The reconstruction of the Grosvenor Railway Bridge

O. A. KERENSKY & F. A. PARTRIDGE

Mr F. A. Partridge

A hundred years ago the design and construction of the so-called Victoria Bridge, built in 1859 and widened in 1865, were discussed at a meeting of this Institution. The two structures were built under similar conditions to serve a similar purpose and yet were designed on totally different principles. This fact gave rise to prolonged argument concerning the validity of several of the design assumptions. There is evidence to indicate that the structures have not in fact behaved as intended.

189. The design of the new bridge is quite straightforward, consisting as it does of substantial monolithic piers supporting each of the ten tracks on a superstructure of two-pin arch construction. After initial foundation difficulties reconstruction has proceeded steadily in all weathers in accordance with a series of extremely tight time schedules. To date (April 1967) seven tracks on the downstream side are in service on new steelwork, one on the upstream side remains in use on the 1901 widening and the two intervening tracks are in the course of replacement. Reconstruction is programmed to be complete in six months time.

190. The pier settlements given in § 166 were recorded in June 1966. Since then settlement has increased more or less as predicted. The piers have a tilt, towards the downstream end, of about 1 in 5000, no doubt due to the preponderance of weight towards that end during Phase II. This should reduce by the time reconstruction has been completed.

Mr O. A. Kerensky

Mr Partridge and I have worked on this scheme from its inception to, I hope, its successful completion. All went according to plan, but took much longer than the Contractors estimated and a little longer than the Engineers anticipated! The delays were mainly due to four factors: (*a*) the very restricted possession of the railway tracks, and of the river and highway underneath; (*b*) obstructions in the foundations which were more troublesome than expected; (*c*) the leaking joint in the sheet piling, and (*d*), last but not least, labour troubles. These were of two kinds. Firstly the men were unco-operative and certain drastic measures had to be taken before good relations were established. Secondly, the normal restrictive practices which exist in the industry were especially damaging on this site. About eight skilled operations had to be carried out in certain sequences and it was almost impossible to plan them in such a way that all the skills were continuously employed. This meant waste of manpower and abortive costs. No matter who foots the bill, all of us pay in the end.

Mr A. H. Cantrell, British Railways, Southern Region

I have been responsible for the maintenance of the Grosvenor Bridge in some capacity or other ever since 1945, and for some time before the war. The Authors, § 3, state '. . . British Railways consider that such remedial measures were no longer adequate to ensure the safety of the travelling public and decided on complete replacement.' Before the decision could be made the expenditure of nearly £3½ million had to be

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justified. Repair and strengthening schemes had been considered, but there were three main factors which weighed heavily on the side of reconstruction.

193. The top section of Fig. 3 shows a continuous plate girder which stretched from one end of the bridge to the other—in all over 900 ft long. It was extremely difficult to determine what happened in relation to temperature stresses in unusually warm weather. Difficulties in the maintenance of the skew-back bearings at the ends of the arches was experienced. The unusual stress at the top was simply passed down to the bearings in an unknown way, and in really warm weather the girders twisted slightly. The second important factor concerned the cross-girder connexions on the arches near the centre of the span. From Fig. 3 the difficulty experienced where the arch rises through the continuous longitudinal girder can be seen. There were several occasions on which one of these cross-girders simply failed. It was then necessary to replace them, but due to the design this was a difficult job and the results were not entirely satisfactory.

194. The third factor, as stated in the Paper, was that the brickwork and stonework of the piers was in an extremely poor condition. Consideration had been given to pressure grouting, but analysis of the piers showed that they were not theoretically stable, and therefore anything we did would not have made an entirely satisfactory job. It would have been impossible to strengthen and stabilize the structure satisfactorily without underpinning and thickening the piers and modifying the superstructure quite considerably. The cost would have been very great. Complete reconstruction was the real answer, giving a modern well designed bridge which should be free from maintenance problems for many years. On the old bridge the tracks were carried on longitudinal timbers from one end to the other. The new bridge has a ballasted track which makes the permanent way maintenance far easier, thereby giving further considerable savings, in addition to those I have mentioned in the maintenance of the bridge.

195. Incidentally, it will be noted that instead of the nine tracks over the old bridge, there will in future be ten. This has been achieved without increasing the length of the piers but by making use of two wide-ways which had existed when there were platforms on the bridge. The tenth track will prove most useful for the greatly increasing traffic which is coming into Victoria.

196. It has been most difficult to organize the reconstruction and keep trains running without a reduction in the numbers going in and out of Victoria. Facilities had also to be provided for river traffic. I am most grateful to the Consultants, the Contractors, our own railway operating people, the signals engineer, and my own staff who have put a tremendous amount of work into this exciting engineering feat.

Mr W. J. Shirley, Marples, Ridgway and Partners

Speaking for the Main Contractor, we are now in the 209th week of construction with another 26 weeks to go before completion. Tenders for the work were submitted in March 1963. The specification for the works in relation to time cycles, giving the restrictions on track possession and the demolition sequences, etc., indicated a construction period of some 48 months. We tendered to complete in 44 months and, in the event, on completion it will have taken 54 months. There were certain major problems arising during the preliminary study of this work.

198. The first problem was the means of access for construction. The use of the existing bridge for handling materials was not allowed, and the alternative therefore appeared to be the construction of a temporary bridge across the Thames, with the same span and clearance limitations as for the new bridge. We estimated that the temporary bridge would cost some £150 000 to £200 000 to be successful, with a possible lengthening of the contract time due to the work required before the cofferdams could be begun. We decided therefore to abandon this solution, and to make use of small gaps in the present bridge for light walk-ways, to use pumped concrete and handle the bulk material by floating craft. We believe this was the correct

decision and to date we have not seen any point at which the possible advantages of the temporary bridge would have outweighed its disadvantages. Also the temporary bridge would certainly have drawn to it some of the problems that beset the works and could well have considerably increased the costs.

199. The second problem was the building of the cofferdams under the existing bridge and the works within. The Phase I operation specified that all the foundation work within the cofferdams for all the piers had to be completed before the construction of the first of the ten track bridges could be begun.

200. It was hoped to complete Phase I for all the piers in 12 months; the work actually took 24 months, although by careful rephasing the steelwork erection was begun only nine months late. The delays in this section of the work were mainly due to problems associated with existing foundations and unknown obstructions. Also certain additional restrictions were applied in the working times in the navigation spans, and night pile driving was suspended.

201. The construction method adopted for this difficult work was able successfully to overcome the problems encountered, and with a considerable sustained effort by all on the site, a reduction of some six months was made in what might have been the eventual total delay period.

202. The main point was that the programme laid down very rigidly the time cycle from the point at which the railways gave us a section of each pier for demolition until the time at which we could obtain the next slice. This time cycle required a minimum of three months' notice for the actual day and hour of possession. At the tender stage this presented a very great programming problem and it became an extremely severe taskmaster during construction.

203. Throughout the operation to complete the ten bridges, we generally had one day of float time in three months, and although we worked a network analysis for each track, every item in the network was on the critical path. Only once was the time cycle missed. In September 1965, due to inclement weather, we were faced with the risk that the loss of only 2 days meant a delay of 26 weeks. Therefore, we had to decide to rebook the date and give a voluntary delay of six weeks. In the event, weather conditions improved and the original programme could have been achieved. This is a measure of the strain under which the site staff have worked throughout this exacting job.

204. It is to the credit of all engaged in planning these operations that as an average, throughout the job one rail track bridge has been built every 24 weeks, and since foundation reconstruction, one bridge every 13 weeks.

205. The Paper describes the labour problems due to the conditions under which men had to work, which had been foreseen. However, we did not foresee the state of the labour market at the time. The programme required a great variation in labour load. For 75% of the time the men were working to maximum effort and for 25% of the time to minimum effort. This could be a major problem in labour relations and required a very even average weekly payment to hold the necessary men. This did not appear to be an insurmountable problem, but unfortunately the London Transport Board started the Victoria Tube programme in competition with the bridge, and the continuity associated with tunnel work attracted the best operators, particularly any engaged in the down time periods.

206. Subsequently, with the delay problems in the foundation works, we almost reached the stage of a Barbican crisis. This labour problem eventually eased somewhat at the later stages of the job, but nevertheless the aftermath continued throughout the work due to the nature of the uneven workload.

207. We estimated that we would construct the works with some 14 000 man weeks of effort and the job will have finally used 22 000 man weeks.

208. The rebuilding of one of the busiest railway bridges in the world is now nearing completion, and the fact that this has been done with the minimum interference to rail traffic of 1000 trains a day and to the river and road traffic, is in great

measure due to the high degree of professional skill used in the pre-planning and co-ordination of work carried out behind the scenes by the engineers, British Rail, and Local Authorities, and put into effect by all the respective site staff.

209. I should like to ask the Authors for their comments on one or two points. Did they consider a temporary bridge was necessary in their pre-planning study? Was consideration given, in conjunction with the Ministry of Transport or the London County Council, to the widening of the bridge beyond the ten tracks? Access across the Thames is so important in the communication problem and Victoria is such a vital centre that it would appear that further rail or road access could have been obtained at this site with a wider bridge at a much cheaper cost than by building another separate bridge.

