Construction of the navigation spans of the Rio Niterói Bridge, Brazil

T. J. UPSTONE & D. REILY

Mr Upstone and Mr Reily

This project is a striking example of how co-operation between the UK Government, represented by the Export Credits Guarantee Department, the financial institutions, in this case headed by N. M. Rothschild, and the construction industry can go out and win major contracts throughout the world. The UK institutions, with Government backing, provided the bulk of the finance for the entire construction, including the concrete work.

66. The bridge is one of the longest in the world. Obviously, most of it is of concrete construction. We wish to pay tribute to the skills shown by the major Brazilian contractors who formed a consortium to do most of the work, which was carried out in very difficult conditions over very deep water.

67. Virtually all the foundations are on bored tubular piles, in steel casings 1.8 m in diameter, founded at depths below the surface of the water up to, in the extreme case, 68 m, and three jack-up platforms were designed and built to erect these long piles.

68. The piers themselves were built by slip-forming. Well over 100 pairs of piers were built, the longest being 57 m clear of the water.

69. The piers are very impressive structures, especially when seen set out across the length of the bay. The construction of the superstructure as a series of balanced cantilevers was carried out using overhead erection girders to place precast concrete boxes in spans of 80 m. The individual boxes weigh up to 105 t each.

70. The bridge received very little publicity in the UK. The technical press were very kind during its construction, but we had the misfortune to open the bridge in March 1974, at the time of the three-day week and a General Election, when the press had more important things to fill their pages. It is a tragedy that the bridge received no official recognition from the UK.

71. At one stage, our two companies were building this bridge and the Bosphorous bridge and rebuilding the West Gate bridge in Australia. None of those bridges was eligible for any award in the UK.

72. The Rio Niterói Bridge did win the American Grand Conceptor Award

and it was fitting that the award was received on behalf of the joint venture by our then Chairman and the late president-elect of the Institution, Brian Hill.

Dr A. R. Flint, Flint & Neill Partnership

The Authors have described what must be one of the most ingenious schemes for constructing a large box girder bridge which has yet been carried out. The excellent erection arrangements must have gone a long way towards meeting the shortcomings of the main construction programme.

74. In the early part of the Paper the Authors comment on the use of grade 55 steel and I am interested in any problems that arose in its application. I believe that there may be little benefit to UK designers in using steels of very high yield strength, particularly in view of the difficult problems of controlling flatness and straightness and dealing with locked-in stresses. I should like to learn of the tolerances that were achieved, and whether during transport and erection some of the locked-in stresses, which must have been very high, shook out and caused changes in the shape of the bridge as a whole. There are technical arguments about whether it is better to have imperfections or residual stresses locked in. I believe that growth in imperfections and a consequent reduction in residual stress does not significantly change the ultimate strength, but might change the profile of the bridge.

75. It is mentioned in the Paper that the erection scheme had no effect on the sizes of the main sections of the girders. I would have thought that this must apply only to the side spans. I would have expected that the pontoon section, in addition to having its diaphragms strengthened to carry the local loads from the side spans, must have required strengthening to resist the water pressures on the box plating. I shall be interested to learn how that plating was designed for erection, and what sea conditions were allowed for (i.e., what local wave loadings, water pressures and buoyancy with the moments were assumed). How were stiffened plates in the floating sections analysed for strength under a combination of wave/water pressure and bending and in-plane stresses? Was there any damage caused by the storm encountered?

76. I notice that the joints in the main span were not placed at the points of contraflexure and I can only assume that this was because the contractors wished to fit the available space on land. If not, why was this the case? If the joints had been positioned at the points of contraflexure they would have saved the cost of the facilities for closing joints, even though full use was not made of the jacking capacity.

77. The energy absorption requirements for weld deposits, that were ultimately agreed at 27 J, correspond with those for about a 40 mm plate in the new draft British Standard for steel bridges, and they were probably not unreasonable. However, I question whether testing at -20°C for Rio was of any advantage. As far as I know, testing at a temperature lower than that anticipated in service only ensures that the transition temperature is not lower than the design life reference temperature. It provides no guidance as to the scatter of the notch toughness. I should like to know the background to the designers' requirements, which seem to have been unduly onerous.

