Paper No. 6290

The Volta Bridge†

by

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and

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Discussion

Sir Hubert Walker referred to the statement in § 14 that the completion date had been February 1956, but work had not started until January 1955 and it seemed to him that this must have been a very optimistic estimate indeed. From his own experience of getting work done in West Africa the 2 years in which the job had actually been done were in fact extremely creditable.

153. He asked if the Authors could give any more details as to the method of calculating the catastrophic flood level. From his own experience the determination of flood levels was one of the most difficult problems in the design of bridges in West Africa, because of the lack of reliable information. For some time before the war, in Northern Nigeria, where the country was somewhat similar to the Volta Valley, they had found that 70% of the Dunn table was a very good allowance for a waterway, where nothing more was available, and in fact Mr J. B. R. Pedder had done some investigations which had been published as a P.W.D. Technical Paper in 1945 and this had confirmed that for most of that area 70% of the Dunn table was about right. It would be interesting to know how the waterway provided at Volta Bridge compared with the Dunn table figures.

154. It would also be interesting to know whether the recent catastrophic floods in Rhodesia had given any cause for thought. Mr Pedder, in his Paper, had made some comparison of the rainfall/area/duration relations between Northern Nigeria and Rhodesia, and had come to the conclusion that in Northern Nigeria rather more than 65% of the Rhodesian rainfall could be expected.

155. Usually, when faced with a complete lack of catchment data, a very fair indication of maximum flood level was the floor of the ferryman's house. The ferryman and his family would have been working there for possibly hundreds of years, and they would have built their hut *just* above the flood level, but not much.

156. On what information had the wind speed of 80 m.p.h. been adopted as a basis for design? Most of the wind records in that part of the world were obtained from aerodromes, but it was always thought that the large river valleys were much more susceptible to gusts than the more open sites where the international airports were situated. During the construction of the Benue Bridge a gust anemometer had been installed and, to the best of his recollection, there had been one or two occasions during the period when gusts in excess of 80 m.p.h. had been recorded, and some years ago a gust of 100 m.p.h. had been recorded at Kano.

157. In § 136 was given a list of the professional staff employed. There were eight engineers; the job had cost £682,000; and the bridge had taken 2 years to complete. The ratio of £42,000 per engineer per annum was an interesting figure and might possibly

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† Proc. Instn civ. Engrs, vol. 9, p. 395 (Apr. 1958).

be useful, especially to Directors of Public Works, when calculating staff requirements for major works.

Mr P. S. A. Berridge observed that the crescent shape of the arch was very pleasing to the eye, and he thought the bridge was a fine example of the export of one of Britain's greatest national assets, namely first-class engineering. A great deal of serious thought had clearly been given to the ingenious ways of simplifying the chord joints of the 805-ft arch and to the avoidance of secondary and deformation stresses in the connexions of the lateral bracings to the cross-girders.

159. He noticed from § 83 that the longitudinal welds on the chord members and the fillet welds on the cross-girders and stringers had been made by the submerged-arc process, using two machines in tandem. He doubted whether there was really any advantage in laying down the web-to-flange fillets in the horizontal-vertical position since it was difficult to avoid undercutting unless the girder was tilted to contain the molten metal. Moreover he thought the submerged-arc process was very prone to hidden defects such as "piping" voids which in fillet welds could not be detected by any non-destructive test.

160. He suggested that any saving in cost by using intermittent welds (Fig. 9) would be negligible compared with the saving in maintenance that would have been achieved had the surfaces in contact been completely sealed with continuous welds.

161. The protection of steelwork by zinc spraying and the subsequent application of four coats of zinc chromate (§§ 123 and 147) appeared to be of doubtful value. Recently a series of tests comparing the protective values of sprayed metal, used instead of, and in conjunction with, paint on mild steel had been carried out by the Research Department of the British Railways Division of the British Transport Commission. The results, given in Report No. 75 published in August 1957, showed that the metallic coatings of either zinc or aluminium did not show to advantage as compared with a grit-blasted steel surface under a good four-coat paint system. The inference was that the advantages claimed for metal spraying were really due to the surface preparation beforehand, namely, the removal of mill scale by grit-blasting, and that if this was followed by good painting the result was as good as a metal sprayed coating but considerably cheaper.

162. Referring to Figs 12 and 14, Mr Berridge asked what provision had been made for lubricating and for protecting the sliding surfaces where movement was bound to take place between the stringers and the cross-girders and between the lateral bracing and the cross-girders. Such places would be particularly vulnerable to corrosion since neither metallic coating nor paint would resist the wear on the surfaces in contact. He asked why the fingers of the expansion plates in the roadway were tapered in plan, and observed that the arrangements for draining water away below the expansion joint might give trouble, for the steel channel was wholly inaccessible without taking off the finger plates. He thought a comb joint consisting of vertical plate fingers that he had illustrated* would have been more satisfactory, because there was no contact between the fingers on one side of the joint and those on the other and it was simpler to drain properly.

163. Was it wise to slacken off high-strength bolts to a predetermined tension (§ 72)? He understood that once a high-strength bolt was torqued-up and the metal stretched beyond the yield point, the screw threads of the nut suffered some slight deformation; and if for any reason that bolt had to be slackened off it had to be replaced by a new one.

164. Referring to § 91, he wondered what had gone wrong with the steel jigs with hard steel bushes used in drilling stringer ends, cross-girders, etc. On Indian railways it had been the practice to insist on the shop-assembly of but one span out of every ten similar ones where all parts were drilled to steel templets with hard-steel bushes. So long

* Proc. Instn civ. Engrs, vol. 9 (Apr. 1958). Discussion on Paper No. 6223, Fig. 20, facing p. 464.

as the work in the shops had been properly supervised there had never been any trouble during assembly on site.

