Paper No. 6425

Design, fabrication, and erection of Ganga Bridge, Mokameh, India†

by

Sidney Turley, B.Sc., A.M.I.C.E.

Sadashiv Gangadhar Savarkar

John Williams

and

Rathlin John Clinton Tweed, M.A.(Cantab.), M.I.C.E.

Discussion

Mr O. A. Kerensky (Partner, Freeman, Fox & Partners) said that it gave him particular pleasure to open the discussion because of the memories of the exciting time that they had had as consulting engineers in the matter of the competitive design, and the very pleasant association with their clients, the contractors.

146. In § 19 Sir Claude Inglis had predicted the behaviour of the river, and no doubt that had been taken into account by the contractors in planning their programme. How had the river actually behaved?

147. Regarding the alternative designs, it was stated in § 23 that a 4% saving in the weight of steel had not made the cantilever with suspended-span design cheaper than the heavier simple span design. Would the Authors agree that this was attributable mainly to the fact that the mass-production method of fabrication weighed very much in favour of the fourteen identical simple spans as against normal economic comparisons based on, say, three or four spans?

148. Alarming tilts of the caissons in both directions were described in §§ 56 and 57. The contractors had done very well to land them so near to their correct positions. What, in fact, were the maximum errors in the positions of the bridge bearings, relative to the centre-lines of the piers, as constructed?

149. Commenting on the configuration of the truss (§ 71), Mr Kerensky said that the adopted double-intersection truss, with decks located approximately at bottom-chord and neutral-axis levels gave the appearance of a single Warren truss to the user of each deck. The motorist, in particular, had an almost uninterrupted view.

150. The timber sleepers had been laid directly on the steel stringers. It was better practice to provide steel packs under all sleepers to reduce the danger of corrosion of the top flanges of the stringers. Unfortunately, competitive design could not include all the niceties.

151. The treatment of secondary (deformation) stresses (§ 90) by the Indian Specification appeared unduly severe. The American and British Specifications allowed about 20% increase in tensile and compressive working stresses, if secondary stresses were included. The Indian Specification allowed only 16½% increase in tension, and no increase in compression. Hence, strenuous efforts had been made to reduce deformation stresses. He thought that the trusses were sufficiently slender not to require any

† Proc. Instn civ. Engrs, vol. 15, p. 231 (March 1960).

prestressing at all. The deformation stresses were between 2 and 3 tons/sq. in. and in almost all modern Specifications that would be permitted in high-tensile steel; thus the whole cost of the prestressing might have been saved. Could the Authors give some idea of that cost, including the cost of the model, and what allowances they had had to make in the price per ton in respect to that somewhat unknown factor?

152. He congratulated the contractors on their jiggging practice which had produced fourteen accurately made trusses, with all members interchangeable. They had been erected without difficulty and had mated easily. What had been the working life of the hard bushes in the drilling jigs? Unfortunately, the Indian regulations stipulated a reduction of working stress in a weld by 33% if it was not X-rayed to the satisfaction of the inspecting authority. In order to avoid this inspection the contractor had chosen to increase most of his welds by 33%. Mr Kerensky thought that was a wrong and harmful decision. If the contractors had had to do the job again, would they have done likewise? If so, what means were adopted to ensure good welding?

153. He approved the use of aluminium in cranes and cradles. It had eliminated a great deal of permanent erection material, which would have had to be built into the structure.

154. The split-pin drift appeared to be a very useful device. Had Mr Tweed invented it, or had it been used elsewhere before?

155. Mr Kerensky was constantly encountering contractors who did not want to use high-tensile rivets. Their use resulted in a great saving of labour (both shop and field) and material, and they ought not to be discarded lightly. Had the contractors received any complaints about high-tensile rivets, which had been used in every joint?

156. It could be inferred from § 141 that the butting ends of chords had been machined to an accuracy of 0.004 in. How had the contractors ensured that the faces came together and butted during erection? Had they adopted some special drifting or uniting device to ensure that the faces were, in fact, bearing?

157. In conclusion, he would remark that the winning of the Ganga Bridge contract against international competition was not a "fluke". This was shown by a recent world competition for a similar bridge across the Brahmaputra River. This had been secured by the same contractors, with practically the same design, against even stronger international competition, including that offered by several German firms. This might give a little encouragement to all those who seemed to be losing faith in the ability of the British engineers to produce bridges of competitive quality.

Mr P. S. A. Berridge (Assistant Engineer (Bridges) British Railways, Western Region), echoed Mr Kerensky's last few words and suggested that the design of the Ganga Bridge constituted a fine example of Britain's invisible exports. He was pleased to see Mr H. K. L. Sethi's name in § 144, and he would like to couple with it the name of Mr G. P. Shahani, Mr Sethi's assistant. Both had received experience in the Bridge Department of the North Western Railway in India. He was particularly glad that Mr Sethi and Mr Shahani had had charge of the Ganga Bridge project and was sorry that neither could be present at the meeting.

159. Mr Berridge went on to say that India, unlike the United Kingdom, still had a good supply of experienced riveters. The choice of riveting for the site joints had been a good one. The alternative would have been high-strength friction-grip bolts. The split drifts shown in Fig. 13 would be most useful in aligning members of truss girders joined by high-strength bolts. He disagreed with Mr Kerensky, for he believed that the principle of prestressing the truss girders was worth while, if only for the reduction in deformation stresses.