210. Much research and study has gone into the question of discoloration of the concrete piers between water levels. This reconstructed bridge is still one of the most elegant structures across the Thames, but would not black piers at this level have been just as elegant and more practical in the long term?

211. The 1859 bridge lasted for a hundred years. It appears from the Paper that the new bridge is also designed for the same life span. Considering the superior materials to be used, what is the thinking on the life span required for this important bridge?

212. The pigeon problem in relation to the steelwork protection was not mentioned in the Paper. I should like to know if this was considered in the design stages, and what provision should be made on future steel bridges across the Thames, or flyovers in the London pigeon areas to minimize this problem.

Dr J. C. Chapman, Imperial College

Reference is made in the Paper to some model tests which were carried out at Imperial College in connexion with the design of the superstructure. The designers considered that, as there were 40 identical spans in the bridge, it was worthwhile to optimize design as far as possible and this they did with very great care. The first model was of perspex, one-eighth of full size, and represented part of the span. Fig. 30 shows the model.

214. The main purpose of the investigation was to examine the diffusion of strain from the arch rib into the deck, which was of course welded to the arch over the length of the intersection.

215. In this bridge there was a fatigue problem, and there was very evidently a major stress concentration. The purpose of the model was to arrive at a suitable design for stress-diffusing devices to transfer the strain gradually from the arch into the deck. The first design incorporated two triangular fillet plates, one in line with each side of the box. The shear stress in the fillet plate and the stress in the deck plate were measured and the two were reconciled. On the basis of the measurements we were able to arrive at the conclusion that a single plate would do the job.

216. Fig. 31 shows the second model, which was a half-scale model of part of the deck. In this model a number of alternative designs for the transfer of load from the cross-beams into the diaphragms in the box were examined. Some diaphragms had a stiffened rib and some did not. The outcome of this was that it was shown that the stiffener was not necessary. The edge of the deck plate where it projected through into the inside of the box rib was stiffened on one side of the deck and not on the other, and it was concluded that the stiffener was unnecessary.

217. We examined the need for haunching the cross-beams, and it was found that haunching was necessary. The haunch which was in compression was in fact stable, although outside the breadth ratio permitted by B.S. 153. We were able to investigate the load-spreading effect of the ballast and the manner in which the load was distributed between the cross-girders and stringers, according to the position of the wheel load.

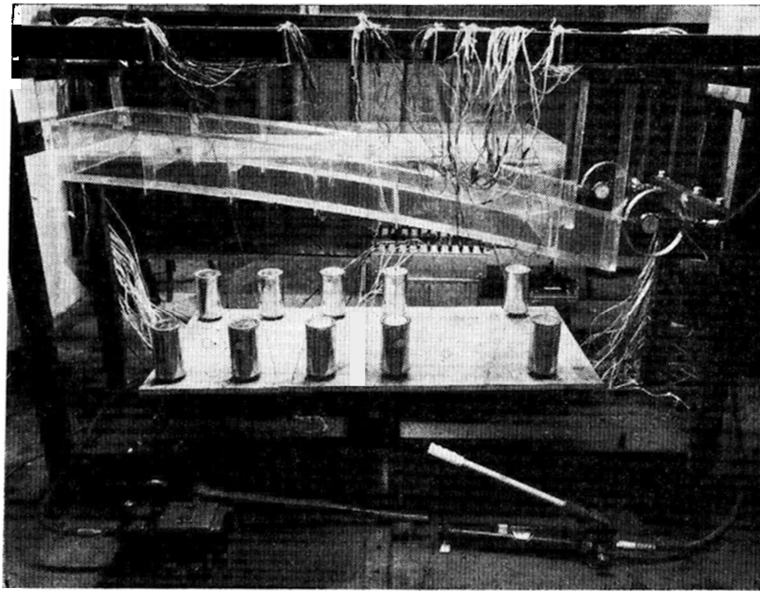

Fig. 30. Perspex model of part of span

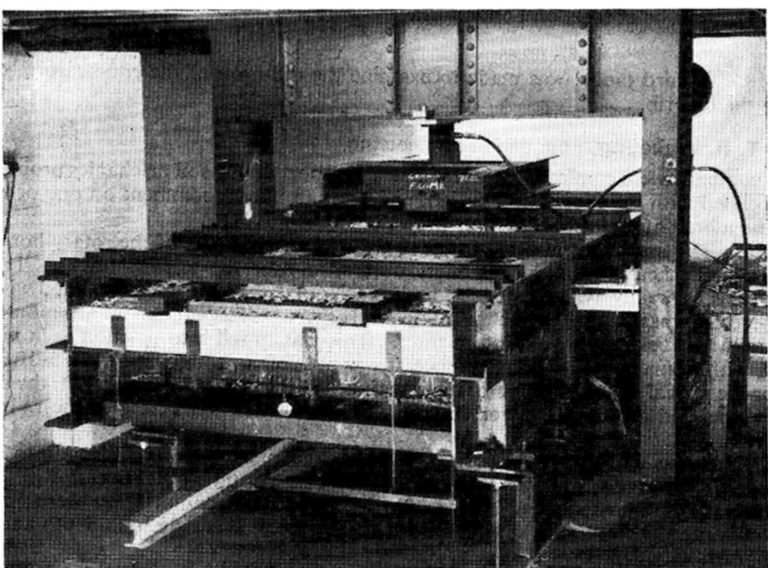

Fig. 31. Half-scale model of part of deck

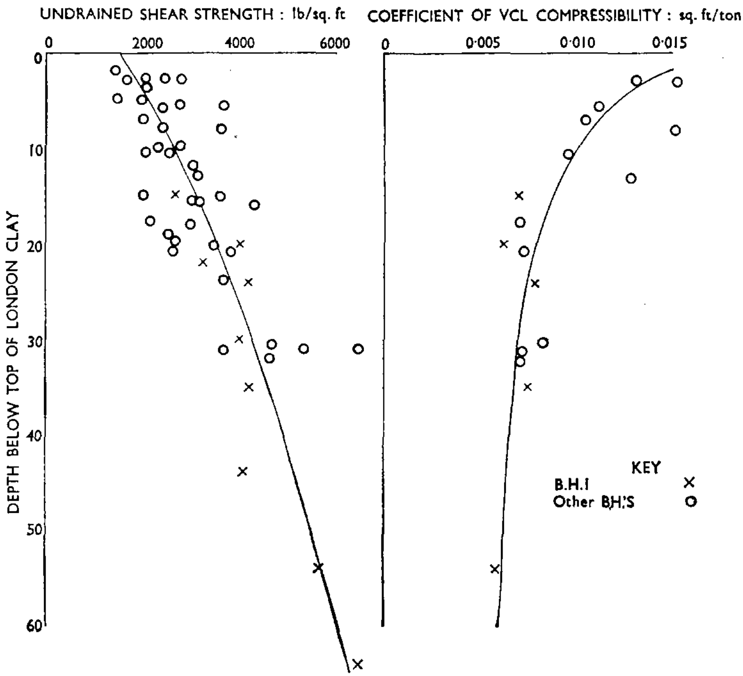

Fig. 32. Results of tests on London Clay

218. A third model was made to examine the diffusion of the end-bearing load into the box rib.

Mr T. R. Wakeling, Foundation Engineering

My company were employed to investigate and report on the soil mechanics problems associated with the foundation design, and I would like to comment on one or two of the more unusual aspects of this project.

220. The first problem arose in the site work because some of the rotary borings through the hearting of the old piers were sited over the railway tracks. These positions were only available for 3–4 hours each night. The drilling rig, fitted with a self-erecting mast, together with the pumps and other auxiliary equipment was mounted on railway trucks assembled into a train. This was towed into position each night and, with a small amount of experience, it was possible to commence work 5–10 min after the arrival of the train. For the remainder of the nine hour night shift, the drill crew operated a second rig working on boreholes which were clear of the railway tracks.

221. Some of the rotary borings through the piers were continued into the London Clay below, and were of sufficient size to permit the taking of 4 in. dia. 'undisturbed' drive samples. Other borings into the London Clay were made directly into the river bed. Fig. 32 shows the curves of shear strength/depth and of compressibility/depth.

222. When a foundation has been loaded and later unloaded, the settlement which it experiences on a subsequent reloading will be very much smaller than that

which occurred on the first application of load. The new extensions to the old foundations were on ground which had not previously been loaded, and hence the settlement characteristics of the old and new parts of the composite foundation were very different. Because of this, the major proportion of the load applied to the new composite foundation was taken on the old original part and only a small amount on the new extensions.

