Proc. Instn Civ. Engrs, Part 1, 1980, **68**, May, 297–308

8248 DISCUSSION

Some aspects of the design and construction of the 17th July Bridge at Northgate, Baghdad

M. E. MORRIS & R. B. HANNANT

The Chairman, Mr J. M. Thomas

From the tenders that were received, were there any offers of a different construction method?

Professor P. B. Morice, University of Southampton

At what stage of the work were the archaeological discoveries made? Presumably there were discussions between the Iraqi Ministry of Construction and the organization responsible for archaeological preservation. It would be of interest to know whether the Engineer was consulted on this matter and, for example, if he was allowed to give consideration to the possible redesigning of the bridge, perhaps by increasing the spans, to avoid destruction of the ancient brickwork.

Mr P. L. Martin, Rendel, Palmer & Tritton

The foundation design for the bridge was dominated by consideration of the extraordinary scour conditions. Design studies therefore centred on pile capacity under both vertical and lateral load at times of flood. This discussion contribution enlarges on the pile testing which was carried out.

45. The first two pile tests did not confirm either that the permanent piles would have adequate bearing capacity or that settlements under working load would be within acceptable limits. A further test was made. The test had two objectives. The first was to prove the effectiveness of the unorthodox procedure for cleaning the base of the pile shaft. The second was to determine the relationship between end bearing capacity and the standard penetration test data obtained from the site investigations. An extrapolation could then be made to determine base capacity under scour conditions. Ideally this test would have been made on a pile founded at $-9{\cdot}0$ m, but, even if skin friction could have been eliminated entirely, it would have been necessary to increase the test load to compensate for the increased overburden pressure (from $+5{\cdot}0$ m to river-bed level) over that occurring at maximum scour. Furthermore, a test pile could not by then

Paper published: *Proc. Instn. Civ. Engrs*, Part 1, 1979, **66**, Aug., 437–456.

have been located in the river and, with ground level at an elevation of at least +35·0 m on either bank of the river, the load which would have been required to reach working load conditions would have been beyond the range of the testing equipment available. A special test pile was, therefore, constructed on the north bank near to test pile 1.

46. A 1·5 m dia. shaft was sunk to a level of +12·0 m (Fig. 15) using the procedures adopted for the working piles. The bentonite used during boring was flushed out with water prior to the pile being concreted to a level of +20·0 m. Having thus formed a pile shaft just greater than 5 times the pile diameter an undersized casing (1·2 m dia.) was placed in the concrete as soon as it had stiffened sufficiently. The water was then replaced by bentonite to minimize the risk of collapse of the unlined hole, and the undersized casing was filled with concrete up to ground level. The pile was tested from 31 July to 3 August, 1976, and the load–settlement curves obtained are shown in Fig. 4. The gradient of the load–settlement plot changes at a settlement of 0·7 mm, where a skin friction of the order of 150t was presumed to be mobilized. The ultimate load was found to be of the order of 970t and the ultimate end bearing capacity was presumed to be of the order of 820t. There was no irregularity in the shape of the load–settlement curve to indicate a thin layer of soft or disturbed material at the base of the pile. Test pile 3 confirmed the adequacy of the installation procedure.

47. Back-analysis of the information obtained from all three tests was carried out, using methods given by Meyerhof,[5] in order to investigate the end bearing of the river piles under conditions of maximum design scour. Meyerhof's recommendations for the design of bored piles in granular soils are based on the analysis of load tests reported in the literature and take account of the initial density of the soil as measured in the standard penetration test. These design procedures were checked using the skin friction inferred from pile tests 1, 2 and 3 and the end bearing load inferred from pile test 3. In each case the skin friction was assumed to be fully mobilized at the load corresponding to the first change in gradient on the load–settlement curve. Meyerhof proposes simple relationships between SPT N value, skin friction and end bearing loads. The mean values of N which correspond to these measured values were calculated and are shown in Fig. 15, together with the envelope of test results obtained in the 1974–75 site investigation. The calculated values for skin friction lie within the envelope of results and the end bearing value agrees well with the SPT result from the base of the pile during installation. Meyerhof's recommendations were therefore assumed to be applicable to the design of the river piles.

48. The SPT N value required for the scoured piles to have a factor of safety of 3 against base failure by Meyerhof's method is plotted in Fig. 15. The value required lies well below the envelope of measured values. With reasonable assumptions as to the N value operative, the overall safety factor was approximately 3·3.

