Gladesville Bridge

by

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Mr J. E. Jones (Ministry of Transport) said that the Gladesville Bridge was a very notable bridge. It was the longest reinforced concrete span which had been constructed to date, being 109 ft longer than the River Duoro Bridge of 891 ft span, built at Oporto in 1963, and 154 ft longer than the Sandö Bridge in Sweden, with a span of 866 ft.

128. He suggested that there was still quite a long way to go, as in 1928 M. Freyssinet had talked not about 1000 feet but about 1000 metres.

129. He would be very interested to know how it came about that a reinforced concrete arch of that magnitude had come to be built at all because it was not self-evident from the Paper that it was likely to be the most economical type for the particular site; the figures did not indicate that at all. He would be very interested to hear what order of comparison there was between the alternative tender and the tenders which were received for the original official steel bridge scheme. He thought it was fair to assume that the design chosen was the cheaper of the two, but he would very much like to know how it all happened. The Gladesville Bridge, as originally designed, was a multi-span affair having a navigation span which was an opening one, so it would seem that there should be no bar to having piers in the river.

130. The loading quoted in the Paper appeared to be about $\frac{1}{4}$ of the British HA standard loading for distributed load and about $\frac{1}{8}$ of the British HA value for the knife-edge load; the abnormal load prescribed for the design was less than half of the British unit or HB loading. If that was the case, he thought that the Australians could congratulate themselves that they had not had to cater for the lumps of iron and other non-ferrous metals which were hauled about the roads of Great Britain.

131. The total weight of the bridge was not mentioned in the Paper but if one took the arch ribs themselves and allowed 50 tons each for the arch voussoirs, the total structural weight in the arch ribs alone must be in the region of 26 000 tons of concrete. Compared with this, there was the Lewiston Bridge over the Niagara Gorge, a fixed steel arch also of 1000 ft span, carrying, however, only four lanes of carriageway loading, the weight of structural steelwork in the arch ribs being 3000 tons. Furthermore the Lewiston arch was also erected by the simpler method of cantilevering out on raking supports from the bank rather than on centering. In view of such a comparison, he was still left wondering why a concrete voussoir arch carried the day in the case of the Gladesville Bridge.

Mr P. Jensen (Société Technique pour l'Utilisation de la Précontrainte) said that as the Authors had pointed out the successful operation of the active joints was the

FIG. 25: JACKING DIAGRAM

main responsibility of S.T.U.P. and P.S.C. In fact, the feasibility of the undertaking depended on the flat jack, and the Gladesville Bridge was one of the very last of M. Freyssinet's great preoccupations before he died in 1962.

133. The Gladesville Bridge had often been referred to as a particularly bold undertaking. This was certainly the feeling of the Department of Main Roads when it asked the University of Sydney and the late M. Freyssinet for a second opinion. The latter had insisted that the depth of the arch should be increased by a couple of feet. As the Authors had explained, this had increased the weight of the arch by 5% and the factor of safety against buckling by about 50%.

134. However, the detailed work of checking the arch behaviour, particularly during decentering, had been left to M. Guyon, who had insisted on examining every possible combination of events so that the decentering operation could be prepared and programmed to the last detail, and any unforeseen circumstance eliminated.

135. A width/span ratio of 20 ft to 1000 ft did not leave much margin for error in alignment. The measure not to be exceeded was $1\frac{1}{2}$ in. horizontal non-alignment. But the method of active joints used allowed not only early detection but also immediate correction of any deviation from the true arch line. The essential quality of the active joint method could best be seen by comparing the apparently simple method of decentering which consisted of lowering the supporting falsework, which would have been dangerous, and the correct method, which was to raise the arch from the falsework by the slow and gradual introduction of arch thrust whilst directing its centre of gravity by means of the trimmer jacks, i.e. three jacks at each corner, each of these four groups being connected to separate and independent pressure supplies. On the jacking diagram (Fig. 25) the main jacks and trimmer jacks were represented by white and black circles respectively.

136. After completion of the jacking operation the pressure had to be transferred from oil to solid. As could be seen from the jacking diagram, the usual and simple

method of transferring the jack pressure to steel or concrete wedges placed between the flat jacks could not be used due to lack of space (local thickening of the caisson walls not being allowed for architectural reasons). Two methods of overcoming this difficulty were possible. The first was to use some set-retarded fluid, such as an epoxy resin as a pumping fluid instead of oil. The second, which was the one adopted, consisted of emptying the jacks one at a time, and replacing the oil with cement paste, which was pumped into the jack by means of an ordinary grouting pump and then pressurized using a high pressure pump. In this way the original jacking force was correctly maintained.

