Paper No. 6223

The design and construction of the Hindiya Bridge †

by

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Discussion

Mr P. S. A. Berridge (Assistant Engineer (Bridges), British Railways, Western Region) referred to the tied arch form of construction chosen for the 100-ft span. The Author had given no details about the design of this span, which was the longest in the bridge. He had passed it off merely as being architecturally suitable and simple to erect. Agreeing with these views, Mr Berridge thought that emphasis should have been laid on the fact that a tied-arch girder required much less steel than a truss of equal strength for this length of span. He wondered why these tied-arch girders had been riveted when welded shop work, combined with high-strength bolts, for site joints would have shown appreciable economy. He instanced the reconstruction in 1951 of a footbridge over the railway east of Cardiff. There, two 100-ft-long riveted mild-steel parallel-chord Warren-truss spans had been replaced with tied-arch girders shop-welded and field bolted (Fig. 19, facing p. 464). Each of the truss spans had contained 17 tons of mild steel which compared with only 12 tons in one of the tied-arch spans. There was therefore a saving of 40% in favour of the tied arch at first cost and, of course, further economy in future painting and maintenance.

53. Referring to the expansion joint in the roadway (Fig. 18), Mr Berridge hoped every city engineer and county surveyor would take careful note of the importance of incorporating properly designed expansion joints at the ends of all steel spans of moderate length. It was no use going to the expense of providing knuckle and sliding bearings under the main girders and then laying the road metalling continuous with the approaches over the ends of the span. The superstructure should be completely self-contained with expansion joints across the roadway, the footpaths, kerbs, and hand-railing. This was just as important in Britain as in tropical countries. The current draft revision of B.S.153 specified an allowance for a range of 100°F of 0·78 in. per 100 ft. And it was as well to remember that neglect in design to provide for such freedom of movement induced a stress of 0·0845 ton/sq. in. for every degree Fahrenheit rise of temperature in mild steel. During a previous discussion[2] Mr Shirley Smith had referred to complaints of bumps when vehicles passed over gaps of 11/16 in. Mr Berridge suggested that a comb joint (Fig. 20, facing p. 464) made up of mild-steel plates, interlaced and more or less self-cleaning, was the proper answer to the problem of designing an expansion joint across a roadway. It presented a smooth level uninterrupted surface to the wheels of vehicles and yet there was complete freedom of movement.

54. Referring again to the tied-arch span of the Hindiya Bridge, Mr Berridge asked for details of the sliding bearings under the main girders. Were they of phosphor-bronze sliding on steel? If so, what lubrication had been provided? The Author mentioned scraping the 100-ft span over the holding-down bolts as it was being floated

† Proc. Instn civ. Engrs, vol. 8, p. 1 (Sept. 1957).

[2] R. P. Mears and E. E. Pool, "The design of Neath and Briton Ferry viaducts". Proc. Inst. civ. Engrs, vol. 7, p. 465 (July 1957). Discussion by H. Shirley Smith, p. 491, § 125.

into position. The river had not come up to expectations! Mr Berridge wondered why holding-down bolts had been used. There could be no question of uplift. He suggested that dowels dropped into holes after the span had been landed, and grouted-in in the bedstones, would have secured the bedplates in position and would have made the site work much easier. He was pleased to see the Author had included some particulars of the cost which, working out to as little as £12·4/sq. ft, was a credit to all concerned with the design and construction of this bridge.

Mr Francis Walley (Senior Structural Engineer, Ministry of Works) referred to the concrete spans. In § 9 they were stated to be "continuous and monolithic with the piers and the abutments." Presumably this meant that they were designed as continuous beams over the piers and not that they were continuous and monolithic with the piers. If they were designed as continuous beams, were they designed as carrying the super-load only or the super-load and part of the decking? It was stated that 16-ft lengths of decking were cast over the piers, presumably to make the beams continuous at an early stage. If they were continuous, it would be interesting to see a detail, because there did not appear to be any way in which the compressive stress was transferred from beam to beam. It was not clear from the slides whether or not any steel had been taken through. If they were continuous, how was the tension steel taken over the support? It was the support which really transferred all the stress across, and no detail had been given of that important part.