49. It was concluded that the river piles would be adequate to sustain the loads imposed under scour conditions and that settlements occurring as load is shed from the pile sides on to the bases as scour takes place would be within the design tolerances for the structure.

50. I would like to acknowledge the work carried out by Mr M. Sweeney (formerly with RPT, now with B. P. Trading Ltd) in the analysis of the pile test results and the derivation of Fig. 15.

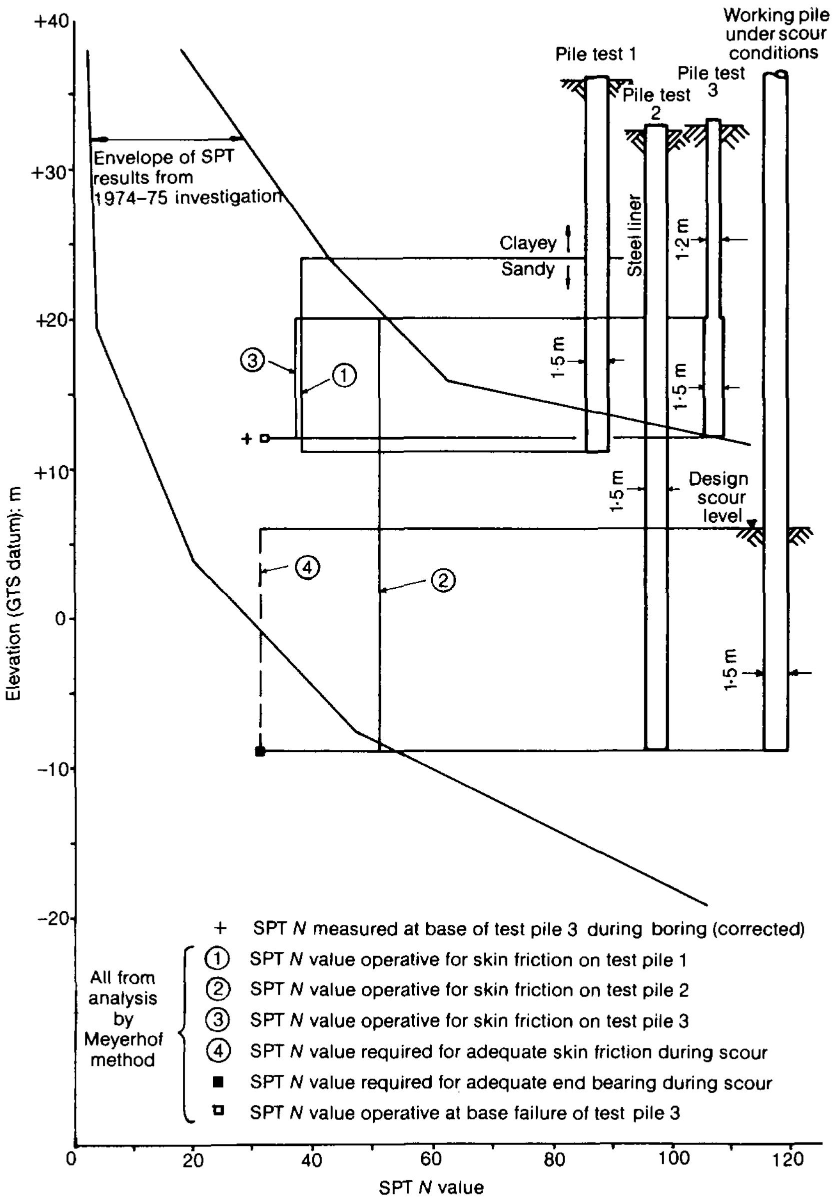

Fig. 15. SPT results in relation to test pile analysis and pile design

Mr J. F. Roe, Rendel, Palmer & Tritton

Rendel, Palmer & Tritton were invited by the General Organisation for Roads and Bridges to advise on the planning of a roundabout on the north bank of the Tigris at Bab-Al-Muadham, and as a result, Mr F. C. Blackmore, seconded from the Transport and Road Research Laboratory, and I went to Baghdad in December 1976. During the course of construction of the bridge, the prison at Baghdad was to be moved, and it was decided that some of the land made available should be used for developing a larger roundabout, capable of carrying the expected traffic. Unfortunately traffic figures were scarce and the only information available was that obtained during an earlier study indicating some 7000 vehicles per hour (two-way flows) over the 17th July Bridge in about 1985. This was a fairly large flow of traffic and concern was therefore very real.