223. This condition was important when considering the stability of the new composite pier in respect of the overturning moment. The vertical loading of the new bridge gave foundation pressures of 0·9 tons/sq. ft and 2·4 tons/sq. ft on the extensions and on the original base respectively. This compared with 1·7 tons/sq. in. for the old bridge. The limiting conditions for the overturning moment were that the pressure at one edge of the foundation should just have become zero and that the maximum foundation pressure should not exceed the allowable bearing pressure for the subsoil. On these criteria, the low pressure on the extensions and the high pressure on the central, or old, part of the foundation gave the composite foundation a relative low resistance to overturning. Various expedients to improve this were considered and it was finally decided to increase the resistance to overturning by anchoring the sheet piles of the cofferdams into the foundations.

Mr K. A. Goodearl, Freeman, Fox and Partners

My comments are on two aspects of the project, first, the design against fatigue-damage and, secondly, painting.

225. The fatigue spectrum devised for this bridge is described in § 59. With Class F welds, application of this meant that on certain deck elements the design stresses had to be kept down to about three-quarters of those that would have been used ignoring fatigue.

226. At the time of design a transverse non-load-carrying fillet weld was classified as Class G, the type most susceptible to fatigue damage, although this detail has since been promoted to the less severe Class F. To avoid this critical detail, which could occur wherever a T-stiffener was connected to a tension flange, a special detail was developed in which the grip bolts attaching the deck plate unit to the plate girder were utilized for this additional purpose (Fig. 33).

227. It is interesting to note that this bridge, with up to 230 passenger trains a day on one track which, incidentally, must be one of the busiest in the country, would

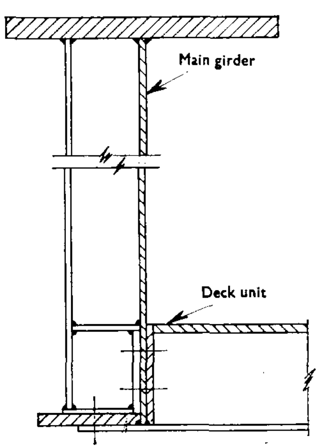

Fig. 33. Connexion of deck plate unit to girder

have had to be designed to even lower stresses if the standard British Railways design spectrum were applied. It would seem to be advisable to have at least one less severe railway spectrum, as it would appear to be uneconomic to design a bridge on a branch line carrying a dozen railcars a day to a spectrum based on 144 heavy freight trains daily.

228. Mild steel was used in preference to high yield steel, since the allowable stresses for fluctuating stresses are the same for both steels, and high yield steel has therefore no advantage when the maximum permissible stress is below 10 tons/sq. in. This is a relatively light steel deck with a severe live loading. There is no reason why high yield steel should not be used at maximum stresses on most highway bridges, and certainly on all with composite decks.

229. The only location in the Grosvenor Bridge where high yield steel could possibly have been used at higher stresses than mild steel was in the flanges of the arch ribs, but the resulting thinner flanges and reduced section properties would have meant excessive deflexion of the structure under live load.

230. Concerning the painting specification, the aim was to obtain a quick-drying priming system at the works which could be applied on consecutive days and handled the day after painting without damage. After abandoning the original system, where the red lead primers were still tacky several weeks after painting, calcium plumbate was used as the first primer, followed by two coats of metallic lead primer. This was a great improvement on the original, but in the winter months the plumbate dried slowly and was replaced by a third coat of metallic lead.

231. It was unfortunate that the use of zinc-rich epoxy prefabrication primer had to be abandoned. This primer, sprayed on before any other work was carried out on the steel, enabled plates and partly fabricated sections to be stored in the open for several weeks without deterioration. Unfortunately, this primer produces zinc oxide fumes during welding and the slight resultant health hazard led to its rejection by the welders unless safety precautions were adequate and condition money was paid.

232. The polyvinyl butryl prefabrication primer which was adopted was not really satisfactory, as corrosion quickly followed a short whiff of fresh Middlesbrough air! In the end large sections of fabricated steelwork had to be transported half-way across the town for post fabrication blasting and painting to be carried out.

233. Grosvenor Bridge provides an excellent test bed for comparison purposes. In addition to the three production systems used, an experimental painting project is being carried out as part of the Road Research Laboratory programme. Six experimental systems are compared with two basic control systems—red lead plus micaceous iron oxide, and zinc sprayed steel plus zinc chromate primer plus micaceous iron oxide. It will be interesting to analyse the results in seven or eight years' time.

Mr T. J. Upstone, Dorman Long (Bridge and Engineering) Ltd

Whilst being intimately concerned with the fabrication and erection of the steelwork, I wish to discuss some points concerning the design of the structure.

235. Hinges were placed in the arches at 164 ft centres, thus reducing the span at the expense of cantilevering out the piers. It is not immediately apparent how this has reduced the overturning moment on the piers by such a large figure as 16%, as stated. The hinges locate the arch thrust line which will be following the general line of the arch. If the hinges were, for instance, 5 ft nearer to the piers, it is difficult to see how the thrust line would be relocated so as to make such a large difference in the overturning moments. Perhaps the Authors could enlarge on their statement.

236. About $2\frac{1}{2}$ in. of pier settlement is expected at the three central piers, compared with practically nothing at the abutments. Were the levels of the structure adjusted at the piers to make allowance for some part of this expected differential settlement?

237. Each arch span relies on the secondary rocker bearings at the end cross-girders for its lateral stability. The preload applied to these bearings to avoid uplift

will be reduced with live load on the penultimate bay of the deck. Such a reduction in the vertical load on these bearings results in a reduction in the lateral forces they are capable of resisting. Would not an independent lateral bearing have given much greater security?

238. All spandrel columns, except the end ones, are simple props capable of carrying compression only. The argument used by the Designers that live load on adjacent spans produces uplift at the end bearings, would seem to apply to some of the intermediate columns also. As a number of the spans have been in service for some time, it would be interesting to know if any chatter of the spandrel columns has been noticed under the passage of a train.

239. The remark is made that on the land spans the stiffener design was altered to avoid transverse welding across the tension flange of the main girder. But the deck plate itself and its associated cheek plates are also part of the tension flange. The cheek plates are equally stressed with the girder flanges if one considers the prestressing effects of the site welding of the deck, and yet these plates have transverse welds at every cross girder connexion.

240. Turning to the pier construction, the new piers are approximately 9 ft wider than the old piers at mean water level. This means an approximate 5% reduction of water way. Do the Authors think that this is liable to lead to any problems of silting or scour?

241. In the new Grosvenor Railway Bridge, the Engineers and their advisers have achieved a structure of pleasing appearance and unique construction. I think that 100 years after H. W. Barlow passed the remark quoted by the Authors, it can be said that the Victoria Bridge is still one of the most elegant structures over the Thames.

Mr D. L. Carden, London Eastern District Engineer, British Railways, SR

I have been closely connected with the development, planning and programming of this reconstruction scheme since its conception and fully appreciate the volume of mental and physical effort that has been put into it by a large number of people. I would particularly like to congratulate the Authors for their vital role, and on the excellency of their Paper. Bearing in mind that this bridge carries one of the three main routes of the Southern Region's rail traffic into and out of London, it has been a tremendous task of planning and programming to maintain the full scheduled daily traffic over it.

243. Referring to Fig. 14, this line diagram gives the immediate impression that all the tracks were plain line, straight, parallel, and at the same level. At the north and south approaches, where each of the track diversions has to be effected stage by stage with gradients as steep as 1 in 50 and curvatures as sharp as 20 chains radius, this is far from the case, and the diagram is an oversimplification of the problems that existed. Fig. 34 shows a corrected form giving the fourteen permanent way stages actually carried out or programmed, with dates.

244. By early construction of new span and track 10 and appropriate track alterations, the South Eastern Division's traffic was able to operate over three instead of four tracks, and in the relatively slack traffic period of 23 January–23 May, 1966, this traffic was compressed to run over one Up and one Down road to help all concerned over a difficult phase of operations. After completion of the South Eastern spans No. 10 track was changed to a Down road and by utilizing the carriage sidings as an Up road, central traffic continued to operate over four roads.

245. The whole of these track alterations involved considerable signalling and conductor rail work, particularly as it was necessary to reverse the direction of traffic running at various permanent way stages. Temporary speed restrictions of 20 miles/h were on each of the roads during the whole of Phase II. The availability of roads dominated and restricted the methods of bridge reconstruction, and without doubt the overall cost has been magnified for this reason.