160. He thought the design of the Ganga Bridge trusses displayed splendid simplicity. The chord flanges were turned inwards, and the vertical and horizontal distances between the backs of the chord angles had been kept constant. This was very sound practice. Mr Berridge discussed alternative methods of erection, giving examples of the Kalabagh Bridge over the Indus and the Willingdon Bridge near Calcutta, where the spans had been erected on falsework on the shore and then floated out and landed

on the piers. He also mentioned the Hardinge Bridge where the new spans had been erected on a service span, which in turn had been floated from pier to pier. He thought the choice of erection by cantilevering at the Ganga Bridge was by far the best.

161. Placing the road deck near the neutral axis had certainly simplified the elimination of floor interaction, but he wondered if an economy would have resulted from putting the road level with the top chords and so eliminating the top laterals. He agreed that some additional sway bracing would then have been necessary.

162. Referring to § 75 he asked whether the fixed ends of the railway stringers had been bolted or riveted to the cross-girders. He thought high-strength bolts would have been better at these joints because the connexions would be in direct tension due to the end fixing moment of the stringers. At the free ends, he asked what lubrication had been provided at the sliding surfaces, pointing out that the bolts in the slotted holes would, unless they had been collar bolts, either be too tight, and therefore restrict freedom of movement, or would be too slack and therefore of little use. He thought there should have been a cross-frame between the railway stringers just clear of the cross-girders at the free end bearings.

163. Referring to § 84 he asked the Authors to give details of the expansion joints in the roadway. Were these, in fact, self-clearing and did they drain on to the railway deck below?

164. Referring to § 113, Mr Berridge asked what form of automatic welding plant had been used for the plate girderwork. Where were the butt welds located? What control had been exercised over the welders making the fillet welds, and had these men been called on to make periodical tests?

165. From § 143 he estimated that the weight of steelwork came to 2.25 tons/ft run on these 400-ft spans carrying a single M.L. track plus a 24-ft-wide road and two footways. That compared very favourably with the 3.6 tons/ft run for a double-track railway with similar loading in the case of the 345-ft spans of the Hardinge Bridge built in 1917. The Willingdon Bridge, with its two H.M. tracks, two 18-ft roadways, and two footways, had weighed no less than 6.9 tons/ft run on 350-ft spans. Those comparisons showed that the Ganga Bridge was a masterly improvement in design.

Dr Brian Richmond (Assistant Engineer, Freeman, Fox and Partners) compared some of the composite-action aspects of the Ganga roadway deck referred to in § 79 with the requirements of the German and American specifications on composite bridges.

167. The basic material from which the German and American specifications were produced was available at the design stage of the Ganga Bridge, though the specifications themselves had not been published. There were only two important divergencies from the specifications.

168. The first disagreement was with the German specification only. Temperature difference and concrete shrinkage produced shearing forces between the steel stringers and concrete deck slab. The Germans, but not the Americans, required that heavy shear connectors be placed at free ends of girders—which was where the shear forces were concentrated.

169. No special end connectors were used at Ganga because it was thought that the shear forces could be safely distributed over a number of ordinary connectors near the ends. That principle was applicable to most cases where the full advantages of composite action were not required at the girder ends, since any overloading of connectors led to further distribution of the end shears; also, the main dead- and live-load shear forces were opposed to the shrinkage shears. It was also true that larger steel top flanges produced larger end shears, but such flanges were better able to withstand increases of load.

170. The Ganga roadway had not been subjected to the design live load, but there must already have been considerable temperature and shrinkage loads, and there had not, so far, been any visible cracking of the concrete slab.

171. The second disagreement could in some cases be more important, and concerned the actual shear force that the shear connectors should be designed to carry.

The Germans required that at least half the dead load should be added to the design load if less than 80% of the dead load was taken on the composite section. The Americans effectively required an increased load to be taken by the connectors because they required the factor of safety to be sufficiently high for the shear connectors to work efficiently up to ultimate-load conditions. Thus, in both cases, extra connectors were provided to ensure useful reserves of shear-connector capacity, commensurate with the reserves of the girder itself.

172. The Ganga shear connectors were designed to carry only the actual live-load shear between the concrete and the steel at stresses approximately the same as those recommended in the German specifications. But since the dead load was small, half of it would increase the shear-connector loads only marginally.

173. However, the principles behind the German and American specifications on that point must be questioned. It was not logical to provide specially for ultimate-load conditions in bridge girders, since such girders had failed, practically speaking, once their elastic capacity had been exceeded. Moreover, for some girders, the ultimate capacity was identical with the elastic capacity, yet the American specification recommended using a constant which implied that there was a constant ratio between them of about 1.5.

174. In both specifications the smallest effective increase in design loading was for composite girders which had the smallest proportion of dead load carried by the steel section alone. However, since, in that case, the top flange of the girder could be made very small, as were those of Ganga, the efficiency of the shear connectors was more important than if a comparatively large load were carried by the steel girders, because any failure of the shear connectors implied serious failure of the girders themselves.

175. The inconsistency of the German and American specifications, when compared, also called for critical examination.

On the Ganga Bridge the tee shear connectors had been welded on to the stringers by $\frac{1}{8}$ -in. fillet welds. There must have been distortions in the stringers. Were they significant and what preventive action had been taken?