52. It was decided that we should study the existing traffic conditions in Baghdad and suggest some solutions. The existing roundabout on the north bank of the Tigris was not suitable as an experimental location, as congestion on the arms of the roundabout was minimal, and it was therefore difficult to assess the capacity of the existing roundabout. A second reason was that the bridge arm to the roundabout was virtually carrying one-way traffic. Thirdly, permission for filming would have probably taken too long to obtain, as the roundabout was adjacent to an important ministry building.

53. We had about 2 weeks to carry out experiments and make recommendations on design; in particular, we had to make a recommendation on the amount of land that should be set aside for the traffic scheme. The figures we were having to design for at Bab-Al-Muadham were of the order of 7000 vehicles per hour, so we selected the large congested roundabout at Tayaran Square, which is near the middle of Baghdad, some way from the river. We had meetings with the Head of Traffic Control and various other interested bodies to obtain permission to use the roundabout, to carry out an experiment including filming and traffic counting, and to obtain assistance by way of staff and equipment.

54. This roundabout has five arms, and there can be some five lanes of traffic on the roundabout. (The drivers in Baghdad make use of every square inch of road space.) Police control is maintained through about 14–15 h of the day. The number of heavy goods vehicles is fairly limited, but there are double-decker buses, ponies and traps, and pushcarts. This mixture of traffic, and the conditions for visibility round the roundabout, were not ideal for traffic experiments and reaching conclusions. Cine-filming of the junction with police control was carried out, and the traffic leaving the roundabout was counted over several relatively short periods of 6 min during the peak periods when there was congestion on every approach. By selecting the 6 min periods when there were full queues on each approach of the roundabout, we were able to determine the capacity of the roundabout under normal Iraqui conditions. The figure came to 4850 vehicles per hour, quite high for any roundabout, even in London.

55. The second phase of the experiment was to remove the police control and to assess driver behaviour at peak periods with no form of control at all. In Iraq and the Middle East generally, there is no offside priority rule, and the drivers give way to traffic from the right. The drivers on the roundabout thus give way to traffic coming into the roundabout. Within about 6 min the roundabout was solid: instead of about 450 vehicles leaving the roundabout in a 6 min period, there were only 200.

56. There was no delineation between the approach arm and the roundabout itself, so we decided to introduce the European system of 'give way' line markings and road signs, and experiment on the use of these with and without police control. Within 24 h the road maintenance department provided and erected ten 'give way' signs, the first ever to been seen in Baghdad. The 'give way' lines were painted on the road with silver spray paint, the markings being approximately 750 mm square. We then carried out another count, with the police removed, which showed that the signs alone, without police control, were not enough.

57. With police control we were able to increase the traffic through the roundabout to 5800 vehicles per hour, which was some 20% increase on the normal operating conditions, and we felt therefore that there was scope for modern European-style traffic control to be operated fairly successfully with the police standing on the central island of the roundabout rather than on the dividing island where they normally stood.

58. As a result of the experiments we reported that a roundabout was feasible at Bab-Al-Muadham to carry the predicted traffic. We developed two schemes, both having an inscribed circle diameter of 132 m, which is quite large. Within the two weeks we produced a set of drawings from which the land requirements could be assessed. The first scheme (which we preferred) had a single central island, and the second had multiple islands.

59. In the end, neither scheme was adopted, because a traffic-light scheme for the whole of Baghdad was being designed in parallel with out work. However, the experiment was, I feel, of interest and relatively successful in that, with the plentiful co-operation given, one could assess fairly quickly alternative methods of approaching some of the traffic problems.

Mr J. E. G. Palmer, Fellow

Were there Rendel, Palmer & Tritton staff permanently on the site, and if so, what was the average number during the period of the job?

61. By my calculations, the cost of the bridge per unit area of decking is about £1500/m^2. What is the Authors' figure for this?

62. Taking the cubic capacity of the 'bridge envelope', that is, taking the deck area (i.e., the overall length of deck times the overall width of deck) and multiplying by the average depth from top of deck to bottom of foundation, what is the average cost, please, per metre cubed?

Mr R. N. Sainsbury, John Mowlem & Company

I would like to know a little more about the cofferdam being swept away. For example, what state was the work at when the incident occurred? Was programme time lost as a result? How did this come to happen if the consultants, who had done so much investigation on scour, approved the cofferdam proposals? Was the risk deemed to be the contractor's?