137. Each active joint had four layers of flat jacks. The fourth layer was provided in case of unexpected arc shortening, and also in case of any subsequent correction of the relative levels of the four arches which might be needed, prior to the final assembly by transverse prestressing. The following table summarizes a number of features particularly relevant to the active joint problem and compares the figures at Gladesville with those of the second, third, and fourth largest concrete arches, i.e. Foz do Iguacu, Arrabida, and Sandö.

TABLE

	Span (ft)	Height (ft)	Span/ height ratio	Total jacking force (tons)	Average concrete compression for dead load (p.s.i.)	Total required active joint opening (in.)
Gladesville . . .	1000	128	7.8	25 200	1760	7
Iguacu . . .	950	174	5.5	10 000	1210	
Arrabida . . .	885	171	5.2	9 000	1310	4
Sandö . . .	865	131	6.6	6 800	1170	4½

Mr S. C. Robertson (formerly Design Engineer, Bridges, Dept of Main Roads, N.S.W.) spoke of an aspect of the work not receiving attention in the Paper, namely the part played by the Department of Main Roads. The Department (then the Main Roads Board) had begun investigation for a bridge at the site over 30 years before. Much work was done on preliminary design investigation, and various types of bridges were considered, among them a reinforced concrete arch bridge with a span of about 650 ft. At that time it was estimated that a steel bridge would be about 25% less costly than a concrete bridge, and a high level bridge better than a low level one because of the high cost of maintenance of the opening span. The approximate cost of the steel high level bridge in the late thirties was about £340 000.

139. The war interrupted work and the whole problem was re-examined afterwards, but similar conclusions were reached, and a cantilever-arch steel truss bridge of the deck type was designed, for which tenders were invited. Both a design specification and a construction specification, to which tenders were required to conform, were prepared and issued, and the design specification allowed for tenders proposing alternative designs to the Department's. Among many features of interest regarding the documents might be mentioned that the Conditions of Tendering did not require a preliminary deposit with the tender. The work was of such magnitude that it was believed (rightly in the event) that only firms of considerable stature and reputation would be able to tender.

140. The Department's bridge design staff carried out very exhaustive checking work on the tendered design by the Contractors and also checked all the subsequent changes. The former Commissioner for Main Roads, Mr H. M. Sherrard, also

arranged for Professor J. W. Roderick and the staff of the Civil Engineering Department of the University of Sydney to carry out checks of the design. The conclusions reached by the University agreed substantially with those of the Department, but in one respect they were at variance. The University insisted that the concrete arch should have approximately 1% continuous reinforcing steel along its length. This was based upon the reports of the Whitney Committee in America. Mr Sherrard decided to refer this matter to M. Eugene Freyssinet, who stated that 1% continuous longitudinal mild steel reinforcement through the ribs was not necessary, but as a precaution to improve long-term stability which might be made at moderate cost, 0.1% high tensile steel could be provided if desired and placed in the top slab in the gaps between the ribs before they were concreted. He also recommended other variations in the design. The Commissioner accepted M. Freyssinet's recommendations, including some minor ones which have not been mentioned and, as indicated in the Paper, were put into effect.

141. This amount of checking was advisable in view of the quantity of materials involved. For example, there were 42 500 cubic yards of concrete, 2200 tons of ordinary steel reinforcement, and about 236 miles of prestressing tendons in the work. This necessitated the expenditure of large amounts of public money, for which the Commissioner for Main Roads, as the engineer in final authority in the matter, was responsible. Further, a large bridge was greatly in the public eye. Deficiency in design leading to repair works after the bridge was completed could scarcely be tolerated, and greater deficiencies which might lead to accident or loss of life were still less tolerable. Any doubt therefore about any aspect of design had to be resolved on the side of the greater degree of safety. The Commissioner was therefore justified in putting in hand double design checks, and having done so to accept the recommendations of the University even though they might differ from those of his own staff. It might be well to emphasize here that the Commissioner always made clear that it was not lack of any confidence in the Department's staff that led to extra checking.

142. The Paper referred to lack of check of the forms for the arch rib blocks. It was true that the forms were not individually checked but the original design was, and the blocks themselves were very thoroughly checked for dimensions, as were also the concrete mix design and the actual mixing and placing.