176. Would the Authors give more information about the Class A and Class AA highway loading referred to in § 64?

177. Regarding the welding of mild steel (§ 117), the Indian welding code called for an increase of welds when there were fluctuations or reversals of stress. In the case of butt welds, that would automatically increase the thickness of the parent sections, but in the case of fillet welds the section was not affected; it was the opposite of normal practice in Great Britain.

178. In neither riveted nor welded construction were the main sections increased on account of repetitions of loading, although the welds, and in some cases the number of rivets, were. Also, the rules were not dependent on the number of stress reversals.

Mr T. J. Upstone (Chief Designer, Dorman Long (Bridge and Engineering) Ltd, Luton) remarked that two designs had been suggested for the bridge by the Ganga Bridge project, and a further twenty-three tenders had been submitted by competitors. Of the four tenders submitted by the successful group the Paper gave some information about three. It would be of interest to have further information concerning the other nineteen tenders.

180. It was surprising that the cost of fabrication of simple spans compared with that of cantilever spans was sufficient to offset a saving of about 600 tons in the weight of the cantilever design. Maximum sections occurred over the piers in a cantilever design and this was where the maximum sections were required for a cantilever erection scheme. A simple span design did not possess this gratuitous benefit.

181. It was stated in § 143 that the tonnage of steel in the bridge was 12,467; that was a rather conflicting figure. Allowing for the fact that the figures given in Table 1 were in respect of thirteen spans, and the bridge was built with fourteen spans, there were still about 500 tons to be accounted for. Was it additional materials, or simply over-run?

182. Approximate figures showed the dead load and live load on the bridge to be nearly equal for spans of about 400 ft. Fig. 20a showed the bridge as designed and built. Fig. 20b showed an arrangement of 397-ft suspended spans, with 82-ft cantilevers and 485-ft anchor spans, the end anchor span being a little shorter, about 441 ft. This design, which produced equal moments over the piers, and in the centres of the suspended spans, and the anchor spans would weigh little more than the simple span design. It could be built in eleven spans requiring twelve piers, as compared with the fourteen spans and fifteen piers, required by Alternative III. The pier spacings would be alternately 485 ft and 561 ft. The 485-ft spans could possibly be erected by cantilevering without additional strengthening, but if the 561-ft spans were erected as cantilevers, they would require strengthening, which would make such an arrangement of spans uneconomic.

183. However, by reducing the length of the cantilever arms to about 49 ft one could produce, as in Fig. 20c, a design of cantilever and anchor-arm construction in which the spacing of all the piers was 440 ft. This would result in thirteen spans and fourteen piers. It could be erected by cantilever principles. All the panels in this design were equal and thereby one obtained the repetition of fabrication which the Authors had emphasized in their design.

184. The fabrication cost of the design shown in Fig. 20c would compare very favourably with the design in the Paper, while the total weight would be less and there would be a saving of one pier.

185. The simple-span trusses were prestressed to reduce the deformation stresses when completed and loaded. When first erected, deformation stresses had to be induced to make the connexions. He had noted in the Paper the fact that the cantilever erection scheme increased the deformation stresses near the pier during erection. In consequence, the permissible stresses were limited to 12.5 tons/sq. in. + 25%, i.e. 15.6 tons/sq. in. If a cantilever design of trusses had been adopted and prestressed the prestressing would have reduced to a very small value the deformation stresses near the pier during erection. According to the specifications as quoted in the Paper, a permissible tensile stress of $14.6 + 25\%$, or about 18.3 tons/sq. in. could have been allowed during erection, if the small deformation stresses had been included. Surely that would have made the addition of material to the trusses for erection unnecessary. He was interested in the question of prestressing and would appreciate further information from the Authors concerning its effectiveness.

186. He liked the configuration of trusses adopted by the Authors. However, a sub-panelled Warren truss would have given the same supporting points and would have been statically determinate.

187. Mr Upstone noted that the Authors had adopted a diamond-type bracing for the main trusses and had included two additional vertical members, one of which was necessary to make the structure rigid, the other made the structure redundant. The diamond system had also been adopted for the upper lateral bracing, but no additional member had been added to make it a rigid system. With lateral load conditions unsymmetrical about the centreline of the bridge, bending moments will be induced in the chords, at their junctions with the diagonals. A single transverse member at the centreline of the bridge would have avoided such moments.

188. The designers were to be congratulated, as were the fabricators for bringing the camber out within 4% of the calculated figure. What effect had the gussets, batten plates, and cover material had on reducing the deflexions? Had this been taken into account in making the calculation? Had any allowance been made for slip in the rivet holes?

189. The girder bearings were of substantial proportions; very heavy lateral and longitudinal forces were involved, but there was only one transverse diaphragm in the grillage beneath the fixed bearing. It seemed to Mr Upstone that this was being asked to do a lot of work. The toothed plates and link bars to maintain the alignment of the four rockers of the expansion bearing were expensive machined work. Could they not

FIG. 20

have been replaced by two 1-in.-dia. dowels for each rocker screwed into the slabs and engaging holes in the rockers?

190. The height of the bridge, together with the river regime and its shifting channels, made it essential to erect the bridge from the top. A high premium was obviously placed on keeping down the weight of the forward crane. What was the actual weight of the crane, whose structural parts had been of aluminium?