Mr H. Shirley Smith said that the Paper was a record of a major advance in bridge construction. The chord joints had each been made by means of four longitudinal high-tensile bolts and during cantilever erection the upper chord had, of course, been in tension. That meant that the whole weight of the arch beyond the point of the attachment of the cables, together with the weight of the erection crane, had been dependent on eight bolts. This, to his mind, was a courageous innovation in bridge building.

166. The method had the advantage that the longitudinal bolts pulled the butt joints tightly together and did not permit any "slip" or opening of the joint, such as was known to occur in conventional riveted connexions. This slip in riveted joints was of no great significance, because when a riveted arch was completed and the chord was all in compression, he knew from experience that the butt joints automatically closed up tight. The elimination of this slip, however, had two advantages. First, it enabled the Volta bridge to be not only closed but also to be converted to the two-hinged condition in one operation, thus saving a week's work; and secondly, it prevented the occurrence of the temporary "dip" which developed in the end of any long riveted cantilever during erection, and to which the Authors had referred in § 64.

167. Mr Shirley Smith then made some comparisons between the Volta and Birchenough bridges. He liked the crescent shape of arch which the Authors had adopted. It was not only aesthetically satisfying but also economical, because the depth had been maintained at the quarter-points, where the bending moments were high. Nevertheless, there were one or two factors in favour of the Birchenough shape, which was spandrel-braced, but without any reverse curve at the ends of the upper chord. First, there was *no significant reduction in depth at the quarter points, whereas there was some reduction* at Volta; and secondly, it had been possible at Birchenough to make all the web members (except three at the ends) of identical length, which meant a considerable saving in shop work.

168. The cross-section of the chord at Volta was similar in size to that at Birchenough and was about the minimum in which a man could work. The percentage for details such as connexion plates, diaphragms, manhole covers, rivet heads, etc., had been kept down at Birchenough to 22%. What was the corresponding figure at Volta? In view of the fact that the Volta steelwork was welded and also on account of the neatness and economy of the details, he thought the percentage would prove phenomenally low.

169. He had known that the allowable stresses used on Birchenough, which had been designed 25 years ago in Chromador steel, were as high as was justifiable; but it was interesting to note in the comparisons given below how little it had been possible to increase them since that time:—

	Birchenough Bridge	*Volta Bridge*	*% increase*
Allowable tension stress in net section: tons/sq. in.	12·0	12·5	4
Allowable compression stress in short struts: tons/sq. in.	10·2	10·4	2

170. Reference to three other innovations made at Birchenough was also relevant, for they had their counterparts in the Volta Bridge. The first was the wind pressure to be adopted in the design, which for Birchenough had been determined by testing models of the bridge in a wind tunnel. Incidentally, he believed that this was the first bridge on which that had been done on either side of the Atlantic. The Authors had adopted a wind loading of 21 lb/sq. ft for a speed of 80 m.p.h. He thought that was a bit low and would have expected 24 or 25 lb/sq. ft; but no doubt the Authors had good reasons for adopting that figure.

171. The Authors had referred in § 49 to the fact that almost the whole area of the thin web plates in the chords could be assumed as effective section. Here again much

less had been known 25 years ago, and for Birchenough, models had been made of the thin-webbed chord members and tested to investigate this question. Had any models been made for the Volta Bridge, or were the facts well established?

172. The third and perhaps most significant innovation made at Birchenough was the omission of the direct transverse members of the lateral system. He thought this was the first major bridge in which these cross-members had been left out, and the lateral systems consisted simply of diagonals with a cross-member at the end. Since then the method had been followed in the Bailey bridges and it had been adopted at Volta.

173. The omission of cross-members permitted the occurrence of small temporary lateral distortions in and out at successive panel points. The broken lines in Fig. 32 show typical distortions (to a much exaggerated scale) in the end panels of a lateral system in which the chords were in compression but the diagonals were unstressed.

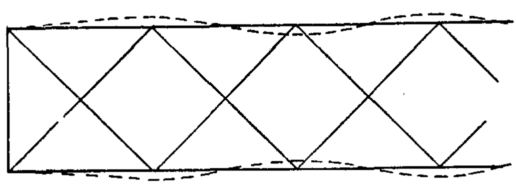

FIG. 32

174. For Birchenough Bridge the chords had been fabricated to such lengths that under the final dead-load compression they assumed their geometric length, so that the arch had then been in its true geometric shape with the chords, of course, all straight laterally. But the engineers at site had been much exercised by the problem of determining what the temporary lateral distortions should be at various stages of erection, when the stresses in the chords were very different from those in the final condition. Did this problem have to be considered for the Volta Bridge or did the Authors take the view that there was sufficient precedence to know that it would all come right?

175. He noted that temporary sway frames had been used during erection and asked if the diagonals in them were of rigid members. At Birchenough, cables had been used for this purpose and when tightened the effect had been simply to pull the panel points in, so that temporary cross-members had had to be added at these points.

176. Turning to fabrication, he applauded the decision to put edge covers on the ends of the butt welds. The Authors had referred in § 81 to cracks forming when gas-cutting high-tensile steel. Mr Shirley Smith recollected that on the arch of Sydney Harbour Bridge slots had been burnt in the lateral gussets to let the main gussets through. He did not think that any special preparation had been made or that the slots had been welded round afterwards; he did not think that they had ever cracked. The accuracy of the fabrication had apparently proved itself in the field.

Mr T. J. Upstone said that the erection of the Volta Bridge had presented the same basic problems as other arches. Practically the only way to erect such an arch was by cantilevering out the two half-arches, using temporary tie-backs. The anchorages of such tie-backs often caused anxious moments. There did not appear to be any satisfactory method of allowing for the effect of the compression from the cable saddle when calculating the resistance of the type of anchorage shown in Fig. 19, Plate 2. When digging one of the temporary cable anchorages, the rock line had been found to be much lower than anticipated. A model of the anchorages had been made, using builder's sand, and this had given satisfactory values for the factor of safety.