246. On the question of future maintenance of the new superstructure, I would be

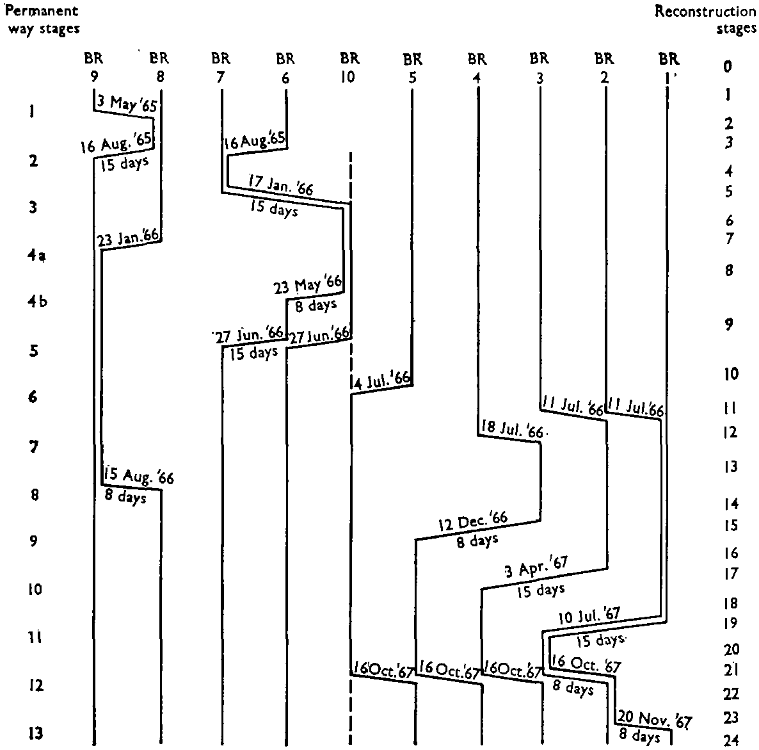

Fig. 34. Permanent way stages

interested to hear what the Authors would recommend for the specification and frequency of repainting, together with their estimated cost, based on the present Contract prices.

Mr W. J. Harper, Dorman Long (Bridge and Engineering) Ltd

Mention has been made several times in the Paper and in the discussion of the complicated nature of the Contract, and it is this complexity about which I should like to make a few remarks.

248. The basic programme for the complete demolition of one of the spans of one track and the erection of a span on the same track is relatively easily obtained, and from this basic programme a programme for the whole Contract was evolved. However, for each track this basic programme requires amendment, because of railway possessions, restriction on railway possessions during certain times of the year, restrictions on the river traffic, restrictions on the complete closure of the span for river traffic, the tides, and the effect of the presence of the new or temporary gas mains required on the bridge. All these can be theoretically obtained and put into the programme and a new overall amended programme obtained. The same exercise has been carried out by the Main Contractors and the railways. There is nothing new in this, except that this programme had to be accurate to a day, and sometimes had to be programmed six months in advance. At the time of tendering it is

virtually impossible to realize the full implications of the complications of such a contract.

249. There are two further effects of the complexity of the Contract. The first is the degree of consultation and co-operation required between the Client, Consultant and Contractor. In this case it was so obviously necessary that one can say it was forced on all. The same comments apply in even greater weight to the Main Contractor and Client. In fact, in this particular Contract the problems of the railways were as great as all the other parties put together, and the way they overcame those problems has my profound respect.

250. The second effect of the complexity of the Contract emphasized the importance of labour relations. The conditions for work in some of the areas were unpleasant, and the peculiar hours required for the work could well have aggravated minor troubles into large ones. I think it is a matter of credit to the site staff and to the unions that once a method of payment and conditions had been decided, it was adhered to throughout the Contract.

Mr L. G. Ellis, Freeman, Fox and Partners

This is the fourth time that a bridge has been built across the Thames in this position, with the same number of spans, the same rise of arch, virtually the same conditions of rail level and navigational clearances, and of similar general appearance. It may be interesting to review some of the factors relating to the earlier bridges, because there are lessons to be learnt by the comparison.

252. Fig. 35 illustrates the construction of the 1859 bridge. Operations began in September 1858 when gravel was dredged from the site of the river piers. On 10 October piling of the double walled cofferdams began and the first stone was laid at the north pier, eight months later on 9 June, 1859. The foundation was taken 8 ft into the clay and consisted of a 4 ft thick bed of 6:1 concrete. Thereafter work proceeded rapidly, with the exception of about seven weeks in the autumn of 1859 when operations were almost entirely suspended by a strike. The erection of the ironwork began on 28 February, 1860, and the bridge was opened to traffic on 9 June after a week of severe tests. The superstructure erection time, including trackwork, was 13 weeks.

253. By dividing the arch into segments, no centering was required, and the only timber necessary was a simple upright support under each segment joint. No part of the superstructure was of greater weight than could easily be lifted by ordinary travelling cranes. Stiff expansion joints were provided over the piers, but according to Sir Charles Fox, who had them watched for two years, they never moved a hair's breadth, even though there were great alterations in temperature. Even the rust between one part and another was not broken. The piers and abutments were founded on London Clay at a sufficient depth to be safe against scour, which had been so destructive to the Westminster and Blackfriars bridges.

254. The 1865 bridge took three years to complete. The piers were built on the same principle as that which had been first applied by Sir Charles Fox to the building of the bridge across the Medway at Rochester. The total time occupied in sinking a caisson averaged eight days and each caisson was preloaded with 1000 tons of kentledge for a week to prevent cracks when the work was bonded to the existing bridge. The greatest recorded movement was $\frac{7}{8}$ in.

255. The Victoria and Pimlico Railway was one of the smallest in England, being only a mile in length, but possessed a bridge which was reputed to be the widest railway bridge in the world. It was evident that the bridge, which contained a station, could be quite easily subject to live load conditions on all tracks simultaneously.

256. During the discussion on the old bridge, Mr Benjamin Baker remarked that there could be no better authority than the structure itself as to the mode in which its work was done. The present sag of the stringer girders illustrated its opinion after

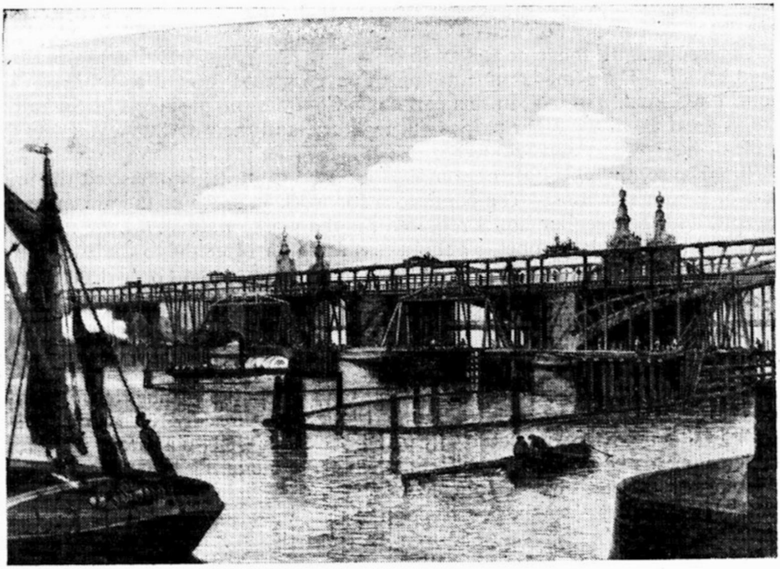

Fig. 35. Construction of 1859 bridge

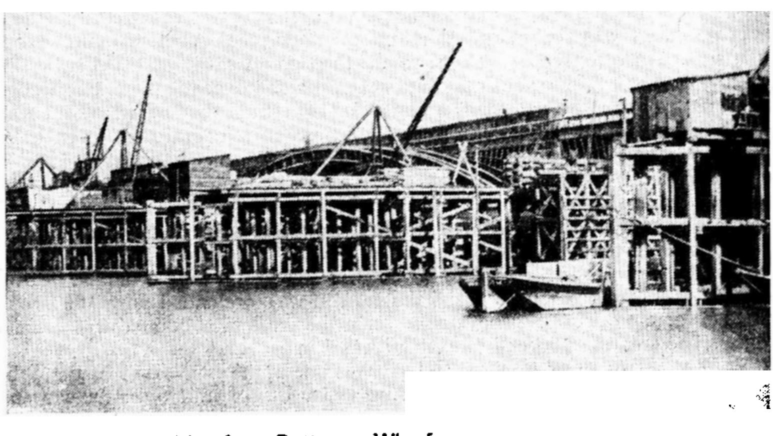

Fig. 36. 1905 bridge from Battersea Wharf

100 years. The south shore span had deflected by an amount which was more than that which could be attributed to compression of the arch. There seemed little doubt therefore that wrought iron was the correct material.

257. In order to make the ribs better able to resist the effect of unequal loading, the Engineer made the transverse strength at the centre three times its maximum strength elsewhere. This was fortuitous, because in 1906 when the south land span girders were removed, continuity with the south shore span was broken on the west side, and lateral displacement followed. The south land span girder was later buckled by fire during an air raid in October 1917.

258. The 1901 bridge was in fact built in 1905 by John Mowlem and Co. The caissons were pitched at low water and their correct alignment maintained by guide piles driven both inside and outside. The caissons were lowered by chains attached to brackets on the guide piles and the process of sinking comprised excavating beneath the cutting edge, weighing and also forcing down by jacks. With the first caisson, at the north pier, the operations were attended with some difficulty, and at the centre pier divers were employed to get down the second caisson, which sank under its own weight. In this case each caisson was test loaded with 1500 tons of kentledge. A photograph taken in April 1905 from Battersea Wharf (Fig. 36) shows the heavy temporary works, staging and overhead plant which were used.