191. The 75-ft receiving bracket fixed in advance to the forward pier was a useful investment, provided it was used on a number of occasions. Additional plant was necessary for its erection and it could be economical only if used several times. On the other hand it appeared that the 25-ft bracket could be put up by the erection crane on the top in the normal way, and was capable of giving considerable relief to the erection stresses.

Dr J. C. Chapman (Senior Lecturer and Deputy Head, Engineering Structures Laboratories, Imperial College) referred to the design and construction of the truss model.

193. The first step had been to make a simple model to clarify the functioning of the truss. Fig. 21 showed what happened if the two vertical end members were eliminated and the truss was taken as pin jointed. It would be noticed that with an even number of diamonds, the end diamonds were deformed in an opposite sense—one elongated and the other shortened. This would not have been the case if the number of diamonds had been odd.

194. Fig. 22 showed the model in three different stages. It was about 13 ft long and was erected horizontally on a table. One requirement was that the model should subsequently be transported to the site, where the engineers would be able to rehearse their erection procedure. It was therefore made of steel, fairly robust, and of permanent construction.

195. The model had been first erected without any camber, as a test of accuracy. The maximum deviation from straightness over the 13 ft was $\frac{1}{16}$ in.

196. Fig. 22 also showed the model erected with camber only and in the cantilevered condition. The full-scale lack-of-fit was reproduced in the model, without reduction; the linear scale was thirty, so the lack-of-fits were magnified 30 times in the model.

197. The lengths of full-scale members were specified to $\frac{1}{4}$ in., which meant that the lengths in the model had to be accurate to within about 0.001 in. The gusset plates, which were drilled for tapered pins, had to be considerably more accurate in order to get the correct angles, and they were specified to ± 0.0002 in.

198. The members themselves were circular solid steel rods. The stiffness was chosen to give the necessary extensional rigidity to the members, and the lengths and thicknesses of the springs varied. Four leaves were chosen so that a rotation would not be transmitted at the spring. In other words, the spring had the effect of introducing a short rigid length at the centre of the member. The pairs of leaves of the spring were spaced apart by sleeves, and the length of sleeve determined the lack-of-fit of the member. If the lack-of-fit was nil, the spacer was a standard 1 in. long, so the lack-of-fit was determined by the deviation of that length from 1 in.

199. Circular rods had been chosen for several reasons. One was that a wide range of diameters was available. Another was to reduce the tendency of model compression members to buckle. It had proved to be a reasonably satisfactory solution. The extensional rigidity was about 1×10^5 , the flexural rigidity scale was about 1×10^8 , and the load scale was 3,600. The load had been applied with an ordinary spring balance. This load scale meant that the loads which would have to be applied to simulate dead loading would be rather large and at first they expected that the scale would have to be amended when a major part of the span was erected, or the model members would be overstressed. Therefore, they subsequently adopted the device of incorporating the extension of the members in the lack-of-fit. In other words, they had had an amended lack-of-fit to take account of the extension of the number, which was calculated for the pin jointed truss erected to certain discrete stages.

200. The extension of the spring had given a measure of the axial load in the member,

and this had been measured simply by an inside micrometer between the pairs of leaves of the spring.

201. It was very satisfying to know that the model had actually been used on the site and that the full-scale forces were of the same order as those predicted by the model.

Mr R. E. Landau (Assistant Engineer, Mott Hay and Anderson) asked about the costs of the job and the details of protective treatment.

Mr K. A. Goodearl (Engineer, Freeman, Fox and Partners) commented on the main trusses of the bridge. The steelwork had been designed in accordance with the Indian Railways Steel Bridge Code of 1951, which, he understood, was being revised to bring it more in line with B.S.153:1958. If the tender design had been to the latter standard, about 5% of the steel weight would have been saved because of the higher stresses under wind, earthquake, and erection conditions permitted by the British Code. Also, some rather severe penalties on the effective widths of thin plates in compression had caused quite a large increase in the areas of some of the smaller chord members.

204. The final truss weight, as built, was about 6% above the tender weight, the increase being the result of agreed changes in make-up and details of construction to suit the clients' requirements which, generally, were a little more conservative than the Indian Code. The major increases were due to the alteration of box-chord sections to laced open members, and to the elimination of $\frac{3}{16}$ in. plate thicknesses in all main members. These each increased the truss weight by 1%.

205. In § 70 the economical truss depth had been given as 60 ft. The calculations actually gave the minimum steel weight at a depth of 70 ft, but because the weight of the shallower trusses was only 1–2% higher than the minimum, the 60-ft depth was adopted, for reduced erection and transportation costs would have more than offset this marginal difference in steel weights.

206. An important point was raised in § 88 regarding the fabrication of the chord members, where the distances over the backs of angles were kept constant throughout. That method of detailing increased the truss weights by only about 1 ton per truss, but resulted in appreciable savings in the cost of jigs, and in simplification of detailing.

207. The bottom lateral system was virtually fixed in plan at one end, and would transmit torsion to the piers. An alternative system would be to fix only one of the bearings at the fixed end and permit angular and longitudinal movements at the three other bearings, thus giving a freely-supported lateral truss.

The following contributions were received in writing.

Mr C. H. Partridge (Engineer, Braithwaite and Co., Engineers, Ltd) said that the successful sinking of fifteen piers without recourse to compressed air was a vindication of open grabbing as a method of pier sinking under really suitable ground conditions. Had adequate information about ground conditions been provided in the contract documents? Sometimes the information given between bed levels and founding levels for each pier was very meagre, and this was the area of prime importance to the Contractor. The clay belt near the south bank must have been a source of difficulty, particularly where the upper surface was sloping. Would the Authors comment on this? Had the contractors been satisfied that they could sink the piers without recourse to compressed air, or had they provided a full set of compressed-air plant on site in case it was required?