143. Mr Robertson also felt that though acknowledgements had been made to the Commissioners for Main Roads, past and present, concerned with the work and to Mr E. A. McKenzie, the Department's resident engineer, the Paper was incomplete without mention, in particular of the Department's bridge engineers, Mr A. J. Clinch and Mr F. C. Cook, and of Mr R. W. P. Hirt, and other engineers of the Department's metropolitan division who assisted him, Mr G. V. Fawcner, Mr A. F. Schmidt, and Mr B. J. Pearson. A full list was given in the Department's journal, *Main Roads*, for December 1964.

Professor A. J. S. Pippard (Professor Emeritus, Imperial College, University of London) said that he proposed to confine his remarks to certain aspects of the arch rib, which was the only part of the bridge with which he had had any connexion.

145. He would like to congratulate the Authors on achieving by modern computing methods a voussoir arch of 1000 ft span and a ring depth of 14 ft, whose centre line departed only $\frac{3}{8}$ in. from the theoretically true linear arch under dead loading, a matter of very great significance in relation to the behaviour of the rib under fully loaded conditions.

146. The original design of four inter-connected ribs, constructed of precast, prestressed, hollow-box concrete voussoirs, would have been a perfectly good structure if the voussoirs had simply abutted on one another without the interposition of any jointing material. Some bedding was desirable, though not for reasons of strength, but the joints should be as thin as possible, for a reason with which he would deal.

147. If the voussoirs were ideally rigid and no jointing materials were used, the arch ring as designed would under dead load be subjected to uniform compressive stress at every cross section, and the centre line would be an exact inversion of a cable carrying the same load system. The voussoirs were, however, not rigid, but were strained by the compression to which they were subjected, producing rib shortening, which unless corrected would modify the configuration of the arch and give rise to bending moments. Since the manufacture of the voussoirs was carefully controlled and their design was such that they could justifiably be assumed to behave elastically, strain would be small and calculable. Hence corrections could be pre-determined and final adjustments could be made by jacking.

148. The mortar joints, on the other hand, were more nearly plastic, and their behaviour was not so accurately determinable. The thinner they were, the less influence would they exercise on the behaviour of the structure, and the more closely would the loaded configuration conform to the desired linear arch.

149. The care taken in the manufacture of the voussoirs and in the erection and adjustment of the structure justified the Authors' major design assumption that the arches would behave as continuous elastic ribs.

150. If the ribs had been cast in situ and had been really continuous, their behaviour would not have been so accurately predictable and the need to restrict shrinkage strains would have dictated the provision of longitudinal reinforcement, which was unnecessary from strength considerations. Most of the shrinkage would occur during the maturing of the voussoirs before erection, and the longitudinal steel was not originally considered necessary by the designers.

151. In his view, this was a correct conclusion and the voussoir arches were entirely satisfactory *per se*. For all purposes they could be treated as simple ribs without the disadvantages of shrinkage which would occur in a continuous concrete structure.

152. Under live load on the bridge the linear arch would no longer coincide with the centre line of the rib and there would be a variation of compression across every section. Since the live loads were small compared with the dead weight of the bridge, the linear arch would not depart seriously from the centre line of the rib and under no combination of load and temperature effects would it fall outside the middle third core. Consequently under any loading condition the ribs were everywhere in compression, and so were effectively continuous members; voussoir construction introduced no difficulties in design assumptions.

153. Even if tension did occur, the arches would still have a considerable margin of strength. He was afraid that Rankine was responsible for the still widely-held belief that tension in a voussoir arch was catastrophic: Rankine stated¹ that the stability of arches whose centre of resistance fell outside the middle third of the depth of the ring was 'either now precarious or must have been when the mortar was fresh'.

154. While having a great respect for Rankine's genius, Professor Pippard felt that on this occasion he was demonstrably wrong: the only result of tension at a single joint might be a crack in that joint which might make it desirable to have it remade rather earlier than would otherwise be necessary, but if there were not mortar to crack it would not be possible to detect that anything had happened once the live load had passed over the bridge, because the arch, even if the joint opened completely, would behave elastically and would regain its original configuration.

155. If the live load were sufficient to displace the linear arch outside the core, a temporary 'hinge' might finally develop at the extrados, but even then ultimate failure would not result. Redundancies were merely reduced from three to two, and collapse could occur only with the formation of four such 'hinges'. If these formed alternately at the intrados and the extrados, the structure became a mechanism and

¹ The references appear on p. 257

its failure would be sudden and catastrophic. Some of his colleagues and he had demonstrated this experimentally to the Institution many years previously.²

156. The load required for the critical condition was very much greater than that which first produced tensile stress, and in the Gladesville Bridge the dead weight was so great that failure due to hinge formation was unthinkable.