178. The design of the creeper cranes had also presented many interesting problems. A crane on a travelling platform was a heavy piece of equipment to haul up an arch, and

consideration should be given to the use of a crane in fixed positions on the arch, it being dismantled and rebuilt ahead of itself as the work proceeded. This method had been used for the Newcastle bridge. Since the cranes used for the erection of Birchenough Bridge had been available for use at Volta, it had been decided to mount them on a travelling carriage with the crane mast on the centre-line of the bridge. The carriage had been made from permanent cross-girders, stringers, and deck bracing.

179. If the crane mast could have been positioned over one truss instead of on the centre-line of the arch, the crane carriage would have been considerably lighter. The crane and carriage had weighed 60 tons and to move it up the back of the arch with a maximum inclination of 40°, paying due regard to safety, needed a little consideration. It was desirable that some safety device be used which would prevent the carriage running away. Automatic devices such as wedges, racks, or counter-weights had been considered and abandoned, for they could not have been made to operate automatically when lowering the cranes down the arch. Security had been provided by duplicating the haulage system of hand winches, blocks, and wire ropes, so that one system could follow up on the other. In the event of any failure of one system, the other could take up the load. This sytem had two other advantages. The second haulage system could supplement the first on the steeper parts of the arch, though each set of tackle was well able to move the carriage by itself. Also the carriage could be held on one system whilst the other was run out to a new anchorage ready to move forward again. The haulage winches had been carried on platforms cantilevered from the sides of the crane carriage. An interesting feature was incorporated in the design of the winch platform; it had three sections at different slopes so that as the crane negotiated the different inclinations of the arch, the winch operators could stand and get some purchase on whichever part of the platform suited them best.

180. The forward anchorage for the blocks had been a temporary transverse beam spanning across and cantilevered outside the two trusses. Two such beams had been provided. The crane carriage had been held on one set of winches and anchorage beam while the second set of winches had been connected to the second anchorage beam fixed one panel ahead. The load had then been taken on the second set of winches, while the blocks of the first had been moved forward. The crane had then been moved forward with both sets of winches operative.

181. Both cranes had been successfully moved up the arch to the centre of the span where they were dismantled.

182. It had been originally intended that the cranes should be lowered down the arch erecting the suspended deck as they went, but the floating crane had proved to be a more useful tool for the erection of the deck.

Mr E. M. Gosschalk referred to two phases of the operation of construction in which, he thought, a different approach might offer advantages in future work of the kind.

184. The excavation of the anchorage shafts had been both laborious and slow. The sinking of a shaft working from the surface was notoriously slow, and in this case the work had been further hampered by the constricted cross-section of the shaft, and by the need to use extremely light charges of explosive, in order not to weaken the structure of the rock. The construction of an adit in order to tackle the excavation of the shaft from below would have been impossible.

185. A shallow anchorage, however, although no doubt more extravagant in quantities of excavation and materials, would, he believed, have proved quicker to construct. A solution neater than the one just suggested, however, employing the principle of drilling from the surface and rock-bolting, might possibly be devised. The design of such an anchorage might be arranged so that it would be possible to apply test loads to the individual ropes after erection.

186. Referring to the approach spans, the construction of the timber trestles which supported the shuttering for the reinforced concrete deck had been a laborious and time-devouring task. In this case there had been, he believed, sufficient reasons for choosing the form of deck construction adopted, but whatever the advantages obtained, the use

of composite reinforced concrete and structural steel construction would have made it possible for the shutters to be supported directly from the structural steel, as had been done for the main deck, and the site work would have been carried out more quickly and easily.

187. With regard to the cement grouting of the abutment foundations, the risk of disturbing the abutments after the arch had been erected was mentioned in the Paper. The possibility of eliminating this risk by drilling the holes in advance of grouting with cement had been considered. In that way the ungrouted holes would act as pressure-relief holes and also as telltales to indicate if the grout pressure were becoming effective over a dangerously wide area. In the event, it had been decided that in this instance the safest course was to discontinue the grouting.

188. The deck steelwork had been levelled in the following manner. Levels had been taken at each end of every cross-girder as soon as possible after the erection of the steelwork was completed. These levels had been plotted, and the adjustments necessary to bring the points on to the nearest smooth curve had been found. The adjustments had then been made to the hanger connexions at the lower-chord panel-points as mentioned in § 53, thus compensating for any irregularities in the lengths of the hangers or in the positions of the panel points on the lower chord. No attempt had been made to make adjustments to bring the finished deck on to the nominal curve designed, and no adjustment had been necessary after concreting had been completed, since the result was satisfactory to the eye.

189. The settlement of the east approach embankment, amounting to 10 in. (§ 111) had continued to be the object of study. Figs 33 and 34 illustrated the course the settlement had taken.

190. There appeared to be a relation between the settlement and the depth of overburden (Fig. 33), and the conclusion which might be drawn was that the major part of the settlement was taking place in the overburden and not in the embankment itself.

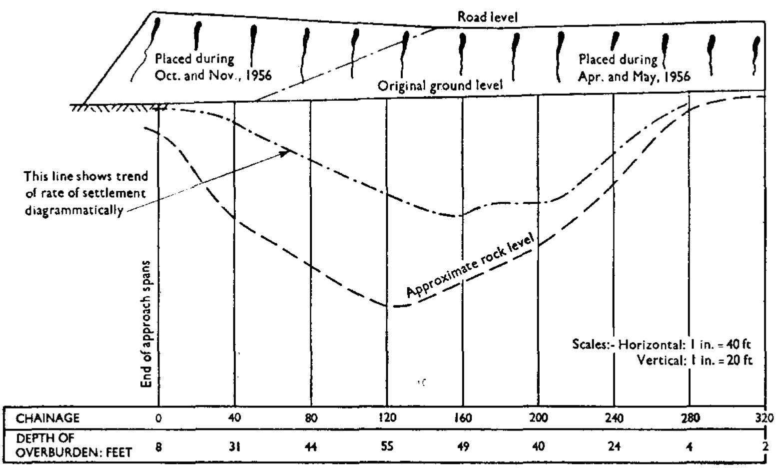

FIG. 33

191. Settlement appeared to be decreasing reasonably steadily (Fig. 34), although there appeared to be a flattening of the settlement/time curve after June and October 1957, which suggested that settlement might be affected by the water-table; the river level had reached a minor peak in June/July, the main flood occurring in October.