78. The sequence of welding of the components was obviously carefully planned to minimize errors in camber. I should like to know whether this sequence introduced an undue cost penalty in the fabrication on the stillages. I wonder

whether the Authors considered the possibility of cooling by evaporation (i.e., spraying with water) rather than using white paint.

79. Concerning the advent of the Merrison rules, were any of the workmanship clauses applied in checking flatness and straightness, and were the strength checks used to deal with the design of panels that were out of tolerance? If they were used, was the design found broadly to comply with the rules?

80. Another point of interest is how the tolerances in alignment in level and relative level of the bearings were dealt with. It seems that the bearings were jacked up on the ring frame and set to line and level on the piers prior to positioning of the boxes. No adjustment appears to have been made to suit the box tolerances. If that was the case, what accuracy of line and level do the contractors think was achieved? How could they be sure that the boxes were bearing evenly on the bearings if no adjustment was made?

81. How was the error in alignment in the centre joint dealt with? The contractors have to be congratulated on the gap being only 5 mm. (Was that the largest gap that occurred in the bridge?) Was the gap made up by filling with weld or was it pulled together? I was not sure why the longitudinal joint in the deck was welded before the attachment of the bracing. It seems that it would have been better to have welded it afterwards, having brought the edges together.

82. The good fortune of the sun closing the joint saved jacking up with Macalloy bars, but I wonder how the contractors could have been certain that when the differential temperature disappeared, the moments at those positions would be in accordance with design. (Had they used the jacking process under conditions of no differential temperature, they would have had a measure of the accuracy of fit-up.)

83. Was the contract cost for the main span steelwork increased as a result of the delays in constructing the approach spans and piers? Did the bridge cost more to construct this way? If not, the process that was used on the Rio Niterói Bridge might well prove attractive for other sites around the world. In other circumstances, without the same exigencies of the shortcomings of the contract programme, would the Authors consider using that erection scheme again?

Mr C. D. Brown, Mott, Hay & Anderson

The most remarkable feature of the work described in this Paper is the unusual boldness of the erection scheme, together with the extraordinary accuracy and the success with which it was executed. The error in alignment of only 5 mm must be a unique achievement on a span of this size, particularly during development of welding techniques for new high strength steel.

85. The relief of stresses caused by the welding is referred to as a natural result of the erection process (§ 56). It would be interesting to know whether this relief of stress has simply been deduced from the calculated values of the erection stress, or whether measured deflexions etc., which were considerable, were such as would not have occurred without relief of welding stresses.

86. An erection scheme of this magnitude could never have been accomplished without a great deal of careful planning. My firm was asked by the contractors to make a check of the erection proposals before the commencement of work on site. The contractors spent a great deal of time and money with us on these check calculations, and this was prior to Merrison and prior to the TA1

approval system. A great deal of care was going into the erection procedures for this bridge before the box girder troubles descended on the UK.

87. One of our recommendations was to have a lower temper for the screwed rods that joined the column sections end to end. The weight of the centre section of the bridge had to be carried by these rods acting in tension. We considered that a higher impact strength than had been proposed for these rods was essential if the risk of sudden fracture was to be avoided. This involved acceptance of a lower yield strength, which was considered acceptable since the weight to be lifted was known with considerable accuracy and a large factor of safety against yield was unnecessary in this case.

88. Another proposal was that there should be some improvement in the design of the connection between the jacking gear and the tension columns, so as to surround the column completely at a level as near as possible to the jack hinge pin, providing, in case of need, a definite restraint on all four faces of the column at this level.

89. Furthermore, we proposed some redesigning of the arrangements for accepting the large angular and translational movements at the tail supports of the side span section when placing on the pontoon and, again, on the ring girder. These movements are referred to at the first position in § 43 and are implied in the second position in § 46. The redesign proposed affected the details of both of the X trestles on the pontoon and the extra stiffening of the side span unit at its point of temporary support so as to make full allowance for the movement of that point. The problem at the ring girders was to allow them to rotate to meet the change in profile of the side span unit, while maintaining control.

90. Another point raised concerned the possibility of aerodynamic instability during erection, particularly for the centre span unit. Each unit would take several days to erect and temporary aerodynamic dampers were recommended.