157. Until tension occurred, longitudinal steel was of no value except for the control of shrinkage cracks which was not an important consideration with voussoir construction.

158. Apart from the possibility suggested by the Authors that steel might be partially stressed to compensate for rib shortening, the provision of 0.14% longitudinal steel seemed like the precaution of a man who wore a very weak belt in addition to substantial braces.

159. In all these remarks he had assumed that the abutments were effectively rigid, since no arch could stand safely otherwise. Longitudinal reinforcement would be useless in the event of abutment failure.

Mr H. J. B. Harding (Consulting Engineer) said that the *Titan* crane which had been used during the erection was very similar to the London Mammoth. To hire this 20 years ago cost several hundred pounds a day. It would be very interesting to know how much it had cost to hire *Titan*.

161. The Authors had stated that it took 11 days to erect the 800 tons of falsework but had not said how long it took to erect the main columns. Also, in connexion with the falsework, on a large bridge in Sweden some years ago this had collapsed. He wondered whether the Authors would say whether they had borne this in mind when designing the falsework, and if so, what points had they had to consider.

The following contributions were received in writing:

Mr P. D. Baker (Robert F. Farley and Partners) wrote that though the Authors had described the design, construction, and behaviour of the hollow voussoir arch ribs, and these aspects of the structure had been further enlarged upon in the contributions of other members, very little reference had been made to the design and behaviour of the foundations of the arch abutments, the stability of which was of paramount importance. It would be appreciated if the Authors could enlarge on this aspect of the bridge. Fig. 14 showed a section through the abutments along the centre line of the bridge. Would the Authors give the width of the abutments in plan? Was it the same as the width across the four ribs or was it enlarged below ground level to give a larger bearing area.

163. Could the Authors give the magnitude of the vertical and horizontal reactions on the foundations with the bridge unloaded and the maximum variations which could occur under live load? Could the Authors state what horizontal movement would be necessary to reduce the horizontal thrust under dead load by, say, 1% and would such reduction be critical to the arch? If a movement occurred of such a magnitude as to cause concern, could the remaining flat jacks at the quarter points be used to apply a correction with the completed superstructure in place.

164. Was any estimate made of the settlement and spread of the abutments which would occur and were any readings of foundation movement obtained on jacking the arch? Had any movements occurred since completing the bridge? Was any allowance made in the design of the arch for abutment movement either on an empirical or calculated basis?

Mr A. W. Shilston (Consulting Engineer) observed that the Paper was a good example of the contribution that the British consulting engineer and the British contractor could make, working in close cooperation on contracts abroad, towards earning overseas income and playing their part in the export drive. He felt that the need for imaginative working relationships would arise increasingly in the future and

that in the national interest there should be no inhibition in achieving the necessary duality of purpose, having regard to the changed position of the United Kingdom in the world scene.

166. The pattern of relationship that existed with the overseas employer at Gladesville had probably evolved as the work proceeded and had perhaps been at variance with that originally contemplated. However it would be reasonable to suppose that on completion conclusions would have been drawn as to the most desirable procedure to be adopted in contemplating a similar venture in the future.

167. He felt that the Authors from the individual viewpoints of both consulting engineer and contractor, could perform a service in setting out their views on the guide lines to be followed in the formulation of a similar contractual relationship for an overseas project. He proposed to discuss some of the issues that could influence the lines of approach.

168. It seemed likely that prospective employers would wish to impose modifications to the design incorporated in a contractor's offer and which would introduce variations on the tendered price. An offer could not be made on the basis of an I.C.E. Form of Contract in view of the position of the engineer defined therein. Nor would an overseas contract have the backing of the Common Law of Britain, with all its subtleties and inner strength. Was the lump sum type of offer most suitable, with subsequent variations negotiated as they arose at the employer's behest, or was a target type contract worth putting up? Mr Shilston thought the accounting procedure implicit in the latter made it cumbersome and probably an unhappy arrangement. Did the Authors consider it to be desirable to associate fairly detailed contract terms with an offer, or was it more expedient to treat those as negotiable once the necessary interest had been shown in the tender?

169. The possibility of the employer wishing to introduce variations raised the question of responsibility for the final design. Was it felt, in general, to be commercially inevitable for contractors to accept full legal responsibility for design even if considerable amendments had had to be made, perhaps reluctantly, to render the project viable?