192. Some settlement must have occurred before the commencement of recordings

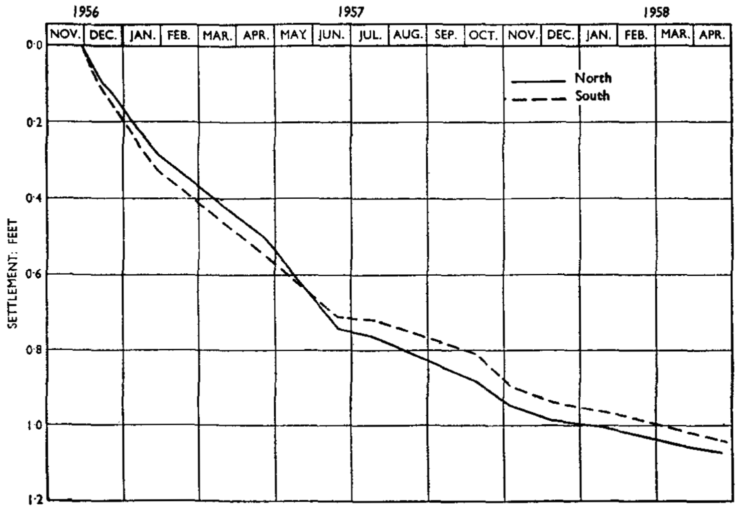

FIG. 34

and a rough estimate showed that this might have been as much as 12 in. Including this figure, the maximum settlement since construction of the embankment might now have reached about 2 ft 4 in. Some compensation for settlement had been made after the placing of the main fill, so that the apparent settlement was not more than about 1 ft 4 in. In this case it appeared to be more practical and economical to await events, and subsequently to reinstate the road if required, rather than to carry out expensive soil investigations and/or expensive works in order to predict and forestall this contingency.

Dr W. C. Brown said that perhaps the most unusual feature of the arch design was the way in which the main chord box members were spliced.

194. A model joint had been made consisting of a 4-ft length of a typical chord member including a butt splice. This had been used, not only to determine the approximate torque-load relation for the bolts, but also to prove all the bolt-tightening equipment and evaluate various types of lubricant. The most effective grease had been found to be an anti-scuffing paste containing a high porportion of molybdenum disulphide. Its use had resulted in an average coefficient of friction on the cut threads of 0·17.

195. The most difficult problem concerning the bolt tightening equipment had been the design of a ratchet spanner which would not only work in the limited space available, but would be reasonably portable. This had been accomplished by making use of the "wrap around" principle as used in the old type of band-brake. After much experimenting, a suitable pawl and rocker had been evolved which effectively held the "tail" of the band and enabled a spanner to be produced, weighing little more than 13 lb., and capable of applying torques up to 6 tons-ft. The load had been applied to the end of the spanner by means of a small hydraulic ram operated from a remotely controlled pump. The tightening technique had been fully developed in the United Kingdom and in practice had worked extremely well.

196. The hollow piston in the ram contained a helical return spring and this meant that once the ratchet spanner was fitted over the nut and the rod locked, all the operating could be done outside the chord.

197. Usually eight or nine strokes sufficed to tighten a bolt and in general a joint

could be completed in just over an hour. The contractors had used the pressure-gauge readings to measure the bolt loads and these had been checked by measuring the extensions with a special micrometer. Any further adjustment had been applied by estimating the rotation of the nut required from previous experience.

Mr C. R. Blackwell said that one of the most unexpected difficulties had been the deterioration of the zinc coating, caused by some sort of chemical attack while the steel was being transported by sea. When it had been decided to protect the steelwork by zinc spray alone and not to paint it, they had expected that there would be some mechanical damage during transit and during erection, and they had been prepared to repair it. The contractor had had a small plant for sand-blasting and spraying on the site, and it had been intended to touch up the small patches with zinc paint.

199. He showed a number of slides illustrating how some of the steelwork had arrived on site showing signs of deterioration in the form of patches of white deposit, through some of which spots of rust had shown. A careful investigation had shown that the damage had not been caused by ordinary sea spray. Nor was it the result of faulty zinc spraying in the shops of the United Kingdom. Eventually the cause of the trouble had been traced to the action of other cargoes stowed with the steel on board ship. One consignment of the steelwork had been stowed for the sea passage in such a way that it had been covered by a quantity of salt fish in sacks, which had been taken on board at a West African port. Four days under these conditions had been long enough to cause the incrustation which had been so disturbing.

200. All the steelwork so affected had had to be sand-blasted and resprayed.

201. The conclusions to be drawn from all this were, very obviously, not to send steel in that way again. If the steel was to be sprayed in the United Kingdom, it should have some sort of shop coat which could then be allowed to weather and be the same colour as the steel. A newly sprayed zinc coat was ideal for painting. It was porous, it had very small microscopic cracks which could be filled with paint. If it were not painted the cracks would fill with zinc oxide owing to normal weathering but in the meantime would retain any material with which the zinc came into contact during transit or handling such as dirt or at the worst salt. Had there not been this chemical damage, he thought that mechanical damage due to handling could have been very easily dealt with on site. The extent of such damage had been small.

The following contributions were received in writing.

Mr K. D. Robinson asked if the aero-dynamic stability of the deck had been studied.

203. Was it expected that the 2¼-in. hangers would stretch during the first few years of their service, and if so, would it be possible to correct the deck camber?