64. With regard to the contractor's proposal for the foundation piles, I would like to know whether the difference of £1 million still held after all the elaborate pile testing had been carried out. I am not familiar with the method of reverse circulation piling which enabled the saving to be achieved and I would appreciate more explanation. Was it the only method that could have enabled piling to be carried out successfully?

Mr W. H. Law, Ove Arup & Partners

I would like to know the difference between the bending moment envelope during construction and the bending moment envelope after the continuity cables had been stressed. What kind of adjustment to the two envelopes have the Authors managed to assess regarding the effect of creep?

66. It is mentioned that the scarf joint was assessed at the point of inflexion of the beam. Was there any other practical consideration regarding the location of the joint (i.e., regarding the bending moment envelope differences)?

67. At the support, the bottom slab was reinforced concrete, not precast concrete. Was this purely because at the ultimate condition more compression area was needed in the bottom? I presume that the second-stage capping cables were stressed from the blisters underneath the top flanges. Were the bottom slabs in place when the capping cables were stressed? The capping cables were stressed when the top slabs were in place, but if the bottom slabs were in place as well there will be some loss of prestress from the bottom flange of the I beam into the bottom slab due to the effect of creep. Have the Authors considered that?

68. In Fig. 6, sections AA and BB show that the main cables were not at maximum eccentricities. Was it because the cables were 19 strand, 15 mm? Using smaller cables would it be possible to put the cables at maximum eccentricity by spreading them out into a horizontal layer at mid-span region in the bottom flange?

Mr F. Irwin-Childs, Rendel, Palmer & Tritton

The Paper records that the feasibility report (§ 2) was submitted in 1957, and that the bridge was opened to traffic (§ 39) in 1978. I think most consultants would agree that cohabitation for 20 years is not a habit to be encouraged. This is not a record for the firm, which probably is held by the Thames Barrier, but in the case of this scheme at least the client had time to express a preference for a design with a flat soffit instead of the more familiar arched form of most of the Baghdad bridges.

70. The Northgate bridge is a five-span bridge which has been made continuous over the supports. This is good practice which is now adopted generally, but not universally. In Sicily, for example, the autostrada has to cut through the spurs of the hills and so is virtually a continuous alternation of tunnel and viaduct; the spans are all simply supported, as a result of which travellers are deeply impressed with the disadvantage of non-continuous spans! There is also the impact on the structures, and this aspect could well be a major contributory factor in requiring the large-scale rebuilding of trestles which has been necessary on the Italian autostrada.

71. In contrast with constructing a multi-span structure in situ, the achievement of continuity when building with precast concrete main elements, as in this case, is an interesting exercise and there are various means of approach to it. One of the early examples (which I have not seen copied) was at Northam Bridge in Southampton, where the deck was completely formed of precast, pretensioned T beams. The flanges were cut back to the web some 20 ft on either side of the piers, and 'replacement' planks were inserted spanning over the supports. The deck was then post-tensioned transversely to 'stitch' the system together and the planks were deemed to act like cover plates to steel joists,

but riveted to the side instead of the top. Transverse stressing of this nature has fallen out of fashion, and the solution at Northgate is an elegant one, with the scarf joint located at the point of contraflexure.

72. I suspect that the method of making the joint owes something to a device used much earlier on a jetty built in Australia in which the longitudinal beams were similarly connected by short Macalloy bars across the joint and the joint was filled with fine dry packed concrete hammered in. The water/cement ratio of this mix was as low as 0·2, and a strength of nearly 2000 lbf/sq. in. was achieved in something over $\frac{1}{2}$ h. I would like to know if this is what is referred to as 'jointing concrete', and if this was the method of application in the scheme in question.

73. The adoption of a continuous deck means that the avoidance of differential settlement becomes more important, which put the emphasis on the foundations in this case. With the threat of serious scour this must have become the major headache. As the Authors have quoted, 'the depth to which a river bed may scour is difficult to measure and not easy to believe',[6] but in this case there was no question of not believing it: it was proven fact which had to be accepted.