259. There is not much to add so far as conclusions are concerned, except to note that construction time and strikes seem to be as relevant in the case of the old bridges as they are today. The introduction of live load movement at the expansion joint of the new bridge is useful because there is less chance of rusting. Both the 1865 and 1967 bridges had considerable lateral strength, thereby reducing the vibration which was so noticeable on the 1859 bridge.

260. The amount of timber used in some of the temporary works for the old bridges was excessive; 200 000 cu. ft in the case of the 1859 bridge and 286 000 cu. ft in the case of the 1865 bridge. However, little comment on constructional problems is available from papers on the earlier bridges because no contractors were represented in the discussion.

Captain D. A. Dear, Port of London Authority

From the navigational point of view, any bridge across the Thames of course is a nuisance! What is needed for navigation is maximum headroom, and maximum width between piers. When it was suggested that this bridge would be reconstructed, and that the piers would be widened, we had hopes that we would perhaps get some extra headroom. Unfortunately, because of the slope out of Victoria Station, this proved not to be to our advantage.

262. The Port of London Authority tried to make the easiest going of what could be a tough situation, and has done its best to speed up the work. But the Contractor found that the conditions imposed upon him were far more restrictive than he had anticipated.

263. The problem basically is that 2000 ton colliers, coasters, 500 ton tankers, and tugs with six barges astern, as well as passenger vessels and the ordinary service vessels, use the river at this point, and though the PLA would have liked to give as many arches as possible to the Contractor to work in it was essential to limit his working to certain periods of the tide. The closing of one of the two navigational arches is now only permitted for about 5½–6 hours out of every 12½ hours.

264. There is also the problem of the Grosvenor Dock, which is immediately upstream of the railway bridge on the north shore, where barges are winched in and out and normally have to go through the north shore span.

265. But probably the greatest worry from the point of view of the PLA is with the floating-out and floating-in operations of half spans. The pontoons were intended to take half spans having a weight in excess of 100 tons. The problem of ensuring that the half span is in a stable and upright condition, bearing in mind that

it is 18 ft above deck, is exacting. If one should topple over in the channel this would virtually close the river to all traffic.

Mr R. B. Selviah, Dorman Long (Bridge and Engineering) Ltd

I should like to congratulate the Authors on their very comprehensive treatment of this rather complicated project, and fully endorse the statement in § 165 that the programme of total operations for each bridge has required the most careful preparation and the closest collaboration of all concerned.

267. Following the demolition and erection of Bridge 9 various improvements were carried out with a view to expediting progress. In the case of demolition of this bridge the arch was supported by service girder tackle and a central section of approximately 17 ft was removed at the crown. Each half arch, weighing approximately 80–85 tons, was floated out on a pontoon. With the pontoon placed as close as possible to the cofferdam, it was still necessary to leave some portion of the steelwork attached to the piers and abutments in order to ensure that the centre of gravity of the piece being lifted coincided fairly closely with that of the pontoon. The centre of gravity was determined by weighing off at the four points of suspension from the service girder.

268. At Site B the piece was lowered on to two dolphins on a subsequent high tide, and then cut up for disposal. This method proved to be slow for the following reasons: (*a*) the difficulty of handling the pieces at the crown and at the abutments and piers, and (*b*) the floating-out operations in the navigation spans could only be carried out during week-ends when full closure periods were permissible. Moreover, the additional limitations of clearances under the old arches and the shallow depth of water in the shore spans led us to modify the methods for the interior bridges.

269. The service girder was strengthened and provided with a runway beam system carrying four 10 ton electric chain pulley blocks. It was thus possible to cut and lower the arches in nine pieces of under 32 tons each, this being the capacity of the Scotch derrick at Site B.

270. In the case of the side span, once the centre piece was lowered, demolition proceeded towards the deep water at the pier end, where the catamaran, made up of two Thames Dumb barges, was positioned. The pieces from the shore end could be transported at will on the runway beam system. In the case of the navigation spans, the use of a catamaran considerably eased the mooring problems and made it possible for demolition work to proceed during the normal daily ebb tide closure of some 5½–6 hours duration. The catamaran was capable of accommodating three pieces weighing approximately 90 tons.

271. Special full-day closures were thus eliminated and there remained the added advantage of possible acceleration by working on each successive tide if necessary. Moreover, the elimination of the pontoon obviated the necessity for fitting it with the appropriate height of trestle supports to suit the particular tide. Under the old scheme an unavoidable delay of about 2 days in carrying out the operation could easily have called for a different trestle height. Similar problems associated with the dolphins at Site B were also eliminated by lifting the pieces by the derrick and cutting them up on shore. Referring to the floating-in of the new half arches, the pontoon trestles used for Bridge 9 gave way to adjustable telescopic trestles, giving a maximum height variation of 5 ft 3 in. The trestle heights were altered by hydraulic jacks.

272. Floating-in clearances under the new and old bridges were limited and the problem became more acute as construction proceeded westwards, as the loaded pontoon had to be manoeuvred under an increasing number of completed bridges on the rising tide. The adjustable trestles thus provided greater flexibility both at Site B where the half arches were lifted off the storage gantry, and at Site A whilst passing under the finished bridges. Together with the trimming kentledge they were also used to advantage in manoeuvring over the cofferdam, during the transfer of load

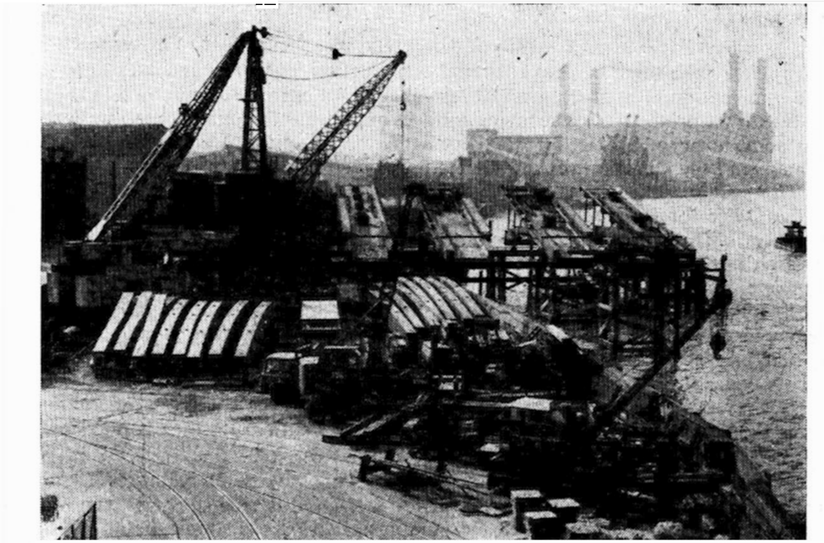

Fig. 37. General view of Site B

from the pontoon to the service girder, and thereafter for quicker release of the pontoon. All movements of the loaded pontoon under the bridges were carried out by hand-winch control as a safety precaution. For similar reasons hand winches were used on the service girder for lifting the half arches.

273. In the case of the side spans with the shallower depth of water, the loaded pontoon was towed to the downstream side of the arch on the preceding high water and temporarily moored in that position. On the following ebb tide the pontoon was gradually moved sideways into the deeper water of the adjacent navigation span, and this coincided with the normal ebb closure for that span. Soon after low water on the next flood tide the pontoon was moved back towards the side span as soon as sufficient water depth was available, and then moved directly under the bridges and to its erection position.

274. Fig. 37 is a general view of Site B showing the completed half arches on the gantry ready for erection at the Bridge site. Leaving room for the handling of demolition material from the bridge, the storage capacity for new arch ribs and deck units is very limited. With careful planning it has been possible to accommodate the new steelwork for one complete bridge, that is, 16 half arch ribs, 8 large deck units and 8 small deck units, in addition to the 8 completed half arches on the gantry and assembly positions. There are two assembly positions. The arch ribs are supported on transverse carriages at the crown end and on temporary skewbacks at the springing end. As described in the Paper, most of the welding is done in the first position, and ancillary welding and the erection of miscellaneous material carried out in the second position. When this is completed, the crown end of the half arch is lifted by the 32 ton derrick and the support trestle retracted out of the way for lowering of the half arch on to the lower gantry bogies.

275. The deck units are fabricated at Dorman Long Works, Middlesbrough, and the half arch ribs at Teesside Bridge and Engineering Works, Middlesbrough. This is therefore the first time that the various components have been put together to form a half arch. Furthermore, the two half arches come together for the first time during erection at the bridge site.

DISCUSSION

Mr A. M. Sowden, British Railways, SR

This excellent Paper gives a wealth of detail relating specifically to Grosvenor Bridge and applicable to other bridges of similar construction, but it also includes some valuable information of almost universal relevance. An example occurs in § 114, which deals with the protection of the faying surfaces of grip-bolted connexions—the decision that some form of protection was necessary, the rejection of zinc epoxy and etch primers and the adoption of zinc silicate paint.