Mr J. F. Pain remarked that outstanding features of the work were the manner in which all secondary material had been reduced to the minimum, the simplicity of the rib joints, and the clean outline of the whole structure.

205. While it was stated in § 68 that it had been realized from the outset that the arch would have to be erected by cantilevering with temporary tie-backs, the contractors had given a lot of thought to possible alternative methods. All of them, however, finally appeared to involve costs and risks higher than those of the conventional method adopted.

206. From the contractors' point of view the most original feature of the design lay in the arch rib joints, particularly those of the top chord. They functioned admirably and had given very little trouble on site, although it should be confessed that the realization that the whole structure depended on the four heat-treated high-tensile bolts in each top-chord joint, stressed to more than 20 tons/sq. in., had caused an occasional twinge of anxiety during the critical stages of cantilever erection.

207. A fundamental problem in the cantilever erection of arches was to get the erection crane ahead of the first cable anchorage. The crescent shape of the arch made this more difficult than for an arch with vertical end-post of the Sydney Harbour Bridge or Hell Gate Bridge type. The difficulty had been solved by the use of the temporary ramp shown in Fig. 21, Plate 2, which formed part of the first anchorage.

208. Mr Pain drew attention to the contract requirement that the bridge should be finished within 15 months of the order to commence, in spite of the remoteness of the site implied in § 10. Incidentally, it seemed possible that the inhabitants of Sydney might resent the linking of the remoteness of their city with the sites of the Birchenough and Volta Bridges. In the case of the Volta the normal interval between the readiness of material at the works and its arrival at site was from 6 to 7 weeks. In spite of the fact that circumstances ultimately prevented its achievement, every effort was made in preparing the scheme of construction and planting the job to meet the contract date. In the circumstances prevailing at the time, all plant and equipment employed had to be of types readily obtainable from suppliers without delay. This fact largely influenced the decision to employ hand-winches for the movement of the erection cranes up the arch, which had prompted the Authors' reference to their "laborious" progress in § 115. The provision of plant and temporary works for the cantilever erection of an arch of this type had to be worked out in detail at a very early stage and could not be left to improvisation at the site.

209. By far the most critical feature of erection lay in the movement and operation of the erection cranes and any mishap there could easily become a major disaster. The weight of each crane and carriage was approximately 60 tons and the maximum chord slope 40°. The suggestion in § 47 for a possibly higher ratio of arch rise to span with correspondingly steeper slope would add further to the crane-haulage problems. The cranes used on both the Birchenough and Volta arches had had a single telescopic back leg for the adjustment of the slope of the mast with wire rope lateral stays on each side of the mast head. The crane mast travelled along the bridge centre-line, necessitating heavy beams in the carriage to carry the reaction to the chords.

210. The whole construction of the bridge was greatly facilitated by the use of the 7-ton floating crane, but the successful use of this tool in an 80-ft-deep river, under flood conditions, had been something of an act of faith.

211. The reference to night-work on steel erection of this kind in § 135 was rather disturbing. European supervising staff was inevitably limited and experienced local labour non-existent. Erection hazards were appreciably higher on an all-welded structure without the customary foothold provided by rivet heads. Effective safety precautions were virtually impossible at the points of greatest hazard where erection was taking place at the ends of the cantilevers. It was felt strongly that the possible gain in speed by working under artificial lighting on a contract of this sort was not justified in view of the greater risk to the lives of the labour employed.

Mr R. W. Turner referred to the close tolerance of the high-tensile bright steel bolts used for connecting the secondary components and asked for details of the tolerance. What steel had been used for the bolts and nuts? What was the method of tightening the nuts and what was the preload, if any?

Mr N. J. Cochrane observed that the Paper illustrated clearly the presently unresolved dilemma which faced engineers in protecting their structures. He knew from direct observation that corrosion at Adomi was very limited but some superficial rusting would inevitably take place and disfigure the structure. In spite of painstaking scientific efforts, most paints had a limited performance, while thin metal spray coatings had obvious vulnerabilities in a structure of such size, if only because of casual damage to the protection before the structure was completed.

214. He suggested that a new approach might usefully be made to the problem of the outdoor protection of structures of such size, where the normal methods were demanding in time and labour and recurrent. He suggested that many of the current complex

mineral oils had been designed to give good spreading qualities and very effective adhesion to metal surfaces. Would it not be feasible to design the bridge to have a system of either drippers or pressure oiling points from which a continuous small supply of the protective oil could be applied at chosen points and rely on its spreading qualities to maintain the protection of the structure?

215. Alternatively, why not consider the kind of underbody protection now applied to motor vehicles, which, from his own observation, had a much longer protective life than the best paints in the exceptionally severe conditions in which they were applied?

216. He had observed on a small scale that normal motor oils did in fact attain the objectives he had mentioned above, and that even one drop remained spread and in protective contact with steel for an appreciable period. Objection might be taken to the appearance, which was naturally not very attractive. However, he did not see why some colouring matter should not be added to the oils if necessary—for example, aluminium powder. In ordinary exposed positions the oil did not seem to retain a thick coating of dust.

217. Finally, he emphasized the part which the African workers had played in the erection of the Volta Bridge. Both the work and the scale had been quite outside their experience and yet with resourceful leadership they had risen to the occasion in a very encouraging way.

Mr T. Ridley asked if the Authors could give their reasons for according with the final choice of a single-span steel arch bridge? From the points of view of maintenance and appearance the claims of a concrete structure would normally be taken to be superior, and it was stated that a prestressed concrete multi-span bridge of 2,970 ft length was considered. It would be of interest to have more details of that scheme, since an estimated period of completion of 17–20 months represented a remarkable rate of progress for a bridge of such a length. In view of the fact that the period of completion for the arch bridge had been increased to 20 months, would the Authors still consider this to be the speediest form of construction which could be adopted in future similar overseas bridge schemes? Where favourable concrete aggregates could be found did the Authors not consider that a prestressed concrete structure offered many advantages in assured speed of construction?