74. Pile tests such as described are not easy to conduct with piles of this diameter and magnitude. Both the consultants and the contractor, I think, have done well to get the results which they have. Often pile test results are not sufficiently meaningful to give confidence in going ahead with a scheme, but in this case I think they were. The explanation given that much of settlement on the test was probably due to a soft deposit from the bentonite is, in my opinion, quite feasible and it is in line with the experience of most of the foundations done through bentonite that I know. It is, however, likely that cleaning up the hole as recommended in the Paper by jetting and air-lifting would, in itself, tend to disturb the soil (which has, at least partially, been relieved of overburden pressure). Therefore that aspect has to be treated with a certain amount of caution.

75. The bridge is described as being fixed at the southern abutment with thermal movement permitted all to the north. Having regard to the scour pattern in Fig. 2 and the distortion arising from the scale of that figure, as well as the type of bearing on the piers, I question whether any flexure of the pier walls or the piles was envisaged in accommodating temperature movements, and whether there has been any recording on site of extreme thermal movement since completion, for comparison with the calculated assumptions. The latter is a feedback which is all too rarely done and which interests me because I believe that the theoretical maxima are seldom achieved in reality. This is because of the constraints resulting in accumulated stress build-up through the structure and the effect of temperature-averaging through the members.

76. The launching technique adopted with its single girder strikes me as being both economical and effective. I find that the slot which is left in the piers is quite a pleasant feature. I am intrigued to find references to ballraces for moving the heavy beam sideways, an age-old device which still seems to be universally accepted for heavy rolls of this nature.

77. I was pleased also—for quite transparent reasons—to read that the contractors, after due deliberation, adopted the construction method assumed in the design. A designer must always have in mind at least one way in which the structure can be built, and generally, and quite properly, the contractor has the freedom to choose an alternative. When he sees advantage in the designer's

suggestion it often means that both parties are thinking the same way, and adjustments to the details are kept to a minimum and the job tends to benefit. In this case a special effort was required from the Authors, as they had no resident representative on site.

Mr P. H. D. Hancock, George Wimpey

How did the Japanese and the Iraquis divide the principal tasks—technical, managerial, and so on? At peak, how many Japanese were there on site and how many Iraquis, including labourers? How does the way in which the Japanese organize a job and workforce compare with the way in which a UK contractor normally operates abroad (e.g., in South America, Iraq or the Gulf)?

The Chairman

I am interested in the good relationship between the standard penetration tests and the settlement of the large diameter bored piles. I am wondering if this is due to the take-up of bentonite or whether it is a feature of the particular strata in Baghdad, and whether the Authors feel that the standard penetration test is a reliable way of predicting settlement of bored piles.

Mr A. W. Shilston, Consulting Engineer

It is not clear from the Paper how far beyond the preparation of the tender documents stage the bridge designers exercised positive control on the subsequent tender evaluation and construction stages. Allowing for the fact that the tender invitation documents apparently did not preclude alternatives being submitted to the use of reinforced concrete caissons for the bridge foundations, was there any outright bar contained in the tender documents to the use of bridge superstructure falsework schemes which involved the driving of temporary supporting piles in the river bed?

81. In retrospect, for a repeat scheme, would the use of caissons for the bridge foundations offer greater peace of mind, *vis-à-vis* scour hazards, during the construction period?

Mr Morris and Mr Hannant

We agree with the **Chairman's** comment on the importance of the construction method on this project. Both he and **Mr Shilston** raise queries on the alternative designs submitted with the tenders. The tender documents permitted tenderers to submit alternative designs but required any alternative to have the same elevation and architectural appearance as the official design. There was no preclusion on the use of temporary supporting piles in the river bed to support falsework but obviously had any tenderers proposed such schemes it would have been necessary to ensure that they were founded well below the maximum scour level and that the river regime was not interfered with. In the event no tenderer put forward such a proposal, presumably because the tenderers came to the same conclusion as the designers regarding the risk of such schemes. All tenderers put forward pile foundations as an alternative to the specified caissons, the piles varying in diameter from 1·0 m to 2·5 m. At the time the bridge was designed there was, in Iraq, considerable expertise in sinking caissons using Kurdish labourers: they worked extremely well under compressed air. At the time tenders were invited there was a shortage of this labour and it was probably more economic to use large diameter piling. For peace of mind we would still have preferred caissons but in this instance it was certainly quicker as well as cheaper to use piles. Two tenderers put forward alternative superstructure designs, one basically the same as the official design but with all spans simply supported, and the other incorporating hammer heads on the piers with suspended spans. Both of these, apart from being more expensive than the lowest tender, introduced a number of discontinuities in the road surfacing over the deck which, as **Mr Irwin-Childs** points out, is not always desirable.