170. The consulting engineer's client was the contractor to whom he owed allegiance. Design changes would no doubt result in the establishment of close direct relationships between the contractor's advisers and the employer and one would expect a considerable measure of goodwill to be built up as a consequence. *The consulting engineer's role might overflow into the financial arena in dealing with aspects of variations and so on, on behalf of the contractor.* It was not unknown in the U.K. for large contracting concerns to employ purely professional firms to deal with interim measurements and final accounts. There could be opposing views on such an arrangement and Mr Shilston asked whether on overseas work the Authors felt on balance that the consulting engineer could usefully perform that additional function.

171. It was necessary to devote considerable thought towards making comprehensive offers attractive to potential overseas customers. One way in which this might be done would be to offer, as an inclusive part of the deal, the services of a U.K. consulting engineer to act as 'engineer' for the contract once placed. In practice the nominated engineer would probably have been concerned with the design on which the contractor's offer was based but that need not be so where the design had been prepared within the contractor's own organization.

172. An example in which the consulting engineer switched sides, as it were, on an overseas contract had been the subject of a recent Paper at the Institution³; in a much publicized scheme in the U.K. recently, the contractor's consulting engineers for the submitted design were subsequently appointed as engineers for the construction stage by the employers. Such relationships might seem unorthodox but inasmuch as they were reflections of changing conditions they merited earnest consideration and discussion.

FIG. 26: PARABOLIC ARCH: COLLAPSE MOMENT ENVELOPE FOR UNIFORM LIVE LOAD

Professor R. G. Robertson (Dept. of Civil Engineering, University of Cape Town) wrote that the collapse moments for an arch of specified profile and known section properties could be found by considering the permissible maximum eccentricity of the line of thrust. The collapse moment envelopes for an arch with a parabolic profile and uniform section were shown in Figs 26 and 27, showing the effect of a uniform live load and point load respectively. The latter had been verified experimentally for a steel rib.

FIG. 27: PARABOLIC ARCH: COLLAPSE MOMENT ENVELOPE FOR POINT LOAD

174. If the live load was increased by a load factor, the eccentricity caused by live load moment must be multiplied by the same factor, until the rib cracked. After the rib cracked on the tensile side, or crushed on the compression side, the collapse moments described above could be used, modified as necessary for a variable section rib.

175. Arch ribs were subject to augmentation of moment due to deflexion, as were slender columns. This effect increased non-linearly with increased load factor and could be significant, but was not mentioned by the Authors.

176. The jacking operation (§ 39) described the benefit of jacking at the quarter span points as being due to the consideration that these 'coincided with the points of contraflexure'. The significance of the change from the usual jacking at the crown must be the reduction in the moment and jack travel required, and the release of the centering, as the desired final stresses could equally well be obtained by jacking at the crown only.

The Authors, in reply, said that they wished to begin with an apology to the Department of Main Roads for whose checking they had the greatest admiration. In New South Wales, although consulting engineers prepared designs for some highway bridges, they did not supervise their construction as this was done by the Department of Main Roads who without question did it very well. They were sorry that it had not been mentioned but there were a great many other complimentary things which could be said about the Department; for instance it did a great deal of surveying and checking on the movements of the banks. It had been thought that much of the work done by the Department, for instance in testing and quality control of concrete would have been covered by a paper prepared by an officer of the Department, but this had not proved possible in view of the limitation of space available in the Proceedings. This had led to the omission of the names of individual officers of the Department to which Mr Robertson had referred and the Authors regretted that this had been the case. They would like to state that relations between the staff and the Department of Main Roads and that of the Contractor had been excellent at all levels.

178. Mr Jones had asked about the wealth of professional advice offered and, although the speakers had explained the position to some extent, the Authors felt the history of the project should be given to show what had happened. The design was a competitive one submitted by contractors and quite rightly, had had to be submitted to a greater degree of scrutiny than might otherwise have been the case.

The Commissioner called in the University of Sydney to assist him, and the major point in the advice which he received from the University was the need for 1% continuous reinforcement. This advice resulted from reference to an American Code of Practice for reinforced concrete arches built in situ which gave this as a requirement. The arch of the Gladesville Bridge, however, had not been designed as a reinforced concrete arch and if it had, there would have been loss of section due to cracking. This would have led to an unknown loss of stiffness in the arch ribs, and would have made the calculation of the stability, which was obviously of paramount importance, uncertain and unsafe. Consequently the arch had been designed on a no tension basis. The American Code of Practice referred to by the University of Sydney was based on Whitney's work, which was for reinforced concrete design where tension was being accepted. The method of construction and the quality of concrete which had been used meant that indeterminate creep and shrinkage effects were eliminated; had the arch been cast in situ the designers would probably have included about 0.4% nominal longitudinal reinforcement.