209. In Fig. 3, Section AA, it could be seen that the voids area was small in relation to the total cross-sectional area at the pier. How had the area under the haunches been excavated? This area was screened from the grabs and any tilting of the pier would tend to screen even more the ground under the higher part of the cutting edge. Mention was made of chisels used during excavation. How exactly had they been employed and had any other excavation plant been necessary? How had the formation been cleaned up and inspected? In under-water work of this nature it was difficult to get

FIG. 21.—MODEL OF PIN-JOINTED TRUSS, SHOWING EFFECT OF ELIMINATING TWO VERTICAL END MEMBERS

FIG. 22

accurate information unless there was an engineer available who was trained in diving. Presumably when founding level had been reached the haunches had been cleaned out and a concrete plug placed. How had the plug been placed, and how had further sinkage of the pier been prevented while the plug was being placed?

210. Presumably errors in the locations of the piers would be corrected by off-setting the pier shafts on the capping slabs, in which case errors in setting the bearings would arise only from discrepancies between fabrication and survey measurements. Would the Authors confirm this, and give information about the actual and tolerated errors in the location of the piers at capping-slab level and in the divergence from the vertical of the lower portion of the piers.

Mr E. S. Kirk (Braithwaite & Co., Engineers, Ltd) asked what method had been used to determine which were the seasons during which wind pressures of more than 10 lb/sq. ft (§§ 131 and 132) could be expected. It was stated in § 43 that the windy season was from March to May. Had there been any system of ascertaining wind pressure from day to day, or even more frequently, and if so were there any records of winds as affecting span erection?

212. Could the Authors give information on the protection of the steel; had it been painted before despatch from the fabricating shops in Calcutta, how had it been treated at the site, and how many coats of final painting had been given after erection?

The Authors, in individual replies collated by Mr Tweed, thanked Mr Kerensky for the very competitive design. The very light designs put forward had virtually eliminated the possibility of losing the work by a cheap continental tender.

214. Sir Claude Inglis's advice had been sought about the probable condition of the river-bed after the guide bund had been built and whether erection by flotation or on piled falsework was practical. His reply, summarized in § 19, indicated that cantilever erection was the only single method possible.

215. The guide bund round the north abutment was constructed with a level apron of pitching finishing at R.L.127.00 and was about 12 ft thick at the toe. This apron would fall owing to scour to an assumed slope of 2:1, making the final toe about R.L.33.5. This scouring had not taken place during the construction period.

216. In the 1956-57 season the stream varied considerably and, as stated, caused considerable trouble in pitching and sinking caissons 4-10.

217. During January to June 1958, although the level of the char had fallen there was practically no water north of span 10, so that Sir Claude's advice had been most helpful.

218. Regarding the decision to use Alternative III rather than the lighter Alternative II the simply supported truss required less than half the number of jigs for main members. Further, the cantilever design had bearings at 455 ft 7½ in. instead of 397 ft 0 in. centres. This would have increased the erection costs and almost certainly have prevented erection during the windy periods.

219. The serious tilting occurred at piers Nos 2 and 3 where clay was met after a few feet of sinking, and so was not so alarming. Further, a tilt in one direction was followed by one in the opposite direction and so the true centre of the pier was not seriously out of position. After founding the caisson was capped with a reinforced concrete slab at R.L.120. This enabled the pier to be built in its correct position. It was therefore always possible to erect the grillages for the bearings and the erection brackets in their correct positions.

220. Trusses with double-intersection web members were unusual in India and although permitted by the specification were not generally approved. However, the Authors understood that most of the Continental tenders were based on that design. Further, the shorter length plus smaller sections of web members made for cheaper fabrication and easier erection.

221. There were many variations for rail supports but assuming normal stringers and

timber sleepers the minor question of sleeper pads was, Mr Tweed considered, more a matter for the maintenance engineers than the bridge designer.

222. Mr Williams agreed that such packs were used on some bridges in India and no doubt saved in labour and other expenses involved in lifting the sleepers periodically to treat the top flanges with anti-corrosive.

223. Mr Turley referred to Mr Kerensky's remarks on the severity of the Indian Code for secondary stresses compared with specifications in other countries. Having prestressed the trusses for dead load plus live load and impact over the whole span, under British Standard rules it would not have been necessary to make further allowance for deformation stresses. Since the Indian rules did not allow any extra stress in compression members, however, the authorities had insisted on some allowance being made for conditions of loading other than live load and impact over the whole span. The figure of 0.08 for the eccentricity factor was agreed upon as being about half of the arbitrary figure of $16\frac{2}{3}\%$ used in the rules.

224. Mr Tweed observed that this treatment of deformation stresses was recommended by Johnson Bryan and Turneur in their book published in 1910 and had been used in India since 1930. From the fabricators' point of view this method was to be preferred. With parallel chords the setting out of all main gussets was the same. Further, the web covers of all the chords could be made symmetrical about both the XX and YY axes, and so could be made interchangeable to a very fine degree, so that jigs could be made for the joint holes which would turn over to almost perfect precision.