91. The lifting weight of the centre section is shown as 3800 t in § 29, but as 3420 t in § 56. This seems to imply that it was carrying nearly 400 t of additional material at the time of first lifting.

92. May I ask what were the acceleration measures referred to in § 38 which cut four months off the programme?

93. In similar projects on which I am currently engaged, we are contemplating the use of heavy floating cranes, mainly developed for offshore processes in the North Sea. I should be interested to know whether this type of erection, which was not readily available during the building of the Rio Niterói Bridge, would be of interest nowadays.

94. I think that it is worthwhile to draw attention to some of the papers dealing with the whole of the project, which is a record-breaking structure and is still the largest box girder bridge in the world. As far as I am aware, a number of problems in the structural design were dealt with by Kuzmanovic¹ in a paper which, although entirely concerned with the bridge, never actually mentions it by name. A more general description is given by Graham² of Howard Needles and a useful account of the site-measured differential temperatures is given by Ostapenko.³ There is a thorough study of the stresses in the orthotropic deck during erection and in service by Daniels *et al.*,⁴ who conclude that only the site splices of the stiffeners constitute a fatigue problem. There may be other references and I think that it would add to the completeness of the Paper if they could be added to the report of the discussion.

Mr D. C. C. Dixon, Carthorpe Consultants

On this contract we had two things—a buyer's credit (which is always useful when trying to negotiate a large contract) and partners. Over a period of 5–6 months, I was there for most of the time with John Harper on behalf of RDL. We had to strike up relations both with our partners and with the customer during the process of negotiation and settling the contract.

96. We had a clear understanding of what we all meant, but we had to put it into words and in English. The contract was in Portuguese, and one could invariably get two or three translations meaning different things. It was important that the people negotiating should understand each other.

97. In five months we achieved understanding and we achieved the words. The combination of the words and the written contracts were adhered to completely the whole way through.

98. With this daring erection scheme of lifting vast pieces through a considerable number of metres, I never liked the fact that it was incapable of a totally fail-safe procedure. Basically, the reason why Mott, Hay & Anderson were retained was because we wanted them to relieve our fears on this score. They made a long report of great competence (needless to say) and they made various recommendations, all of which were adhered to.

99. In the end we had a scheme where the odds against a disaster such as a sniper's bullet hitting one valve in the hydraulic system from, say, 500 yd away was 10^7 or 10^8 . Personally, I am happier with a risk of 10^4 and a fail-safe system for a risk of 10^4 . Perhaps that is an indication of one's frailties.

100. In the company that I was running at that time, we had (as have all good fabrication companies) sets of tables for the pre-set that should be allowed to plates. The easiest example is the plate girder, where flanges are welded to the webs and pre-set to a certain degree in order that when welding is completed the contraction shall have resulted in the flange being level. In the case of grade 55E, the amount of pre-set required was exactly double what was required for grade 50 and this was observed mostly in our blacksmith's shop where the work was done. It was clearly recorded by the blacksmith who did the work and I have heard no explanation for it. There seems to be no connection between the factor of 2 in the pre-set and the modulus or tensile strength or any other property of the steel that we could discover. I shall be interested to know if any such explanation has been found for this peculiar phenomenon.

Mr C. D. Crosthwaite, Freeman Fox & Partners

This was the biggest box girder bridge ever built, and it got practically no recognition in the UK at all. Why was that? Was it because it was designed by a US firm? Is that why we were not prepared to give any awards? Does a bridge have to be designed by a UK consulting engineer before it can have an award from this country?

Mr A. J. G. Sturgeon, Pauling & Co. Ltd

I should like to know the value of the contracts involved as a whole, and the value of the steel contract. I am also interested in the steel tonnage and

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unit price per ton erected. I should value a summary of how the programme fitted together, from award of contract to mobilization, fabrication and erection, and how acceleration measures were incorporated.

Mr E. C. Hambly, Consulting Engineer

Can the Authors make any comment on their competition and how the different financial arrangements of the competition affected their competitiveness?

Mr B. W. Smith, Flint & Neill Partnership

I wonder whether during or subsequent to the erection of either the centre section or the side spans there were any aerodynamic movements which were significant in any way. These may have been expected, considering the very low damping of such an all-welded structure.