179. This could have been argued about for a long time from the technical point of view but from the practical and contractual points of view it was vital. In order to obtain support for their views therefore the designers had asked Professor Pippard to investigate their design. The Commissioner then had the advice of Professor Pippard, who was not unknown to him, and that of the University of Sydney, and he thought that the only thing to do was to have an arbiter. Monsieur Freyssinet was therefore called in, and on the particular point of the continuous steel a very tactful solution was put forward.

180. Before leaving the question of longitudinal reinforcement, the Authors would like to make it clear that the longitudinal steel had not been put in to take account of rib shortening. They had not put it in as they did not consider it necessary, and Monsieur Freyssinet had suggested it on the argument that a failure of the arch must be a four-hinged mechanism and that by the very simple and inexpensive expedient of putting steel in the top flanges, the ultimate moment in one direction was increased. Thus of the four hinges, two had a higher plastic hinge moment and ultimate moment, though not much higher, as Professor Pippard had said. In fact the increase was 14% at the axial load, which was not a very significant increase on what was already an enormous factor of safety against failure.

181. Another major change which had resulted from the professional advice which the Commissioner had sought was the increase of the crown depth of the arch from 12 to 14 ft. It was quite true, as Mr Jensen had said, that the greater depth increased the factor of safety against buckling by 50%, but it was 50% more than something which a great many thought already adequate. The depth of the crown had been arrived at by stress analysis on an elastic basis and it was checked for vertical stability. The factor of safety against vertical instability was more than 2.5, assuming that the effective modulus of elasticity was 2×10^6 lb/sq. in., which was judged satisfactory for that class of concrete, bearing in mind the probable age of the units on loading. It was certain that the units would be at least six months old before being subject to the unsymmetrical type of loading which would cause instability. However, Monsieur Freyssinet felt from measurements taken on some of his own bridges that the effective value of the modulus could be as low as 1.4×10^6 lb/sq. in., so that a 50% increase in the stiffness of the crown was necessary if the factor of safety of 2.5 were to be maintained. It was almost certain that the effective modulus on the bridge would not drop below 2×10^6 lb/sq. in. from the readings which had been taken to date. These lay on a good curve, which could be extrapolated, and the ultimate value would certainly be greater than 2×10^6 lb/sq. in. This could not have been ascertained before the bridge was built but it was thought that Monsieur Freyssinet's figures had probably been obtained from bridges in which the concrete was not of such a high quality as that under consideration. The Authors agreed with Mr

Robertson that the arch would have looked more elegant had its crown depth been kept at 12 ft.

182. The close checking of the design by several organizations tended to produce a situation approaching 'design by committee' with the unfortunate result that the designers were obliged to take notice of the most pessimistic view on each item. For example the University of Sydney and the Department of Main Roads were satisfied with the vertical stability but Monsieur Freyssinet was not, so the depth was increased. Monsieur Freyssinet and the Department of Main Roads were satisfied without continuous reinforcement, but the University of Sydney was not and so some continuous reinforcement had been put in. As a result the bridge was somewhat over-safe. The Authors appreciated the clarity with which Professor Pippard had underlined the unlikelihood of failure of an arch in which the preponderance of axial dead weight was as great as in this bridge. Such failure was almost inconceivable provided that the transient strains were properly accounted for, as had been done by taking out the elastic rib shortening strain and then overjacking to compensate for the further creep and shrinkage. Admittedly this had been an estimate, but it appeared that the estimation was fairly accurate and that the arch would finally be very close indeed to the funicular profile. A point to be clarified in this connexion was that although $\frac{3}{8}$ in. was the maximum deviation from the true funicular profile of the trigonometrically derived centre line, this was then adjusted to the funicular profile and any deviation in the actual arch was due to erection tolerances. The Authors were quite sure that these were less than $\frac{3}{8}$ in.