225. By starting from that point and building the jigs from the covers to the gussets and then the main members it was relatively easy to ensure a high degree of accuracy which so simplified assembly that the overall cost of fabrication and erection was less than it otherwise would have been.

226. This led to the question in § 156. To fabricate the full chord members the flange holes had to be connected to the web holes. The same degree of accuracy could not be maintained here since there was up to 0.01-in. variation in the size of 1-in.-dia. holes.

227. The ending of a chord member was measured from a machined surface set to a group of holes in the two webs. With this method it should not be possible to insert a 0.004-in. feeler in the joint between two chord members. Frequent trials were made in the shops to ensure that this accuracy was maintained.

228. At site the bottom chords were assisted by the temporary wire tie which tended to close the joint. Against this the gussets had to be riveted to the chords before erection to enable the diagonals M4-L5, etc., to be completed in the truss, before erecting the chord L5-L7. The top chords had to be held by the crane until the joints were made tight by drifting in carefully selected holes working from the top down.

229. All joints were inspected from the inside to ensure they were up to the permissible tolerance before finally bolting and riveting.

230. The model was primarily for educational purposes. Mr Tweed felt that the erection of fourteen spans which had been fabricated for prestressing by cantilevering was a sufficient novelty to justify the model. The cost spread over 10,000 tons was negligible (less than half-a-crown).

231. Referring to the jig bushes, Mr Tweed said that although for easy assembly and riveting all holes must be accurately drilled, shop rivet holes need not be so accurate as joint holes. All bushes were regularly checked with No. 90 gauges, that for shop rivet holes being $+0.010$ in. and for joint holes $+0.003$ in. Bushes for batched plates obviously were quicker than those for angles. Floor system jigs required more than chords or webs but an average of two replacements was probably generous.

232. Mr Tweed agreed that the requirement for welding in the specification was wrong. However, the railway were insisting on the same policy for the girders of the Brahmputra Bridge even though the quality of the welding on Ganga was thoroughly satisfactory.

233. The split-pin drift had been designed by Mr Tweed specially for Ganga. It

had been tested by joggling two 15-in. $\times$ 4-in. channels in the depth by suitably drilled holes.

234. The high-tensile rivets had caused some trouble at site until the riveters gained experience and confidence. Mr Williams considered the essentials were air pressure as near 100 lb/sq. in. as possible, with good heating and clean snaps.

235. Mr Tweed was glad that Mr Berridge had mentioned Mr Sethi and Mr Shahani and also the Indian riveters. He would like to add the India surveyors and Kalasis and the shop staff.

236. Mr Berridge had suggested that it might be more economical to put the road deck level with the top chords instead of near the neutral axis. Mr Tweed had heard that one alternative design did not only place the roadway at the top chord level but made the road slab to act with the trusses and so reduce the steelwork. The end spans were designed with sloping top chords to reduce the run-off.

237. The seismic effect must have added appreciably to the weight of the chords.

238. On the same subject Mr Turley thought that although this might possibly have been so when considering the bridge steelwork alone, as regards the whole project it was not an economical proposition to raise the road by 37 ft since it would have meant a considerable extra length of road embankment on each side of the river. Moreover, the 104-ft-8-in. shore spans would have had to be deepened.

239. The fixed ends of the railway stringers were riveted to the cross-girders, as was the usual practice on Indian railways, although the Authors would agree that the use of high-strength bolts for this type of connexion was an improvement.

240. There was no lubrication of the sliding surfaces at the free ends of the stringers. The stringers were bolted to the cross-girder stools with $\frac{7}{8}$ -in.-dia. turned barrel bolts having $\frac{3}{4}$ -in.-dia. threads, the barrel being $\frac{1}{16}$ in. longer than the nominal grip.

241. A cross-frame was provided between the stringers at the free end, although this was not shown on section AA of Fig. 9. The frame, however, was used only for squaring up, and the lateral load on the stringers was delivered by the stringer bracing to a bracket on the cross-girder web midway between the stringers.

242. The roadway expansion joint used between adjacent spans was shown in Fig. 23. No gutters were provided beneath the joint, and the rain-water drained on to the railway deck.

243. The welding of main fillets in plate-girder work was carried out using submerged-arc automatic machines. Braithwaites used a machine designed by their home works; Burns used a Lincoln and Jessops "Fusarc".

244. The only butt-welds were in the flanges to connect the footway brackets to the roadway cross-girders.

FIG. 23

FIG. 24

ELEVATION OF ALTERNATIVE II

ELEVATION OF ALTERNATIVE I

Scale 1 in. = 200 ft.

245. All welders were tested regularly as laid down by the specification (I.R.S. was nearly the same as B.S. in this respect).

246. The T-bar shear connexions to the roadway stringers were welded to the flange first. To overcome distortion the plates were clamped to a circular frame having a versine of 21 in. in a 44-ft girder.

247. The Authors thanked Dr Richmond for his valuable contribution on shear connectors and on welding.

248. Mr Savarkar, the only Author at present in India, said that no reports of cracking in the road slab had been received.

249. In answer to Dr Richmond's request for details of the highway loading, Class A consisted of 28-ton wheeled vehicles with a maximum axle load of 11 tons, towing two trailers of $13\frac{1}{2}$ tons each, the distance between such trains being 60 ft nose to tail. Class AA consisted of 70-ton tracked vehicles at 324-ft centres, or alternatively a 40-ton bogie of two axles at 4-ft centres. No impact was added to the highway loading in the design of the main girders, but an impact factor of $37\frac{1}{2}\%$ for Class A and 25% for Class AA was used for the floor members and connexions.