83. **Professor Morice** raises the question of the archaeological discoveries and the actions taken by the Engineer. When building in any city as old as Baghdad one finds that the top 2–3 m consist of old brickwork, and brick traces were found in all borings. However, as the method of manufacture of brick has changed little over the period, one is unable to determine the age of the brickwork. No very ancient structure was expected to be found in Baghdad. When the contractor began pulling back the right bank to re-align the river some six months after the start of the contract, the remains were discovered. The consulting engineers were retained to give only technical advice during construction and not the normal full supervising service. Thus, although we received a report from the client's Resident Engineer as soon as the remains were discovered and gave our opinion, it was not until shortly before the next major flood season that we were asked to prepare schemes for saving these remains. It was not known, nor is it yet known, how far these remains continue on the land side, so extending the bridge would not necessarily solve the problem; however, the current state of construction, and the extent of river realignment with the consequential property acquisition necessary for such a scheme, made it not feasible. Various schemes were prepared and considered by a joint committee of the client and the Archaeological Directorate who agreed on the one described in the Paper.

84. **Mr Palmer** asks about the consultant's staff on site; we did not supply any staff, the whole of the site staff being provided by the client. Concerning the unit costing, the cost of the structures works out at ID 293 per square metre of deck, equivalent to £440 per square metre at 1974 rates of exchange. The cost of the bridge envelope would be complex to calculate and would, we think, not provide any useful information.

85. **Mr Sainsbury** enquires about the cofferdam and the piling. The consultants were not asked to check the contractor's proposals; at the time the cofferdam was lost, the contractor had completed piers 1 and 2 and was programmed to complete pier 3 before the May flood which in the event he was unable to do. Fortunately, with piers 1 and 2 constructed, work could proceed with the superstructure construction on these spans while the river level precluded work on the foundations. We believe that this loss was deemed to be the contractor's risk.

86. The contract stipulated that any alternative design put forward by tenderers would have a ceiling price, with any testing being inclusive in this price; therefore the savings to the client were maintained.

87. The reverse recirculation piling system is a bored pile with the drilling head suspended by a rope from a crane. Three rotary cutters around the base are almost completely counterbalanced so that there is no torque. The spoil is fed into the centre of the hole and excavated up a central 150 mm pipe into a cleaning tank, and the bentonite is returned into the pile shaft. Other

pile systems could have been used, particularly as this system has the disadvantage that it cannot be used for raker piles, so necessitating the provision of shear stops at the bearings described in the Paper.

88. Concerning the prestressing of the bridge, **Mr Law** asks if the bending moment envelope as constructed was the same as the bending moment in the continuous state. We think he is probably referring to the bending moment envelope during construction when the beams were being rolled out over the already placed beams. In fact, it was found that the maximum bending moment at any given point in the beams during the erection process was much the same as the maximum bending moment in the finished structure, so we did not have to put in any extra prestressing to cope with the rolling loads.

89. The scarf joint was placed at the point of contraflexure because the joint, from the point of view of bending strength, was the weakest point along the line of the beam. It therefore seemed sensible to place the joint at the point of contraflexure under dead load where one had the minimum bending moment. The calculations showed that the minimum bending moment under the rolling loads occurred at that point as well. It was found that the Macalloy bars—there were about 16 of them stressing each scarf joint together—were adequate to carry the moments due to the erection rolling loads and also those in the finished structure.

90. The reinforced concrete slab infilling between the bottom flanges and the precast beams was for ultimate conditions but it was also required for the maximum live load bending moment over the pier, as the bridge, being a constant depth structure, was not the optimum shape to carry the maximum hogging moments over the pier. Ideally one would have liked a parabolic soffit bridge over the piers, but we solved the problem by having a short stretch of in situ slab some 5 m to either side of the bearings on the pier, which was sufficient to carry the compressive stresses induced by the maximum live load hogging moments.

91. The capping cables were stressed when the in situ bottom slab was in place; they were designed to reduce the compressive stress in the bottom flange, such that the flange could accommodate the full live load compressive stress.

92. The cables in Fig. 6 were at the maximum eccentricity. It may be that the drawing is slightly misleading, as it is to a very small scale, and it looks possibly as though the three chain-dotted lines are not absolutely at the maximum eccentricity. The main cables are one below the other concentrated in the width of the web. They are at the maximum possible eccentricity without being spread out into the flanges, which would have been undesirable from a practical point of view.