183. Mr Jones and Mr Robertson had both raised the question as to whether the right type of bridge had been chosen as the solution to the problem. Mr Jones had assumed that there was no objection to piers in the river as the earlier bridge had only short spans. In fact the specification for the new bridge contained stringent requirements regarding the location of piers in the river, which in effect precluded a span less than 750 ft which was the span of the steel cantilever bridge designed by the Department of Main Roads. Cantilevering would have been possible with a steel bridge, but it was difficult to see how it could have been achieved with a concrete arch bridge. The advantages of casting the rib as a series of voussoirs were considerable since, from the construction point of view, it was possible to exercise factory-line control of dimensions, concrete and curing, not usually possible in site concreting. Site concreting would also have necessitated a considerable increase in shuttering costs or an extension of time.

184. The design had been successful in competition with alternatives to it but whether this was due to the contractors having priced the design very competitively or because it was basically a cheaper design no one would ever know. The higher tenders for the steel designs which had not been accepted had not been published. Mr Jones has said rightly, that there was about 25 000 tons of concrete in the arch. That works out at an average thickness of about 4 ft 6 in. This sounded a great deal, but was not disproportionately heavy compared with a multi-span bridge when account was taken of the fact that the columns for such a bridge would have to rise 120 ft above water and be founded about the same distance below water. In addition all the precast arch units were identical in cross section except for one dimension—the overall depth. The width and the thicknesses of the webs and flanges and of the chamfers were all constant, only the distance between the top and bottom flanges varying.

185. Mr Jones had mentioned the design loading and had inferred from the Paper that it was considerably less than Ministry of Transport loading. Although it was true that the loading taken was $\frac{7}{8}$ of H.A. or $\frac{5}{8}$ of H.A. + knife edge loading, the abnormal loading involving two 75-ton vehicles on one span gave similar results to the British loading. Aesthetically 100-ft spans had seemed about right for the deck but as it was 72 ft wide between kerbs and 88 ft overall, the spans were almost square, and the abnormal loading had dictated their design. The transverse diaphragms of

the deck were a detail which the Authors would have preferred to avoid, but without them the square span designed for abnormal loading would have been grossly over-designed for the standard loading.

186. The Authors would like to correct an impression which might have been given by Mr Jensen, that the consulting engineers had done very little in working out the jacking system. It had been very thoroughly checked by Monsieur Guyon but it was the consulting engineers who had worked it out. It should perhaps be made clear that the Freyssinet organization fulfilled two functions: it was responsible to the Commissioner for Main Roads for checking the design and advising on design matters generally; and it was also a sub-contractor for supplying and operating the jacks. It would be quite impossible to say which part of any discussion had related to one function and which to the other. There had been a great deal of discussion and, of course, on the practical sub-contracting side, there had been a great deal initiated by the Freyssinet organization. Referring to two particular points raised by Mr Jensen the Authors agreed that, as stated in § 108, there had been some difficulty in maintaining the jacking force. Second, this method, although satisfactory for a large number of jacks as used at Gladesville, had disadvantages where a small number of jacks was used, as dangerously high jack pressures could be generated adjacent to a relieved jack.

187. Referring to Mr Harding's questions the cost of hiring *Titan* for the two operations could not be directly compared. When erecting falsework it was possible to hire the crane for an extended period at a rate of £A195 per day. In the case of the main columns the necessity for jointing and stressing precluded the erection of more than two units at any one session. The charge for this work was based on a tonnage rate and the charges for towing *Titan* and the punts were extra. The erection of the permanent work for the arch, which included columns 5, 6, 7, 8, 9, and 10, and eight deck beams was carried out at the standard rate of £A4.5.0/ton. As each of these lifts required double handling and there was a quantity of heavy lifting gear involved, the actual cost was about £A11/ton erected concrete work.

188. The unfortunate incident at Sandö was not particularly significant in relation to the falsework at Gladesville. At Sandö the falsework was a timber arch and failure occurred through vertical instability due to asymmetrical loading during the placing of concrete. This could not happen at Gladesville because the falsework was supported on columns and did not depend upon thrust for its stability and, in any case, symmetry of loading was ensured because the units were cast in such a way that one could be placed on one side only immediately after placing the corresponding unit on the other.

189. Referring to Mr Baker's question, the width of the abutment in plan was the same as the overall width of the four ribs, i.e. 83 ft.