250. Mr Upstone had asked for details of the other tenders. These had not been released by the Indian authorities at the time of writing the Paper.

251. As mentioned in reply to Mr Kerensky, Alternative II was not suitable for erection during windy periods, which would naturally have increased erection costs.

252. The figure 12,467 tons included the weight of the bearings, trolley, refuges, etc. The weight of steelwork in the fourteen main spans comparable with the estimated figure of 10,655 + 195 for thirteen spans was 12,073 tons for fourteen spans, an increase of 388 tons which was due to the alterations asked for by the railway.

253. On the same subject, Mr Turley observed that Mr Goodearl had answered in part Mr Upstone's query about tonnage of steel in the bridge. Apart from the increases caused by the introduction of open-chord members and the elimination of $\frac{1}{16}$ -in. plate (this thickness was allowed by the tender documents but the clients later decided to increase the minimum thickness to $\frac{3}{8}$ in.) other changes in design and detailing after the contract was let were:—

- (a) Provision for jacking each span.
- (b) Reduction in working stress in top chords to allow for secondary stresses as mentioned in § 92.
- (c) Minor changes such as sizes of certain gussets and members, lacing, and extra diaphragms in diagonals.
- (d) Consequential increase in sections due to increase in dead load.

254. Mr Upstone had also discussed the arrangement of spans, suggesting that a cantilever design could be made more economical than a simply supported one, and had shown two alternative arrangements. However, as stated in §§ 16 and 20, it was decided early in the tender stage that any alternatives should be based on the pier designs already made by the Ganga Bridge Project, since there was insufficient information or time available for redesign of the piers. Therefore the chosen average spans had to be not greater than those of the two official tender designs, namely 600 ft and 420 ft, and it was not quite fair to state that the arrangement shown in Fig. 20c with piers at 440-ft centres would show a saving of one pier, since it was not known that the smaller pier would be accepted at 440-ft centres while the use of the larger pier would be quite uneconomic.

255. Elevations of the two cantilever and suspended-span designs, Alternatives I and II, were shown in Fig. 24. Alternative I used suspended spans almost identical with those of the accepted Alternative III. Alternative II had suspended spans of 318 ft $11\frac{1}{2}$ in. with piers spaced alternately at 364 ft 6 in. and 455 ft $7\frac{1}{2}$ in., and had almost all the features claimed by Mr Upstone for the arrangement shown in Fig. 20c, namely, erection by cantilevering and some repetition of fabrication, but it was still found to be less economical than Alternative III in spite of a saving in steel of 461 tons.

256. A cantilever and suspended-span design must necessarily have less repetition

than a comparable simple-span design, and the need to provide links and pins was also a disadvantage, both in detailing and fabrication and in erection.

257. Although Mr Upstone was correct in saying that when deformation stresses were included the allowable tensile stress could be increased to 18.3 tons/sq. in. the design must also satisfy the primary stress condition, as in all bridge specifications. It was not possible, therefore, to take advantage of the fact that in a cantilever design prestressing reduced the deformation stresses during erection.

258. H. J. Nichols¹ had written a very interesting Paper on prestressing the girders for the Nerbudda Bridge Broach. Mr Tweed had been responsible for the fabrication and erection of that bridge. The bridge carried a double-track broad-gauge railway and the girders were all mild steel. The weight of the main members had been very much heavier. The erection of all the spans had been done on falsework, the bottom chords being fully riveted before adding camber. The web members helped to bend the bottom chord. The final closing was done by pulling the end rakers down to the L0 gusset. The collision strut M1-L2 had caused difficulty in the erection if inserted before closing at L0; the end raker was too stiff to bend without, as was done then, damaging the holes in the $\frac{3}{8}$ -in. web of the bulb angles. That was the beginning of the split-pin drift.

259. Mr Tweed thought that the length of the struts for a warren truss would have made the members heavier and not so suitable for either fabrication or erection.

260. Mr Turley observed that the diamond system adopted for the top lateral bracing was rigid for loads applied to one or more intersections of the diagonal members (midway between chords), and was also rigid for equal loads applied at any pair of opposite diagonal-to-chord intersections, i.e. on each side of the longitudinal centre-line. In the top lateral bracing most of the load was applied at the web intersections, and bending in the chords was induced only by any unequal sharing of the remaining wind load between windward and leeward top chords.

261. In calculating the camber of the trusses, no allowance was made for gussets, cover material, etc., or for slip-in rivet holes.

262. With regard to the grillage beneath the fixed bearings no figure was given in the Paper for the lateral load on the bearing, although it was stated that lateral earthquake could produce a longitudinal force of 500 tons. The maximum lateral load was, in fact, 94 tons, and the single transverse diaphragm provided was quite capable of resisting this. In any case, the grillages were later encased in the pier concrete.

263. The use of two dowels top and bottom in each rocker, i.e. sixteen dowels per bearing, instead of toothed plates and link bars would have reduced the machining somewhat, but the link bars were possibly of some advantage in keeping the rollers aligned when jacking down during erection.

264. The erection crane was designed by Sir Wm Arrol to lift 15 tons at 78-ft radius with the back bogies held down to the trusses. The weight of the crane was 34 tons.