93. We are impressed that **Mr Irwin-Childs** achieved 2000 lbf/sq. in. in $\frac{1}{2}$ h with his caulking concrete. We did not attempt to achieve that. As we had a 100 mm thick joint to fill and as the beams were so large—being 3 m high—we would have needed $\frac{1}{2}$ m^3 of jointing concrete in each scarf joint. We in fact used a 50/10 jointing concrete which was designed to have a 28 day cubic strength of 50 N/mm^2 using maximum 10 mm dia. aggregates. We believe that the contractor achieved an adequate strength to enable him to stress the bars in about a week.

94. With reference to the cleaning of the pile shaft bottom, we were very conscious of the danger that soil might be disturbed, and developed a method of checking for any loose material. This used a plumb bob which was 'bounced'

up and down on the bottom and enabled one to feel any soft areas: it proved to be very effective.

95. The pier walls were designed to take a certain amount of flexure. Under the original design we had well foundations which were considered as rigid. When the change of design to piles was agreed the pier walls were again checked for bending, but owing to the type of bearings used there was very little longitudinal force induced in them.

96. Mr Irwin-Childs also comments on the ballrace. This system is still often used in the UK although for British Railways replacement bridges we usually specify machined bronze on machined steel as this avoids the necessity of jacking up the bridge to remove the ball-bearings. In the case of Northgate the number of beams requiring moving made the ballrace system more appropriate and we incorporated a ledge in the pier head to accommodate the race.

97. In reply to **Mr Hancock**, generally Obayashi-Gumi undertook the structural work and the State Constructional Contracting Company the civil engineering element and the administration of the joint venture. We are unable to give the breakdown of staff employed on the project but can only give general information. The project manager was Japanese with an Iraqui deputy. The site engineers were of the nationality of their respective interests in the project. Nearly all the specialized trades were subcontracted to Japanese 'labour only' subcontractors and the main labour force was Bangladeshi. The site organization was very thorough, with numerous drawings for temporary works being produced, and the whole of the contractors' plant was brand new, imported from Japan for the project. For all the precast work, steel shuttering was made in Japan, as was the launching girder.

98. It is presumed that the **Chairman** is referring to our application of Meyerhof's methods for predicting the load carrying capacity of piles. As **Mr Martin** states in his contribution. Meyerhof proposed simple relationships between standard penetration test N value and the shaft friction and end bearing capacity of bored piles in sand. By using the values of shaft friction and end bearing capacity deduced from the pile tests in these relationships, N values were obtained. It was found that these were compatible with N values determined in the site investigation (Fig. 15) which indicated that Meyerhof's recommendations could be used in the determination of the load carrying capacity of piles at the site. By this means we were able to assess, with some confidence, the probable behaviour of the river piles under scour conditions, and it was concluded that the additional settlement of the river piles during scour would be within acceptable limits.

99. The validity of Meyerhof's relationships for the design of the bored piles at the site, as indicated by the back-analysis of the results of the pile tests, may be fortuitous. The relationships are empirical, being derived from the analysis of pile tests carried out in several countries. It is not known, however, whether or not any of these test piles were installed by methods comparable with those used at the site, and so the significance of bentonite take-up cannot be assessed.

100. Prediction of settlement for any structure is difficult and no single method can be considered reliable. For structures founded in granular material there is often greater uncertainty because estimates of settlement have usually to be made on the basis of the results of in situ penetration tests, commonly the standard penetration test. The Authors consider that the use of N values in appropriate empirical relationships generally enables bored piles in granular deposits to be designed to ensure that settlements under working loads are within

tolerable limits. However, ideally pile tests to failure should, if possible, be carried out in order to check the installation procedure and performance of the piles, but this was not possible on this project.

101. We wish to thank **Mr Martin** and **Mr Roe** for their interesting contributions, and also wish to thank all our colleagues who assisted in the design, in particular Mr R. J. Honor and Mr S. Lecki who headed the teams responsible for the approaches and bridge superstructure respectively.

Reference

5. MEYERHOF G. G. Bearing capacity and settlement of pile foundations. *J. Geotech. Engng Div. Proc. Am. Soc. Civ. Engrs*, 1976, Mar., No. GT3.
6. SMITH D. W. Bridge failures. *Proc. Instn Civ. Engrs*, Part 1, 1976, **60**, Aug., 367–382.