219. It was mentioned that the Volta Bridge was designed to the revised B.S.S.153 loading specification which had not at that time been published. The Iddo Overbridge, at present under construction in Lagos, Nigeria, was also designed for this loading specification before publication, and it was a point of interest to know that the period between draft agreement and publication of this specification was being eagerly anticipated by other bridge designers. Did the Authors not consider that for a bridge in such a location as the Volta Bridge, the requirements of the B.S.153 specification were too severe, and that some reduction of normal type HA. lane loadings could be adopted?

220. The design of the reinforced concrete approach spans was pleasing and the cantilever footways were very slender. Full advantage of the accepted concrete design stresses had obviously been taken. These were higher than was usual for West Africa and it would be valuable to have more information on the actual cube results and standard deviation obtained in the work. The difficulty in the supply of sound cement through the usual dock facilities at Takoradi was surprising. Could the Authors give details of any B.S.S.12 compression tests made on this cement, and state the type of containers used? Experience in Lagos with high-strength reinforced and prestressed concrete had shown that the cement arrives in first-class condition, and B.S.S.12 test results are up to normal U.K. standards. That would seem to indicate that the deterioration experienced in the cement supplied to Volta Bridge might have been due to the methods of transport from the docks and subsequent storage on site, and any further information which the Authors could supply on this subject would be most interesting.

Mr D. W. Manton congratulated the Authors on the clean outline of the main arch which reduced to a minimum the number of pockets in which water might collect.

222. He had been mainly concerned in the preparation of the scheme of erection and particularly with the layout and details of the anchorage cables and cable masts. The ingenuity mentioned in § 113 had been mainly patience and perseverance. It had originally been intended to make much greater use of screwed rods in these anchorages but, since H.T.S. rounds proved to be unobtainable, flat links were substituted. The resultant limited amount of available adjustment in these cables had necessitated very careful calculation of the rope lengths and structure deflexions, particularly when changing from one rope arrangement to the other.

223. Tests had been made, in the 1,250-ton testing machine at Middlesbrough, on sample lengths of permanent suspender rope and of the $1\frac{7}{8}$-in.-dia. temporary rope, in order to determine the modulus of elasticity and the permanent stretch to be allowed for the cutting and socketing of these ropes. Figures returned from site suggested that the permanent stretch was possibly under-estimated in these "short-time" tests. It was also interesting to note that, although the ungalvanized $1\frac{7}{8}$-in. rope (which was originally used on the Menai Bridge reconstruction) had been stored for many years with only the most elementary protection, the inner wires of the broken test piece had been clean and bright and the lubricant still moist and sticky. The design of one type of socket used on these $1\frac{7}{8}$-in. ropes had not permitted the ends of any wires to be turned back in the socket, in spite of this the connexion had proved to be stronger than the rope itself.

224. Would Authors permit the use of a thin parting plate (say $\frac{3}{8}$-in. thick) between the butting ends of similar chord members in order to allow the correction of possible shop errors in milling the butting faces? If this were incorporated in the design of the joint, a bad butt could be corrected on the parting plate, without returning the chords themselves to the ending machine.

Mr J. T. Gregg observed that the width from curb to curb, namely 22 ft, was precisely the same as the pavement width on the approach roads. Had the designers not considered it desirable to provide paved shoulders on the structure as was prevalent in the modern design of long-span structures?

226. He thought that 8 months was a disproportionately long time to take for a geological investigation. His experience had generally been that it was better to produce a fairly "flexible" design for footings founded on bedrock, the exact extent of excavation being determined during construction.

227. He did not agree that this was the first major bridge designed in high-tensile steel, completely welded. This was quite common practice in North America and Europe.

228. The main arch details were highly ingenious, in particular the method of splicing the chords. Were any special measures taken to weatherproof the butt joint, or was this not considered necessary?

229. The rise/span ratio of 1/5 for the main arch seemed rather high. The average ratio for this type of bridge was of the order of 1/6.

230. He noted that the stringers were designed as composite. With A.A.S.H.O. loading it was questionable if composite construction was economic for a span of less than 40 ft. Unfortunately, he was not familiar with B.S.S.153 but he suspected that the H.B. abnormal unit loading was heavier than A.A.S.H.O. H20–S16. If this was the case, a normal simply supported stringer would have been more economical. Surely the small extra weight required for a simple beam could not have been considered of major significance.

231. The T-type shear connectors were interesting but would not be acceptable by A.A.S.H.O. standards since they did not tie down the slab to the beam. Loading tests had proved that, to be fully effective, shear connectors should possess the double function of transmitting the horizontal shear between slab and beam and of tying down the slab to the beam.

232. He would not have thought it desirable to pour the concrete deck continuously over the cross-girders. Since the stringers were simply supported, it would have been preferable to insert a small cork-packed transverse breather joint to coincide with each

cross-girder. Without such a joint the sliding bearing on each stringer was an unnecessary elaboration since the stringers and concrete were connected by the shear connectors and thus, the stringer could not move relative to the deck slab.

233. Mr Gregg would have thought that, for a span of this size, a comb-type roadway expansion joint would have been preferable to the finger plates adopted. The long fingers required were very slender. Were they flame cut, and if so, what was the minimum radius at the finger tip? He had found that with a flame-cut radius of less than 2 in. for plate $1\frac{1}{2}$ in. thick, there had been a tendency to burn off the tip.

234. He noted that a drainage trough had been provided under the roadway expansion joint. Such troughs usually became clogged up with dirt in a very short time and were in need of constant maintenance. He suggested that it would have been better to dispense with the trough and let the dirt fall free.

235. There was no mention of deck drainage. On similar bridges over water it was usual to place short vertical downspouts at approximately 20-ft centres, letting the gutters drain directly into the river below. The outlet of these was generally just a few inches below the bottom of the deck slab.