Mr Upstone and Mr Reilly

In reply to **Dr Flint's** question concerning the flatness tolerances of grade 55 steel, the tolerances from the Merrison committee work as they then existed were used as a guide. A few distortions exceeded the Merrison tolerances but generally they were within the specified figures. As a result of the Merrison work on box girders which was going on at that time a very close check was made of some of the plate panels. Some of the curved panels in the girder haunches were stiffened as a result.

106. At the time the erection design checks were being carried out the Merrison rules were in a continual state of flux, and the design strength checks on stiffened panels were carried out using other procedures.

107. Dr Flint mentions the shake-out of residual weld stresses. There was a weld problem which was thought to be both time- and stress-dependent. To investigate this, stiffened plate panels were loaded on a lorry and driven through Durham and North Yorkshire to see if the shaking and vibration of such handling had any effect. This treatment certainly did not have any significant effect on the panel shape as a result of residual stress shake-out.

108. Some camber in the girders was lost due to a number of causes, the main one being the big difference between day and night temperatures of the girders on the assembly stillages. Some camber may have been lost due to shake-out of residual welding stresses but no detail measurements were made which would differentiate between the various causes.

109. Paragraph 28 describes the strengthening required for the bridge section used temporarily as the pontoon. The side and bottom platings of the pontoon were checked for the hydraulic pressures, both external and internal, the former including the effect of waves of appropriate height and pitch, and these forces were combined with those arising from the pontoon carrying the side span ballasted in selected tanks to zero freeboard.

110. The positions of the points of contraflexure of the main span change their positions depending on the loading conditions and are even affected by temperature variations. Thus it is not usually possible to ensure that erection joints are at these points. It is therefore necessary to provide temporary means of inducing moments in the girders at the erection joints. At the time of the erection of the centre piece only one of the 44 m spans had been erected. This lack of symmetry also moved the points of contraflexure. It was necessary

to provide the screwed rods and jacking gear first to pull the joint together and secondly to hold it in a position despite the changing position of the point of contraflexure long enough to bolt up the permanent joint.

111. As stated in § 24, notch toughness was specified by the Engineer as 41 J at -20°C . The background to these specified values was never understood by the contractor. The requirement was considered by him to be unduly harsh and in fact proved very difficult to achieve. As a result of discussions with the Engineer, the requirement was reduced to 27 J at -20°C . Dr Flint is correct in saying that this provided no information as to the scatter of the toughness results.

112. The sequence of welding up the box girder components to combat the worst effects of differential temperature introduced hardly any cost penalty at the stillages. Keeping the girders at a uniform temperature by the use of water sprays as suggested by Dr Flint would introduce serious risks and difficulties in a welded structure such as this.

113. Great care was taken in the setting of the main girder bearings. The bearing locations on the girders were carefully surveyed while the girders were still on the stillages in an unstressed situation. The results of these surveys were used to set the bearings on the piers to line and level. It is certain that the box girders bear evenly on the bearings.

114. Dr Flint asks how the centre longitudinal joint in the side span was aligned. The bracing between the box girders (§ 53) was installed before the joint was welded. The completion of bracing aligned the two halves of the span. Such large girders are really comparatively flexible and deflexions of 5–10 mm are easily induced in them. The detailed alignment of the plate itself was carried out by normal fabrication methods.

115. Dr Flint doubts that making the site joints to the correct geometry would produce the correct design moments. It is an accepted principle of design that in redundant structures or redundant parts of structures the internal distribution of forces can be calculated from the theory of least work and this distribution is assumed for design purposes provided that the unstressed geometry is correct. It is therefore more logical and more practical to achieve the correct geometry at the site joints than to jack calculated forces into the structure. Any errors in fit-up, unless they were of gross magnitude, would have an insignificant effect on the forces and moments of such a large structure.

116. The help that **Mr Brown's** firm gave by checking some of the vital aspects of the erection scheme was much appreciated. The arrangements were modified accordingly. During the lifting of the centre section of the bridge the whole weight of 3420 t was hanging on the 16 screwed rods. A number of the rods were tested to destruction and many others were proof loaded. The whole end connection of column to column was tested by bolting two columns end to end with four screwed rods and jacking a tensile load of 900 t into the joint. Two diagonally opposite screwed rods were then removed and the test was repeated with the full load of 900 t. This demonstrated the adequacy of the screwed rods and of the column end detail to carry a very unevenly distributed load.