190. The reactions on the foundations were:

Horizontal reaction due to self-weight	= 31 120 tons
Vertical reaction due to self-weight	= 16 892 tons
Maximum horizontal reaction due to live load	= 1 169 tons
Maximum vertical reaction due to live load	= 656 tons

191. It was not considered essential to make an exact estimate of the probable spread of the abutments. An approximate calculation based on the value of the modulus of elasticity derived from the compression tests on the rock cores indicated a probable spread of about $\frac{3}{8}$ in., which was not only small compared to the available displacement of the jacks, but was also considerably less than the possible effect of the residual tension or compression developed in the arch before jacking by temperature strains and shrinkage in the units whilst they were supported on the falsework. It was also felt certain that any settlement of the abutments which took place would occur during jacking, when over three-quarters of the total load would be applied. Because it was impossible to separate this effect from the other strains referred to above, as they each resulted in a displacement of the origin of the graph of jack

pressure against displacement, the spread of the abutments could not be calculated by analysis of the readings taken during jacking. However, the lateral movement in plan of the first rib jacked was about 1 in. at the crown. This indicated a rotation of each abutment of about 1 in 3000 which in turn suggested an outward movement of that part of each abutment under the loaded rib of about $\frac{1}{4}$ in. Exact measurement of the spread of the abutments by normal surveying methods was obviously extremely difficult but readings taken by the Department of Main Roads on datum points situated approximately 150 ft behind the face of each abutment gave consistent results and indicated a spread of those points of just over $\frac{1}{2}$ in.

192. A horizontal movement of each abutment of about $3\frac{1}{2}$ in. would be necessary to reduce the horizontal thrust under dead load by 1%. The spare ring of jacks at each quarter point could certainly have been used to compensate for any spread of the abutments up to about 2 in., but only before the construction of the columns on the arch ribs. It was difficult to say exactly what reduction in thrust would be critical to the arch, but a reduction of about 10% would be necessary to induce tension under the action of dead load only, and it was certain that anything less than this could not be critical.

193. Replying to Professor Robertson the Authors would refer him to a paper by Gee.⁴ The analytic computer programme used in the calculation of stresses in the arch automatically took account of the magnification of the bending moments under load caused by the distortion of the arch and the elastic shortening of the rib. Being an elastic analysis, however, it could not take account of non-elastic effects which might normally occur before failure. The point here was that the live load could be factored by a more than sufficient amount, with or without factorization of the dead load, without even approaching the limits of elastic behaviour.

194. Regarding the joints between the voussoirs the Authors were of the opinion that the 3-in. joint was the minimum practicable for units of those dimensions but they would like to emphasize that the joint was not a mortar joint but was composed of 6 'K' concrete with a maximum aggregate size of $\frac{3}{4}$ in. It was a standard mix of concrete used for the prestressed concrete of the deck beams and columns.

195. Referring to Mr Shilston's contribution, the Contract was, in fact, a lump sum tender. However, in payment for the various modifications requested by the Department, the rates laid down in the Schedule were used where possible. The submission of alternatives was welcome to the Department, who had issued a separate specification designed to cover such cases. In these circumstances the procedure was laid down and variation was neither advisable nor, in the Authors' opinion desirable. The Consulting Engineers to the Contractors had been quite satisfied with the way in which things had worked out, which was as foreseen. The only function which they had been called upon to fulfil which they had not foreseen from the start was the supervision of the actual jacking operations. In this connexion it had been decided that despite the various interests involved, the direction of jacking should be the responsibility of one man and Mr Baxter had undertaken this on the principle that the jacking was in effect an application of the design process rather than a simple construction operation.

196. The Authors agreed with Mr Shilston that the target type of contract would probably have proved cumbersome in application and they would not suggest any different approach from that adopted at Gladesville. When submitting a tender it would be difficult to attempt to foresee what variations might be asked for, and any changes required should be the subject of negotiation between the submission of the tender and the award of the contract. They thought it unlikely that the type of contract proposed by Mr Shilston would be acceptable to the Department of Main Roads, or to the majority of public authorities in Australia.

197. Regarding the possibility of a consulting engineer acting for a contractor in the preparation of a tender and then being offered as the 'engineer' under the contract, they thought this unlikely to become an accepted mode of procedure either in

New South Wales or elsewhere. Most clients would be suspicious of such a proposal and the consulting engineers thought that most members of their profession would prefer not to initiate such an idea. Admittedly in the cases Mr Shilston mentioned the result had come about, but it had not been suggested in advance.

198. The question of responsibility for design was frequently extremely complex and ultimately might depend upon the view which a judge took of a particular incident. It could however be said that it seemed probable that a designer's responsibility could be lessened by amendments required by an employer. Undoubtedly the consultants role could overflow into the financial area, as Mr Shilston had suggested, but in this particular case it did not.

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