236. He did not think it was good practice to cast steel handrail posts into the deck concrete. It was inevitable that a vehicle would eventually come in contact with the handrail and bend a few of the posts and panels. The posts should have been bolted to the concrete to facilitate repair.

Mr Scott, in reply, agreed with Sir Hubert Walker that the completion date had indeed been an optimistic one. Nevertheless, the job could have been done quicker but for certain factors which, it appeared at the time, could have been avoided. However, it was precisely those things which one encountered in a remote site which should be taken into account when one was assessing the time required for the bridge. He agreed that in many respects the contractors had done very well indeed to complete the bridge when they did.

238. More details regarding the method of estimating the catastrophic flood had been requested. There had in fact been a great deal of information available on the flow of the Volta because the work which had being going on since 1950 on the Volta River Project had involved very extensive hydrological investigation. Moreover, much work had been done in the past on the same subject, and records existed over a period which, for Africa, was very long. Fortunately a model had been built in the United Kingdom in connexion with the Volta River Project, and it had been possible to check some ideas as to levels on that model. The assessment of the catastrophic flood had been done theoretically, of course, and two approaches had been made.

239. In Richards's method[7] the effect of major rainstorms of extreme intensity on such a catchment was considered in relation to its geography and slope. It was estimated that a "catastrophic" peak of about 1,000,000 cusecs should be expected.

240. For what it was worth a probability method had also been used based on 19 years of known river flows. Apart from these, a flood in 1917 was thought to have reached a peak of about 500,000 cusecs. The probability method suggested that a 900,000 cusec flood would have a probability of occurrence of once in 10,000 years. Nevertheless, in the 11 years including and subsequent to 1947, floods of over 300,000 cusecs were experienced no less than six times, and if this be added to the 1917 flood, some doubt must arise as to the reliability of the probability method in dealing with these extremes.

241. In the event, the Richards catastrophic flood of 1,000,000 cusecs was taken for design purposes.

242. It had been asked if the Kariba flood gave food for thought. The Author was not qualified to speak on the recent history, but a lot of hydrological and other work had been done on the Kariba scheme some years ago and he would say that the catastrophic flood, if one liked to call it that, of 1958 in Kariba had been within the figure estimated.

[7] B. D. Richards, "Flood estimation and control". Chapman and Hall, Lond., 1950.

A point which made the recent flood particularly exasperating for everyone was that in the nearby Shire Valley in Nyasaland the year had been a particularly dry one.

243. With regard to wind speeds, an anemometer had been installed at Ajena, but in 4 years wind speeds in excess of 30 m.p.h. had not been recorded there. Estimates based on the observed effects of gusts at the site, however, suggested that 50 to 60 m.p.h. would be more realistic. The maximum wind speed recorded in the country since 1947 was 69 m.p.h.

244. In any case, when considering the matter of the stability of the bridge as a whole, it was questionable what fraction of the maximum gust velocity should be used in arriving at the mean, and it might be noted that experiments to investigate this type of problem were being carried out by the Meteorological Office.

245. The Engineers had considered that the job required close supervision and so had staffed it fairly well. The Author agreed with Sir Hubert that almost any Director of Public Works would give an eye to have as many engineers as there had been on this job. As a matter of interest, the supervision costs alone, i.e. the cost of recruiting and sending the men out and of maintaining them there, amounted to about 8% of the contract total.

246. Mr Berridge had dealt with metallic coatings and had plumped in favour of the four-coat paint work. As indicated in the Paper, a great deal of thought had been given to the alternatives. The guiding principle had been that in this isolated area, and with a bridge of this height, the maintenance problems should be cut down to the absolute minimum, and it had been felt that the results of tests and reading on the subject fully justified the choice of zinc spraying. Had four-coat paint work been adopted, it was felt that the site coats, particularly in the less easily accessible parts, would not have received the treatment which was their due. As a general principle on work of this nature, the more work that could be done in a shop the better, and of course zinc spraying was, in general, done in the shop.

247. Mr Scott agreed with Mr Gosschalk's remarks—again from the point of view of having as little site work as possible—that there was room for consideration of the adoption of some form of composite design for the reinforced concrete approach spans which would have avoided the use of struts for shuttering.

248. Mr Blackwell had also referred to the zinc coating. The Author agreed that the lesson was to shop-coat the zinc before it was shipped and to take very great care with the shipping company to see that the stowage was very carefully done.

249. In reply to Mr Robinson, it was thought that it should be possible to correct the line of the deck, and on completion of concreting an attempt would have been made to do so if the line had been irregular.

250. Mr Ridley appeared to favour the alternative of prestressed concrete construction. Clearly the programme and the form of construction of any bridge is a matter for individual consideration in each case. A disadvantage of prestressed concrete construction in this case was the high quality of concrete which would have been required, and which it might have been difficult to achieve consistently at this remote site. The provision of more skilled labour might also have been difficult, and the advantages of having the bulk of the skilled work carried out in the United Kingdom would have been lost.

251. It was not considered that the loading specification adopted was too severe, and Mr Ridley was referred to §§ 3, 4, and 25.

252. Further information on the concrete-cube test results had been asked for: altogether about 560 cubes of all classes were tested and in the case of the highest quality of concrete the mean 7-day strength had been 4,190 lb/sq. in. and the standard deviation 730 lb/sq. in.

253. Compression tests on the cement had not been used. The testing of 2·78-in. cubes was considered, but in the past results had proved erratic. The latest revisions to B.S.12 confirmed the opinion that cement was best judged by the strength of 6-in. laboratory cubes made with a standard mix of concrete, although the revised B.S.

requires smaller cubes which appeared to offer no advantage and some disadvantage compared with the 6-inch size.

254. To Mr Ridley's comments on the deterioration of cement, it could only be added that the storage shed provided at the site was of sound design, but it would not be surprising if deterioration had commenced during transport from the coast, when it had not been found practicable to ensure that proper precautions were always taken.