277. I should like to comment on this by virtue of my membership of a Committee of the Office for Research and Experiments (ORE) of the International Union of Railways. This International Committee is currently sponsoring some detailed research in Stuttgart into the protection of faying surfaces. The work is far from complete, but certain preliminary findings may be of interest.

278. By comparison with the design slip factor of 0·45 quoted by B.S. 3294 for clean but unprotected surfaces, the actual coefficient of friction found for a grit-blasted, unprotected surface averaged about 0·66 (from 56 results), with a lower limit of confidence for the true mean value in excess of 0·60.

279. The application of a zinc epoxy primer reduced the coefficient of friction to below 0·25, the zinc chromate and etch primers to below 0·30, thus confirming the unsatisfactory values reported in the Paper.

280. Sprayed metal coatings and zinc silicate paint gave much better results, again confirming, to some extent, the statements of § 114. Zinc silicate paint gave values averaging nearly 0·70 and sprayed aluminium over 0·75. It should be noted, however, that these results were obtained with relatively thin coatings and there are indications that thicker coatings may give higher values, as well as better protection. Possibly, it is for this reason that sprayed zinc ($\frac{1}{1000}$ in. thick) gave such a disappointing result (0·45) which has not been confirmed by tests elsewhere. An epoxy resin adhesive gave excellent results, with an 'equivalent coefficient of friction' approaching 0·90, but this type of connexion is not a truly 'friction-grip' one, and is really beyond the scope of the ORE investigation.

281. Tests are being continued into the effects of weathering before assembly and into 'creep' under sustained static loading. Preliminary indications are that weathering may well reduce the slip factor and that zinc silicate paint is subject to slow but continuous creep under sustained loading if the latter is based on a coefficient of friction in excess of 0·5. Sprayed aluminium coatings, however, continue to give uniformly good results under sustained loading, with negligible relaxation of the clamping force in the bolts.

282. It must be emphasized that these are only preliminary results. It would be inappropriate to apply such values to future designs—especially those of railway bridges—until the investigation has been completed and has covered, in particular, the effect of fluctuating loads over a period of time.

283. They do, however, hold out very hopeful prospects for sprayed aluminium coatings, and, with hindsight, it might have been better to have used this form of protection on Grosvenor Bridge. For, at the same time, these tests warn that the good results obtained by various researchers with zinc silicate paint under short-term static loading may not be maintained under more realistic forms of loading, and they suggest reservations on the use of this material, at least for the time being.

Mr E. Phillips, Messrs Marples, Ridgway and Partners

Mr Kerensky mentioned the difficulties arising from the seal in the cofferdams and since much of the delay had been incurred in Phase 1 during construction of the cofferdams and in concreting the foundations and base, further detail might be of interest.

285. Construction of the cofferdams was originally programmed to take 28 weeks but took 71 weeks. The individual pier cofferdams took 37, 45 and 51 weeks

respectively working, south to north, even though experience gained should have reduced the time. There were three main reasons:

(*a*) inadequate passive resistance of the ground which was relied on to withstand the differential head while installing the third frame below low water level;

(*b*) the exceptional buildup of silt during the period in which the incomplete cofferdams remained tidal (at one stage the divers were wholly occupied in controlling the level of the silt and they made little headway against it in the north pier);

(*c*) greater reliance on divers to carry out work in the north pier carried out earlier by surface gangs in the south and centre pier of cofferdams. Following difficulties in installing the third frame in the earlier cofferdams it was decided to use divers to place the necessary walings with temporary struts underwater at the north pier. On initial dewatering it was found that the work was not entirely satisfactory and the installation had to be repeated by surface gangs.

286. With regard to the seal itself, various alternatives had been considered. These could be best explained by referring to Fig. 15 illustrating on one side the cross section through the original pier and cofferdam, and on the other that through the reconstructed pier.

287. Two sheet piling suppliers put forward schemes at the time of tender. One proposed an inner line of piling where the No. 3 piling is shown in the figure, with an outer concentric line of piling occupying the position of the Rendhex No. 3 fender piles but driven to a lesser depth. This scheme required one joint in each length of piling. The other proposal was to pitch and drive the piling on one line in three separate lengths, which would require two joints, one of which would lie below and one above low water in the fully driven position. Both schemes had obvious difficulties since the lowest joint must be above the level of the base concrete and the second must lie above the low water mark during construction, while still leaving enough headroom to get suitable equipment on to the extended piles for redriving. The second supplier also proposed that the lower joint should be fishplated with the plates bolted to the upper lengths of pile only. This joint would in fact have occurred just above the base concrete, below the low water mark and near the original river bed level.

288. The actual joint chosen was considered to have the most advantages. It limited construction to one joint only, enabled the driven length of piling to be pitched in one length, and ensured that sufficient headroom was available to use the various hammers within the limits of their driving ability. The arrangement also meant that maximum driving was obtained in the time available, since driving was essentially a tidal operation. To summarize, the seal was chosen from the point of view of the length of piles which could be pitched, the difficulties involved in making the joints, and their positions in relation to the permanent work.

Mr G. K. Wood, Messrs Sandberg

In the two Middlesbrough works involved in the fabrication of the steelwork, the dimensional tolerances were known from an early stage, with the result that the need to refer actual measurements back to the Client for a decision was minimized. It also enabled the Contractor to make suitable jigs in time.

290. The method adopted by Teesside Bridge Co. Ltd for the welding and handling of the large thin flimsy plates for these arches minimized damage by placing all the plates in the vertical position, supported in jigs, and by using two operators to make all the welding in this position. The resultant welding was very clean and free from slag inclusions. All welding was visually examined and 10% radiography was carried out. The number of rejections was extremely small.

291. The Dorman Long (Bridge and Engineering) works made the lighter but more complex deck structures, which being of thinner materials, were more likely to distort, although the welding was of a much lighter nature, with none of the heavy butt welds encountered in the arches. Nevertheless, distortion of the structure was small and the mating of the two main parts, arches and deck, was extremely good, particularly bearing in mind the fact that two separate manufacturers were concerned. Part of this was due to a welding sequence which shrank the welded deck before cutting to size. The welding sequence developed for the longitudinal and transverse stiffness was such that distortion of the deck plates was virtually eliminated.

292. The site welding was made using manual metallic arc throughout, the main problem being to ensure that the half arches which made up a complete span had approximately equal shrinkages to make a 'match'. This was particularly difficult before a welding sequence was established. Weld shrinkage was minimized by a blocking sequence allied with back stepping the welding.

293. Generally, the site welding was checked by spot check radiography, followed by further radiography if defects were found. Some of the butt welds were checked by ultrasonic means, when either speed of inspection was essential because of tides and delivery programme, or when the general public might be endangered from the radiation hazard.

294. Occasional difficulty was also encountered with the vertical butt welds between the stubs of the cross girders attached to the ribs and the lengths attached to the deck units. This generally arose from inadequate removal of the root of these welds, which were welded by the vertical-up method. Initially, difficulty also arose from weld cracking, because the opposite ends of stiffeners were welded after the deck had been fixed. A change in the welding sequence overcame this.

295. With regard to the temporary and permanent gas mains, these were built to high standards of workmanship including the welding, the temporary main having the same standards of inspection applied to it as the permanent main. The pipes were sufficiently large for a welder to crawl inside, and chip and seal the root, which made for easier welding of these joints.

296. Finally, the last job in the production sequence was the protective treatment. Modern grit blasting tends to blow out slugs of embedded slag or scale which before the days of grit blasting remained hidden and unknown. Nowadays a gouged surface sometimes scraps plate after plate to the consternation of all concerned. Grit blasting and prefabrication protective treatment at the mills should catch such troubles at the outset.

297. Many important points in the bridge painting have been dealt with by previous speakers. But as regards the future, there is much to be worked out on paint thickness requirements, shrinkage allowances, etc., the position being complicated when results are on the borderline of the specification. On the subject of paint durability, there is much salesmanship and theoretical reasoning as to the merits of various systems and there are many experiments with specially prepared exposure panels. What however seems more necessary is the Grosvenor Bridge type of experiment, that is, large portions of structure which have been treated under realistic site and shop conditions, thus enabling practical evaluations to be made. This sort of trial covers the usual weak spots, for example, painted items damaged in handling, painted surfaces being used as foot paths, or affected by welding, etc. We should like to see several of these large-scale trials in different areas of the country to investigate repeatability, and we are glad to say that, apart from the Grosvenor Bridge, we are concerned with a similar large-scale experiment in a more rural area.

Written contributions

Mr R. W. Turner, R. J. Crocker and Partners

In § 30, loading on the bridge is given as 'full live load on all tracks', etc. It is assumed that the live loading in this case is that given in B.S. 153, Part 3a, which is the outcome of the work of the Bridge Stress Committee. In view of the fact that the loads investigated by this Committee were those imparted by heavy steam traction, I should like to know if any consideration was given to alternative loading derived from the traffic conditions which now obtain, i.e. the use of diesel stock with high P/D ratios of wheel loading but of generally lower overall load than the heavier steam locomotives.