117. Mr Brown refers to the statement in § 56 that the residual welding stresses were relieved by the erection processes. Due to the various reversals of movement in the girders during the erection, the peaks of the residual weld stresses were reduced. There will, of course, still be some residual stresses remain-

ing. No allowance for any such relief of stress was made when calculating the girder deflexions at the various erection stages. These calculated deflexions agreed closely with the measured figures, indicating that the relief of residual weld stresses had little effect on the deflexions.

118. As to the angular movements of the temporary trestles on the pontoons, the final arrangements for rotational and erection movements were developed following discussions with Mr Brown's company.

119. The large angular movements of the bridge girders as they were landed on the ring girders, the twin arms of which were some 25 ft apart, created a problem. This was solved by placing extremely compressible packing on the two ring girder arms. As the girder supported on the pontoon was lowered, the two piles of compressible packing allowed the angular rotation of the girder to take place without overloading either ring girder arm. Subsequently the girder was transferred to the temporary bearings and linked jacks, which permitted the smaller subsequent angular rotations referred to by Mr Brown.

120. The question of aerodynamic stability during erection is raised by Mr Brown and Mr Smith. It was deduced that there might be some such instability during the lifting of the centre span section. Accordingly additional temporary damping devices were attached to the structure. No aerodynamic movements were reported. The conclusion has to be that either the structure was not unstable or that the additional damping was effective.

121. The apparent discrepancy in the weight of the centre piece mentioned by Mr Brown is explained by the fact that the figure of 3800 t in § 29 was the weight at the initial lowering of the centre piece into the water, at which time it had extra bracing frames at each end to increase its effective span to match the spacing of the temporary jetties. It was also dressed with the trestles, diesel winches, hawsers and other temporary gear. The weight of 3420 t in § 56 was the weight of the piece stripped of much of this temporary erection gear and ready to be lifted into its permanent position.

122. The increasing availability of heavy floating cranes, as mentioned by Mr Brown, is welcomed by all bridge-builders. If such cranes are capable of placing complete spans, their use has obvious advantages for the erection of those parts of bridges which are over navigable waters.

123. Mr Dixon refers to the fact that the distortions arising from the welding of grade 55 steel were much greater than were expected from a consideration of the higher yield point of this material. No logical explanation was found for this. There are still some things that have to be established by trial and error.

124. Mr Crosthwaite asks why this 'novel' bridge did not receive some recognition in the UK. The regulations governing entries for UK awards prevented it from being entered. The steel navigation spans were awarded the Grand Conceptor Award of the American ACEC Engineering Excellence Awards.

125. Mr Dixon, who was Managing Director of The Cleveland Bridge and Engineering Co. Ltd for the whole period of negotiation, construction and opening of the bridge, has rightly drawn attention to the necessity of having partners. There were two different levels of partnership. One was the partnership between Cleveland Bridge and Redpath Dorman Long, which has been an on-going relationship for many years. We have known each other long enough and well enough to be able to get through a job like this without too much difficulty.

126. The main partnership that had to be forged, and in which Mr Dixon

played such a major part, was without doubt with our Brazilian partners, Montreal Engineering. The Paper pays credit to the Brazilian workmanship on the bridge and we certainly wish to join Mr Dixon in paying tribute to the very good partnership that we had with our Brazilian colleagues. It has left friendships that are working well to this day.

127. Several people have said kind things about the fabrication accuracy. There are two reasons for that. We decided that the only way of making sure that two very large boxes—the two halves of the bridge—fitted after erection was to build them together, tack welded and held together so that they were the same shape, and then cut them apart and take them away for erection.

128. That works well with the longitudinal splices, but as for site splices, alignment etc. credit must go to our site staff. It reflects the highest credit on them that the accuracy of fit that we mention has been achieved.

129. One of the great difficulties in achieving the necessary accuracy of fabrication was the temperature differentials between different parts of the structure at different times of the day. The greatest temperature difference recorded between the top flange and the bottom was 40°C at the same time. Due to the effect of differential expansion, much of the checking and setting-out work had to be done at night.