255. In reply to Mr Gregg, it was hoped that the relatively high kerbs on the deck would have a psychological effect in deterring prevalent impetuous drivers from mounting the footway.

256. The geological investigations ran concurrently with the preparation of designs and in no way held up operations. It was essential to confirm the soundness of the main foundations, but Mr Scott agreed that it was desirable for the design to be flexible, and in fact several modifications had been made as a result of conditions revealed during excavation.

Mr Roberts, in reply, said that the maximum wind speed for which the bridge had been designed, 80 m.p.h., was supposed to allow some margin over the maximum likely to occur at the site.

258. With regard to Mr Berridge's remarks on welding, the reference to tandem was a mis-statement. The machines had run in parallel and not one behind the other. A characteristic of submerged arc welding was supposed to be that any faults present always showed on the surface, instead of being concealed, but that might not be altogether true. There certainly had been some faults at the surface which had been corrected. For checking important welds they had relied on the run-off pads (made by the same process and at the same time), which could be broken open and examined.

259. The virtue of the waisted bolts was that they could be considerably overstrained without the risk of breaking. In fact, the longer bolts would stretch about 3-in. before failure. In some cases where the heat treatment had not been quite right, bolts had been stretched to bring up the elastic properties, instead of having them re-treated. He could see no objection to overtightening and then slackening off the bolt as a means of proof loading the connexion as a whole.

260. The error with the steel-bushed jigs had, no doubt, been in their placing. The practice quoted by Mr Berridge in § 164 referred to the drilling of clearance holes for rivets and it was a very different proposition to locate steel-bushed jigs sufficiently accurately for reaming of holes for close-tolerance bolts. Where one had small groups of holes near the end of the member it was possible to drill and ream them to a jig and then apply that jig to the member that it was going to connect to; theoretically the holes should match, but all depended on the accuracy of locating the jig.

261. The advantage of intermittent fillet welds, referred to by Mr Berridge in § 160, lay in the reduced distortion as well as saving in cost. Provided the edges were tight (and this could be verified by visual inspection after welding) there was no disadvantage in intermittent welding so far as maintenance was concerned. The sliding surfaces in the deck lateral expansion joint were completely enclosed and the initial grease lubrication should last for years. The joint could be examined and greased, if required, by removing the centre bolt and the top plate. The expansion joints on the stringers were not required to move under the maximum load.

262. The percentage for details, mentioned by Mr Shirley Smith, works out to 6% compared with 22% for the riveted Birchenough Bridge. In comparing the working stresses it should be remembered that the steel used at Volta was of the weldable quality with somewhat lower tensile properties than the steel in Birchenough Bridge. If the difference in specified minimum yield point was taken into account it would be seen that the load factor had been reduced from 1·84 at Birchenough to 1·68 at Volta. The virtual increase in working stress was thus about 10%, not 4% or 2% as stated.

263. No wind-tunnel model-tests had been made for the Volta bridge, but the wind-drag coefficients used for the various members had been based on data from experiments made during the past 25 years.

264. The difficulties experienced with the lateral system in the erection of Birchenough bridge, indicated in Fig. 32, must have arisen from some circumstances other than the absence of cross-members. The effect illustrated could be produced by delaying the riveting of the chord joints until the members were carrying appreciable erection stresses. No such effect had been observed at Volta.

265. With reference to Mr Pain's remark in § 206, there was an inherent margin of safety in the use of prestressed high-tensile bolts of this particular shape, owing to the large plastic deformation that must occur before failure. The longitudinal bolts, highly stressed though they undoubtedly were (27 tons/sq. in. including wind loading) were one of the most reliable features of the erection scheme.

266. In reply to Mr Gregg, no special measures had been taken to seal the joints in the chords other than a coating of white-lead paste applied to the plate edges before bringing the members together. All the joints had appeared absolutely tight when the bridge was complete. The heavy British Standard (HB) loading was greatly in excess of the A.A.S.H.O. loading, and the composite deck construction was undoubtedly economical in this case. The T-section shear connectors were drilled with two holes to take $\frac{5}{16}$-in.-dia. longitudinal reinforcing bars (unfortunately not shown in the illustration) so that the concrete was, in fact, anchored to the stringer. The transverse "breather" joints in the concrete slab suggested by Mr Gregg would defeat the object of the design, which was to minimize leakage through the deck by making the concrete slab continuous. The sliding bearings at the stringer ends were not intended to accommodate temperature movements, but to allow for the unequal extension of the hanger cables under heavy loads.

267. The toothed expansion joints had been flame cut from $1\frac{1}{2}$-in.-thick plate and had a $\frac{3}{4}$-in. radius at the tip. There had been no tendency to burn away the tips and this should not occur if the flame cutting was correctly done. It would be noted from Fig. 15 that the fingers were supported by a channel cross-member so that they were quite capable of supporting heavy wheel loads. The plates were easily removable for cleaning the drainage trough, which, incidentally, was essential if road dirt and water were not to be distributed over the steelwork below. In many designs too little attention was given to the drainage of expansion joints with the result that deterioration of the structure rapidly set in at these points. The deck was drained by short vertical downpipes at about 35-ft centres.

268. With regard to Mr Gregg's comments on the handrail standards, the 9-in. kerb would prevent most vehicles from mounting the footpath and striking the handrail. If this should occur, however, the cast-in posts were much more likely to stop the vehicle going over the side than the bolted ones that Mr Gregg preferred. As stated in § 47, the rise/span ratio of one-fifth was just about the economic value for this bridge and was, in fact, very close to that generally adopted. The ratios for four similar arch bridges were as follows:—

Hell Gate	(977 ft span)	1:4·5
Sydney Harbour	(1,650 „)	1:4·7
Kill-van-Kull	(1,652 „)	1:6·2
Birchenough	(1,080 „)	1:5

The average of these is 1:5·1.