299. Has any facility been included in the design of the bridge to permit stresses or deflexions to be measured in working conditions so as to substantiate the loading and design assumptions made?

Dr J. E. Spindel, Assistant (Development), British Railways Board

I feel that the Authors are to be congratulated on having established a fatigue load spectrum for the rather special conditions at this bridge. It has certainly allowed them to use simple details for the design of the battledeck floor, which can only be tolerated with uneconomically low working stresses under the sort of traffic pattern assumed for the British Railways load spectrum.[4]

301. It is realized that this spectrum caters for very heavy traffic conditions and therefore includes a very large margin of safety in terms of life for the average railway bridge. However, these large factors of safety on life mean only small factors on stress. Special load spectra are therefore only justified for bridges as exceptional as the one under discussion.

302. When considering designs of battledeck floor units for normal main-line loading and the British Railways fatigue spectrum, attention has been focused on two features: the stresses at the rail bearer/cross-girder connexion, and the stresses in the deck plate.

303. Calculation has shown that the stresses at the rail bearer ends are exceedingly troublesome from the point of view of fatigue when allowance is made for the deflexion of the loaded cross-girder, which will produce a sagging bending moment in the rail bearer at the cross-girder. Floor interaction stresses add to the difficulty. These considerations led to the development of deck units with discontinuous rail bearers, i.e. rail bearers are connected by means of flexible conexions which transmit no moment.

304. The stresses in the deck plate have been determined by tests on two full-size deck units. Stresses at the weld between deck plate and longitudinal stiffening rib of the order of 5 tons/sq. in. have been found under the 20 ton design wheel load. In these tests, full-size sleepers and track were used, and the ballast under the sleepers was packed as it is in practice—only for a distance of 1 ft 3 in. either side of each running edge. This differs from the almost uniform distribution of load suggested by Clause B.5 of B.S. 153. The tests also showed that the sleeper immediately under the applied load transmitted up to two-thirds of that load to the ballast.

305. Similar tests were made on the model of the Authors' battledeck floor, which was originally tested at Imperial College. These too showed that stresses in the deck plate near the welds would be about 5 tons/sq. in. under the nominal 20 ton design wheel load. Since the weld between the deck plate and the longitudinal stiffening rib will be Class F, the permissible stresses under the British Railways load spectrum will be only of the order of 4 tons/sq. in.

306. A separate investigation carried out by British Railways, however, has disclosed that fatigue failure in the T-joint between the deck plate and the longitudinal stiffening rib will be in the parent metal and not in the weld, so that the weld can be considered Class A, which makes the stresses found quite tolerable, even allowing for some later corrosion of the deck plate. This result contradicts tests on a similar detail reported by Senior and Gurney.[5] The difference is probably due to the fact that the vertical force associated with the bending moment is very much less in the

battledeck floor case than in that of the crane girder, so that shear stresses in the weld arising from bad fit will be much lower.

307. I would like to ask the Authors whether any difficulty has been experienced with the fit-up of the bolted connexions for the floor units in the land span steelwork shown in Fig. 9. I have usually found that the welding distortion of cross-girder end plates leads to only limited contact between faying surfaces. Fatigue tests carried out by British Railways on such connexions invariably led to fatigue failure of the high-strength bolts. This danger will, of course, mainly arise near the ends of the girders, where they are restrained against free inward rotation.

Mr B. G. R. Holloway, Rendel, Palmer and Tritton

This interesting Paper describes a very fine example of reconstruction under traffic which has called for the utmost co-operation between Consultant, Client and Contractor. It is understood that a period of 12 weeks was allowed to the selected Contractors for submission of their tenders and it is stated, not unexpectedly, that the final cost will be in excess of the contract price of £2 309 341. Could the Authors give any further details as to the nature of the Contract to provide for uncertainties in relation to programme times and delays in completion and to what extent the work was discussed with the selected Contractors before tenders were issued?

309. On questions of design, it is stated in § 42 that mild steel was chosen for reasons of deflexion and fatigue. In § 60 it is stated that fatigue affected the deck structure only. Could the Authors say whether there were other reasons favouring the adoption of mild steel? The close agreement of measured and calculated deflexions is remarkable and it would be interesting to know what value of E was adopted and to what extent the deck construction affected the deflexion.

310. It is seen that future maintenance of the structure has been given careful consideration. Fig. 12 shows three cradles in position and Fig. 8 indicates the position of runway beams. Could a few words be given to describing the system in more detail particularly with reference to the method of passing at cross-girder No. 6?

311. An interesting point is raised by the difficulty in assembling correctly the south land span steelwork as described in § 146. By inference the steelwork for land spans other than for bridge No. 10 were not trial erected in the shops. In this connexion what use was made of turned parallel drifts to facilitate accurate assembly? Have any checks been made on the lateral deflexion of the top flange as induced by traffic on the rather shallow deck connexions to the main girders?

312. Considerable interest has been aroused in relation to the treatment of faying surfaces and it is noteworthy that this point received full consideration as described in § 114.

Mr Partridge and Mr Kerensky

We would like to thank the many members who contributed so much valuable information during the discussion.

314. Mr Shirley gives an excellent description of the difficulties encountered. Successful progress and completion of the work under such exacting conditions reflects great credit on the Contractors and on all the railway staff involved. However, in § 200 he states that the foundation took two years instead of one year as expected and in § 207 that the job will finally have used 22 000 man weeks instead of an estimated 14 000. Some of this extraordinary increase in time and labour can no doubt be attributed to unexpected obstructions in the foundations and to unavoidable restrictions of river and railway possessions, but much of it was due to faulty cofferdams and to labour troubles.

315. In reply to Mr Shirley's questions in § 209 concerning the use of a temporary bridge, we did not consider such a bridge to be essential. As regards the desirability

of widening the bridge to accommodate more than ten tracks, the Railway considered ten to be sufficient to cope with the then foreseeable increase in traffic. As far as the reconstruction of the bridge itself was concerned it would have been possible to accommodate eleven tracks within the available width. The unsightly gas mains, in that case, would have had to be carried in exposed positions at the sides of the bridge. At a very late stage in the development of the reconstruction scheme the idea of incorporating an overhead roadway was looked into but it was found to be quite impracticable.

316. We are not aware of any 'black' treatment of the piers that would be permanently effective but believe that the measures taken to combat discoloration will be successful.

317. In § 211, Mr Shirley's point seems to be that since the new bridge is built with materials superior to those used in the old bridge which lasted a hundred years, why should it not be designed to last longer? Firstly steel and concrete are not in all respects superior to wrought iron and masonry. The life of a structure will be shortened or lengthened according to whether the material parts, whatever their quality, are poorly or well designed and poorly or well protected. With the ever-increasing rate of technical progress there will undoubtedly come a time when means of travel will have changed so much that Grosvenor Bridge will be obsolete! It seemed reasonable, therefore, to give the bridge at least another hundred years of life. There was no more to our thinking than that. The vulnerable part of the bridge is the welded deck of one track in particular which at present carries a quarter of the total traffic and is likely to do so in the future. The deck is designed to resist fatigue damage for a hundred years and could be replaced with relative ease at the end of its life. The decks carrying the other tracks should have a much longer life. The arch ribs, with proper maintenance, could last indefinitely.

318. The pigeon problem referred to in § 212 was not considered in the design stage. It was only when the birds started fouling the freshly painted steelwork of bridge No. 9 that serious attention was given to ways and means of dealing with the nuisance. The superstructure of the old bridge provided ideal living quarters for pigeons and was in places thick with accumulated droppings. Hundreds of birds have been dislodged during reconstruction and are trying to come back. Though the new bridge offers far fewer nooks for roosting, the flanged stringers do provide convenient perches. Baffle wires alongside the stringers and almost invisible nylon netting at the up- and downstream sides of the bridge have been found to be effective in reducing the birds' activities, and their installation is being considered. It is possible that measures of a similar kind might prove useful on other bridges, but each case will have its own particular solution.

319. We would like to clear up a doubt arising from § 217 of Dr Chapman's contribution. The haunch of the cross-beam is very short and has a depth/thickness ratio which is just on the B.S. permissible limit. Since the compressive edge stress varies, along the length of the haunch, from a maximum at its join with the rib to something of little account at its other end, the stability of the plate was never doubted by us. Nevertheless the assurance provided by the tests was welcome.

320. Mr Upstone, in § 235, asks how a reduction in arch span from 175 ft to 164 ft can reduce the overturning moment in a pier by as much as 16%. He has perhaps not appreciated that, whatever the span, the arch pin is fixed at a level to be clear of high water, and the mid-point of the arch rib at a level to give requisite navigation clearance. In other words the rise of 17 ft 6 in. is fixed.

321. In § 236, he asks whether adjustment was made to the superstructure for pier settlement. To have made such adjustment would have led to unnecessary complication in the setting of the spans. The estimated settlement would produce an insignificant slope of about 1 in 1000 in the spans adjacent to the abutments. With ballasted track, rail levels are easily adjusted if necessary.