265. The Authors thanked Dr Chapman for his detailed description of the model and the reasons underlying its details. These should prove of use to Members in the future. A copy of the report and description had been given to the library.

266. Mr Williams explained that the model's closing forces were very close to the actual when the weight of the members was allowed for to reduce the applied forces.

267. In reply to Mr Landau, Mr Tweed had the following figures giving the costs of the main bridge on its foundations.

268. Of the fifteen main piers thirteen were founded at R.L. - 50.00; piers Nos 2 and 14 were founded at R.L. - 60.00.

269. The steel in the cutting edge and the first inner strake with seating ring for air dome amounted to 146 tons each. The upper strakes were 40 tons each.

270. The cost of the fifteen piers as constructed was Rs.2.76 crores (2,76,00,000).

271. The fifteen grillages under the bearing weighed 165 tons. The weight of steel in the fourteen spans was 12,072.74 tons.

272. The cost of the steelwork in the main spans supplied and erected was

Rs.2,24,20,000; the bearings supplied and erected, Rs.3,94,000; painting the steel-work after erection, two coats of red oxide, Rs.4,26,000. Reinforced concrete slab for roadway, 4.2 lacs. Reinforced concrete slab for footway, 1.3 lacs.

273. Mr Savarkar confirmed that the information from borings given to tenderers was most accurate.

274. Since the caissons were founded on sand the level depended on calculations for scour rather than on bearing pressures. These levels were not altered.

275. No information was given regarding the probable cross-section after closing the north channel.

276. It was generally expected that a stream would follow the pitching and that the apron would scour. This did not happen. Instead the big majority of the water flowed the whole season between the south bank and pier No. 10, but the bed levels varied almost every day and so gave rise to the trouble in pitching and sinking caissons in the 1956-57 season.

277. Compressed-air plant was available at site. Many engineers would have used the plant for piers Nos 2 and 3 but in practice it was not necessary.

278. In reply to Mr Partridge, Mr Savarkar confirmed that the contract documents did provide adequate information about the ground conditions. Clay belt had naturally been a source of difficulty as it always was. If the upper surface was sloping, the caisson was bound to tilt as soon as the cutting edge touched the clay strata. But then, when it pierced the clay strata all round, dewatering was usually possible and people could be sent in to excavate on the higher side only. Mainly the usual difficulty in clay strata was that the stiffness of clay varied from place to place along the cutting edge and also from time to time. Thus the wobbling of the caisson was a normal factor in clay strata, and when dewatering was not successful then the grabbing and chiselling on higher side and loading eccentric kentledge were the only remedies; and if these operations did not succeed, recourse to compressed air was the only solution. Though there was every chance of sinking all piers without recourse to compressed air, two full sets of compressed-air plant were always kept on site in readiness for emergency.

279. Mr Partridge had asked how the area under haunches had been excavated. Usually in clay strata, when dewatering was possible, manual excavation was possible and the question did not arise; otherwise, the following methods were adopted:—

- (i) By people, standing on the top of the well, pulling the wire rope of the grab to the edge of the dredge hole on the higher side by means of pulley tackle when the grab went inside the haunch. This way, it was possible to tackle an annular strip about 5 ft wide all round the dredge hole; and for portions lying beyond, it was necessary to depend upon the angle of repose of strata there, with the weight of caisson working at the cutting edge.
- (ii) By chiselling. There were several types of chisel separately improvised on different occasions. They were a cluster of rails with pointed ends; sometimes a bunch of rails was bundled together to get concentrated load effect, sometimes the rails were joined together fan shaped to give wider coverage, and sometimes a tackle was prepared by two rails, one long and the other short attached at the lower end. The short rail was bent upwards and was independently pulled up by another wire rope to cut the clay in the haunch,

280. Usually all the rail chisels were hung from such a point that they descended in a tilted fashion, so as to go inside the haunch. The rail chisels were almost dropped inside the well by means of gravity since they were lowered for the last 20-25 ft by releasing the clutch, and not lowered in slow motion by a winch.

281. Since all caissons were ultimately founded on sand, no dewatering could be done for cleaning or inspecting the formation. Further, such inspection was only necessary when strata were of hard and composite nature, and it was then necessary to see that the cutting edge was resting well all round. Since there was everywhere pure sand, no inspection was necessary. Even then, one or two trained divers were maintained. Further, since the angle of repose of sand could not be very steep, and since

almost everywhere 4–6 ft of sump was left below the cutting edge, the final shape of formation had to be such as to leave no doubt about the cleanliness of the haunch.

282. Plug concrete was placed in water by means of bottom-opening skips.

283. Study of records from surrounding weather stations over a large number of years and from an anemometer set up at the bridge site nearly 2 years before commencement of erection, indicated that wind pressures up to 30 lb/sq. in. maximum were possible in the summer months from March to May. Wind pressures more than 10 lb/sq. in. were possible from the end of January until about the middle of October. The records obtained from the anemometer at site were most useful and of great help to the site staff in planning the work, particularly during the March–June period.

284. All steelwork was given one coat of red lead paint in the shops and two coats of red oxide (Gulf Red) after erection.

285. The cost of this was approximately Rs.4 lacs or about £3 per ton.

REFERENCE

1. H. J. Nichols, "Pre-stressing bridge girders". J. Instn civ. Engrs, vol 5, p. 91 (Feb. 1937).