from the concrete slab;

(*e*) strain measurements on a stiffening girder of the completed bridge when a heavy load passed over were slightly less than had been calculated—§ 86 (*c*).

145. I was very interested to learn that the fatigue spectrum given on page 244 does not disagree too badly with the more up to date information now available.

146. The girder which Mr Manton referred to as a braking girder at the end is not a braking girder at all. It was a system of bracing to make it easy for the Contractor to square up the bridge. The arch rib, as he pointed out, was made under no load slightly longer than the stiffening girder, so that under dead load they would agree, and this again is a means of reducing secondary bending stresses. I do not really agree that it makes any difficulty in getting the things to fit in trial erection. If the Contractor had been willing to do this trial erection, which he was not, we should have been most delighted. All that would have been necessary, on a distance of 329 ft, would have been to put a steel rule on them and measure the $\frac{5}{8}$ in. or so of difference, and I do not think that would have been very difficult.

147. Replying to **Professor Stephens**, the benefits from computer calculation, if it had been adopted for this bridge, would have arisen mainly from the need to repeat the analysis several times during the evolution of the design and erection schemes, and from saving in tedious calculations of creep, shrinkage and fatigue. The errors due to known simplifying assumptions were, in this instance, easily shown to be too small to demand accurate assessment; indeed, they were apparently cancelled out (and slightly more, in the safe direction) by what may perhaps be described as 'unidentified simplifying assumptions'—§ 86 (*c*).

148. When a moment is applied to the ends of a girder continuous over many spans, it decays very rapidly as it passes along the girder—more or less by a factor of 4 as it passes every interior support. The Scotswood Bridge stiffening girder of course behaves in a more complex manner than as if resting on unyielding supports at hanger positions; but it was judged without investigation that nothing could be gained by an eccentricity in the connexion of the arch rib to the girder, except a slight simplification of detail geometry.

149. **Mr Landau** referred to the tests on the bearings and asked whether these took account of the longitudinal movement. The bearings were pure roller bearings and the test was a pure pressure test.

150. **Mr Williams** asked about the thickness of the concrete plug in the pier—a very important matter. We showed on the tender drawings 5 ft at the landward side and thickening up to 6 ft 6 in. at the riverward side, and some reliance was placed on tension in the piles additional to that. I believe that the Contractor on his own initiative was a bit generous in all these dimensions so as to make quite sure, though no substantial increase on the thickness shown on the drawings was used, and there were steeply raked piles in both directions, so that they had a capacity to hold down the concrete. At any rate there was no trouble when the cofferdams were dewatered.

151. I was very grateful for Mr Temple's remarks. If the foundations had not had to be constructed on expensive long piles there could well have been a case for a prestressed concrete tied arch, but the weight would have increased the foundation costs very considerably and the economics were definitely against this choice. I think the choice was right on this point.

152. One or two speakers mentioned a steel deck, which would have got over all the difficulties of creep and shrinkage, and would have been a very good idea altogether. It would have cost more, however. What we really need, as I was mentioning to Professor Baker earlier, is a concrete that will not shrink, or rather a concrete that would expand just sufficiently to cancel out the creep.

153. **Mr Bell** mentioned criticisms of the appearance. This bridge has been designed for future widening by cantilevering out a footpath, and the flat face of 7 ft

of stiffening girder will not show when that has been done. It is not easy to make a bridge look quite perfect both before and after widening.

154. The designers chose the design life of 150 years. I do not know whether any really scientific ground can be given for it. I am told that 120 years has received quite strong and official recognition subsequently, so no doubt future bridges will be based on that.

References

8. Provisional Fatigue Requirements for Steel Bridges. *Ministry of Transport publication*. Jan., 1969.
9. Maunsell G. A. and Pain J. F. The Storstrøm Bridge. *Proc. Instn civ. Engrs*, 1939, **11** (April) 391–448.

Acknowledgements

Mr H. C. R. Smith also played a considerable part in the design of the centre span of the main bridge. A further acknowledgement is also due to Mr I. A. Stewart, BSc, MICE, who was the agent for part of the contract.

Corrigendum

In Fig. 6 the number of strands in the box in the bottom left-hand corner is shown as 29. It is correctly referred to in the text as 28.