322. In § 237 Mr Upstone suggests that independent lateral bearings at the ends

of the deck would have given greater security. It was considered that the positive reactions always present at the vertical bearings were sufficient to provide against the possibility of uplift and dislodgment of the keyed rollers. If any safeguard against uplift was necessary it was provided by dead load from the adjacent unloaded span brought into play, but not assumed in design, by reason of the continuity of the welded rails across the expansion gap. With regard to the possibility of chatter of the intermediate columns during the passage of a train (§ 238), there is no tendency for uplift since the columns are always in compression, and no chatter has been detected.

323. In § 239, Mr Upstone draws attention to the fact that the land span girder cheek plates, which form part of the bottom flange, have transverse welds at every cross girder connexion and are therefore liable to fatigue damage. The stress at the very bottom of the cheek plates at these points could be critical, but any fatigue cracks developing would tend to be checked in their upward travel into an area of diminishing stress. Moreover, the cheek plate forms a relatively small part of the total flange area and damage to it would cause only minor redistribution of stress in the girder as a whole. The wide deck plate is 30% nearer the neutral axis and, the stresses in it being proportionately smaller, its transverse welds are not critical from the fatigue point of view.

324. Mr Upstone finally asks (§ 240) whether the 5% reduction in waterway will lead to silting or scour. Apparently not, since the Port of London Authority raised no objection to the wider piers.

325. Fig. 14, referred to by Mr Carden in § 243, was purely a diagram indicating the sequence of track diversions and not a plan view of the tracks. He asks for recommendations for the maintenance of the steelwork. The protective treatment given to the steelwork during reconstruction should initially last for perhaps ten or twelve years, providing that the bridge is regularly inspected and minor damage is remedied. Afterwards, most of the paintwork will require cleaning down and the outer coat of paint will need to be renewed. We would recommend that, after, say, the first ten years of inspection and touching-up, cleaning and repainting of all exposed surfaces should be repeated every seven years. With the available set of painting cradles eleven men should be able to repaint the river spans in one full season. Alternatively two men might be engaged on a permanent basis to work during the summer months. The cost would be about £10 000, or £1500 a year.

326. One of the conclusions that could be drawn from the information given by Mr Ellis is that by working harder with inferior tools our forefathers reached rates of output which compare very favourably with present day achievements.

327. Captain Dear says (§ 261) that the Port of London Authority had hopes of getting extra navigation headroom. Their hopes were in fact realized. At the time when the scheme of reconstruction was being developed, the matter of headroom was discussed with his predecessor who explained that a small gain in headroom was of more concern to the Authority than the loss of waterway occasioned by the wider piers. It was found possible to increase the headroom by 11 in., as will be noted from Table 2. Settlement of the piers may reduce this to about 9 in.

328. Referring to Captain Dear's remarks in § 262, there was a misunderstanding by the Contractor as to the meaning of the Authority's term 'flood tide'. The Contractor assumed a river possession of longer duration than the Authority could permit. This loss of time to the Contractors made it unsafe to pursue their original scheme for demolition and erection of the superstructure, and in order to speed up the operation of transferring load to and from the river a service girder became essential.

329. The improvements in erection plant described by Mr Selviah became necessary because of lack of foresight in original planning.

330. Mr Sowden gives much valuable new data on the behaviour of grip-bolted connexions, some of which confirm the results of various tests carried out especially for Grosvenor Bridge. We have no regrets over taking a conservative value for the coefficient of friction, because the effects of weathering and of repeated loadings are

not yet thoroughly known. In the light of what has been learnt during the last year or so, we would now recommend the use of sprayed aluminium coatings on surfaces in contact.

331. Mr Turner asks whether considerations were given to alternative loading. Both superstructure and substructure were designed, as he presumes, for loading specified in B.S. 153, Part 3(a). No alternative lighter loading was considered, except as noted in §§ 58–62 on fatigue. British Railway bridges are still designed to the standard loading; work on new loadings is well advanced. In answer to Mr Turner's second question, no devices were installed in the bridge for measuring stresses under working conditions. The agreement between calculated and observed deflexions, noted in § 167, was sufficient substantiation of the accuracy of live load stress calculations.

332. Dr Spindel's suggestion (§ 300) that the simple details in the design of the battledeck were made possible by the establishment of a special fatigue load spectrum is not quite correct. With the standard spectrum the deck plate and one or two other elements would merely have been slightly thicker. In reply to his question on the fit-up of the bolted connexions for the floor units in the land spans, there was difficulty at site to begin with. Welding distortions of the cheek plates at the ends of the cross-beams did prevent the parallel shank grip bolts from bringing the lower edges of these plates to within the specified 0·005 in. of the main girder web plates. After the erection of the first bridge, special measures were taken during trial erection at Middlesbrough. At those beam ends where distortion prevented fit the end welds were cut, the plates pulled together with service bolts and the welds remade. Altogether some 30% of the welds were dealt with in this way. Nothing could be done to improve the fit at each of the four rigid corners of the span except to substitute six waisted grip bolts for bolts with parallel shanks.

333. In reply to Mr Holloway's question, the final cost will exceed the contract price by (*a*) the increased cost of labour and materials under the variation of price clause in the General Conditions of Contract, (*b*) the cost of some additional works requested by the Client and (*c*) the cost incurred by the Contractor during various delays due to factors beyond his control. There was no contingency provision in the Bill of Quantities, though some provisional items for foundation work and for the demolition of the old bridge were included. Programme times were fully detailed in the tender documents and general basic information was issued well in advance to prospective tenderers. No discussions were held with them before tenders were issued.

334. As regards the decision to use mild steel, high tensile steel could only be used to economic advantage in parts of the flanges of the arch ribs. The theoretical saving would have been about £1000. This was considered insufficient to compensate for all the complications that would have arisen in design and fabrication. In the matter of deflexion under live load, mild steel had a desirable advantage.

335. In calculating deflexion, the value of E was taken as 13 000 tons/sq. in. The deck construction affected deflexion only to the extent noted in §§ 52 and 167.

336. Mr Holloway requests a more detailed description of the cradle system and its operation. Fig. 12 shows a 3-man 'transverse' cradle in the foreground and two 2-man 'longitudinal' cradles behind. All cradles are 9 ft long. The transverse cradle is suspended by chains of fixed length from two 2 ft 6 in. long carriages. The carriages are propelled along the run-way beams by handles operating sprockets geared to the carriage wheels. The cradle travels from the pier, or abutment, to cross-girder No. 6, but no further, to deal with the underside of the deck (and parts of the ribs within reach) and particularly with the tops of the flanges of the stringers (see Fig. 8).

337. Each longitudinal cradle is suspended from a 9 ft long carriage at points 6 ft apart under working conditions. Suspension is by wire ropes operated for height adjustment by two 'Strateline' winches. Auxiliary suspension chains of fixed length

are provided, one set at each end. Travel along the run-way beam is effected from the cradle by wheel-and-chain drive. The two longitudinal cradles deal with the lower parts of the ribs beyond the reach of the transverse cradle and everything beyond cross-girder No. 6. As a longitudinal cradle approaches the cross-girder, the leading 1 ft 6 in. length of carriage passes through a rectangular hole in the cross-girder web, through which the run-way beam also passes. When the leading suspension rope actually arrives at the girder the auxiliary chains are then hooked on to the ends of the carriage, one set in advance of the girder and the other behind. The cradle load is then transferred to the chains by unwinding the winches, whose ropes are then unhooked from the carriage. The cradle is now at a level to pass beneath the cross-girder, and with its carriage is propelled forward 7 ft 6 in. until the rear set of chains reaches the girder. The ropes can now be hooked on again to the carriage on the far side of the cross-girder and, by winching up, the chains can be slackened and unhooked. The cradle has negotiated the cross-girder but now has to be transferred to a run-way beam that has been set at a higher level to clear the navigation opening. For this purpose a second carriage, carried with the cradle, is installed on the higher run-way beam and the cradle is transferred to it with the use of the chains.

338. In answer to Mr Holloway's question about the trial erection of land spans, the intention was to trial-erect every span until it was deemed unnecessary to continue to do so, but welding distortion of the cheek plates of the deck units where they attached to the cross-beams prevented satisfactory bolting at site. Every span had to be shop-erected in order to make the necessary adjustments. The procedure is described in answer to Dr Spindel's query. With the use of temporary erection beams and adjustable hangers, at both the works and the site, there was no need for turned parallel drifts to fair the bolt holes.

339. Measurements taken at site show that during the passage of normal traffic over a land span the top flange deflects inwardly about $\frac{1}{16}$ in.

References

4. BRITISH STANDARDS INSTITUTE. *B.S. 153 Part 3A.* London, British Standards Institute 1954. Reprinted 1966. Appendix 'E'.

5. SENIOR A. G. and GURNEY T. R. The design and service life of the upper part of welded crane girders. *Struct. Engr*, 1963, **41** (Oct.) 301–312.