130. In reply to **Mr Sturgeon**, the joint venture's contract, when signed, was for about £8½ million, allowing for exchange rates. Of that, about £5½ million was to be spent in the UK and £3 million in Brazil.

131. It is difficult to define the final cost because large sums of money were paid in Brazilian currency which changed value extensively throughout the course of the contract. For example, when the contract documents were signed, the exchange rate of the cruzeiro to the pound was 9·14 and it is now about 65. Our estimate is that the final payment made to us, making allowance for converting cruzeiro payments to sterling at the exchange rates current when each payment was made, was about £11 million. We have no information on the total costs of the civil contract work carried out by the Brazilian consortium.

132. In reply to **Mr Brown** and **Mr Sturgeon**, the acceleration measures taken included the following:

- (a) Additional main jacking equipment was provided for the erection of the centre span, thus permitting the installation of the jacking equipment in the main span while the second side span was being erected.
- (b) Certain subsidiary winching and jacking equipment was duplicated to permit certain key operations to proceed concurrently rather than consecutively as originally planned.
- (c) Additional supervisory staff and labour were provided to permit double-shifting of key operations, such as the setting up of the jacking columns and equipment.
- (d) The client arranged that the very large floating dock crane would be made available when required. This enabled the contractor to prefabricate the ring girders and other equipment in large pieces, thus greatly shortening the time for installation on the first pair of piers and transfer to the second pair. Under normal circumstances it would not have been possible to rely on the availability of such a crane when required and this operation had been planned to be carried out piecemeal using the contractor's own cranes.

- (e) The jack-up platforms with their cranes were made available from time to time in the latter days of erection to speed up the lifting and transfer of certain erection equipment.

133. The main contract was signed on 30 April, 1969. Detailing of the structural steelwork and ordering of the special steelwork commenced in May 1969 and fabrication commenced in the UK shops in September 1969.

134. The contract programme provided for the completion of the bridge 14 months after the first two piers were available. These were released on the 24 May, 1973, hence the contractual completion date would have been 24 July, 1974. The acceleration agreement provided for completion on 26 March, 1974. The steel structure and associated works were actually completed on 26 February, 1974, five months ahead of the contract period.

135. It will be obvious to those who have studied the Paper that the civil construction ran into serious difficulties due to the fact that the first Brazilian consortium was unable to continue with the work for various reasons. The job was taken over by the Government agency ECEX and completed by another consortium formed by three of the largest contracting organizations in Brazil. They did a magnificent job in taking over and completing the work and we do not wish there to be any suggestion that the delays caused to the project were of their making. We are also anxious to avoid any sort of criticism of the situation that caused the initial difficulties. Perhaps people overreach themselves, perhaps they try to do too much too early. One should be sympathetic to them and congratulate Brazil on the final successful outcome of a major engineering project.

136. **Mr Hamby** asks about the commercial aspects and effectiveness of the competition. In the introduction we state that the best way to do business abroad these days is to put together a complete package with the help of the finance houses, with British Government backing and British overseas contracting experience. It is a great pity that we could not include British consulting engineers and make it a grand slam, but that could not be.

137. By setting up a complete package with finance acceptable to the client and giving him the sort of service he wants, one is in a negotiating position having advantages to both parties. The advantage to the Brazilians was that they got the finance quickly and that they got the bridge under construction as early as they wished. The competition is really competition in the whole field of financing, setting up the contract and price, rather than competitive tendering for a cash contract in the form to which we have been accustomed in the UK.

138. In reply to **Dr Flint**, the construction of the main span steelwork was dependent only on the construction of the piers, the client having taken special measures to construct the concrete spans supporting the 44 m spans out of sequence. The joint venture was involved in substantially increased costs caused by the delays in construction of the piers and these costs were reflected in the final contract price.

139. During the tender stage, estimates were prepared of the costs of 'conventional' erection and of the scheme finally adopted. The choice between the two was made on the basis of both time for construction and avoidance of the uncertain physical characteristics of the bed of the bay. Events proved that the choice made was correct, both with regard to costs and time. In other

situations with different requirements and conditions the choice might not be so clear-cut, particularly now that very large capacity floating cranes are available.

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