56. Fig. 17 was a cross-section of the test beams. Mr Walley did not see how it was possible to pour good concrete into such a section. What was the reason for adopting such a narrow breadth? Had it been for reasons of weight, and, if so, could not a larger number of beams have been used? Had there been difficulties on the site in placing the concrete? It was stated in § 38 that a 6-in. slump had been necessary for the shells, and Mr Walley could only imagine that something similar had been necessary for the beams. It was not surprising that some of the cube results had been low if that had been the case, particularly in view of the statement that vibrators had been used. He could not imagine a specification allowing vibrators and a 6-in. slump. As a rule, good practice would limit the slump to 2–2½ in. Had any segregation taken place? That would be expected with a 6-in. slump.

57. Why had the test been necessary. What had been the cube strengths? It did not appear, from the particulars given, to have been a very difficult test to get through. Reference was made in § 40 to an overload stress of 40%. Presumably that referred to concrete compression in the top flange, because a check calculation showed that the steel stress was well below the design stress and probably did not exceed 16,000 lb/sq. in. If so, the compressive stress in the concrete was about 1,600 lb/sq. in. only, so that it was not a very severe load, and one would not expect any cracking or spalling unless the concrete strength had been well down.

58. Still referring to the concrete spans, Mr Walley would like more information about the expansion joint. The impression was given that the whole of the expansion was taken up on the central pier, which was presumably the pier which supported the steel span. From that it could only be assumed that all the other piers flexed when taking up their expansion or contraction. Was that the case, or had any rollers or sliding bearings been provided at the other piers? Presumably the tying back of the abutments had been to take the whole force of the bridge should the 240 ft of side spans wish to expand in one direction.

Mr A. Goldstein (Partner, Messrs R. Travers Morgan and Partners, Consulting Engineers, London) asked it the Ministry of Development loading in Iraq was at all similar to the Ministry of Transport loading in Britain. The appearance of the bridge suggested to him that the Ministry of Development loading was in fact lighter. If it was not similar, what was the extent of the difference?

60. In connexion with the articulation of the bridge, what was the traction load for which the bridge had had to be designed?

61. In § 9 it was stated that "Figs 2 to 7 indicated the main principles of the design and the following points may clarify some of the detail." They did not appear to do so. The first statement made was "The five spans at each end of the bridge are continuous and monolithic with the piers and the abutments." The next relevant statement was that "Expansion of each group of side spans is taken up on rollers at each of the centre piers." Those two statements, he submitted, were mutually contradictory. If indeed the beams were continuous and monolithic with the piers, then the piers seemed stiff enough for the beams to be designed as virtually fixed-ended beams. If they were merely continuous, and not monolithic with the piers, it was difficult to understand the detail in Fig. 6, which certainly seemed to indicate that the whole was quite solidly fixed down. If it was fixed down, then although Mr Walley had said that the expansion movement must take place in the piers, Mr Goldstein did not see how that could be so without inducing appreciable temperature stresses. Since the piers were stiff, any expansion could not be taken by a rocking movement of the pier, but only by a bending movement, and, owing to the fixity at haunch level, there could be quite large bending stresses caused in the beam by temperature movement. Perhaps the temperature stresses had been allowed for. If so, what were they?

62. With regard to the actual beams, he noticed that the span of the beams, between the centres of the piers, was about 46 ft. Had all the steel bars been delivered in 46-ft lengths. If not, then, as Mr Walley had said, the cross-section in Fig. 17 looked quite full enough of steel as it was, without the additional complication of splices which would seem difficult to achieve without prejudicing satisfactory concrete placing. Perhaps the Author could give a longitudinal elevation of reinforcement to clarify this point.

63. Apart from those practical details, which had probably all been considered, there was the question of what the effect would be in time with regard to crack distribution. He was speaking not of any catastrophic or failure cracks, but of the normal flexure cracks which occurred in reinforced concrete work. It was one thing to say that no cracks appeared under test, but, as was generally known, it was not possible to find out what a crack pattern would be like until 5 to 10 years, and sometimes longer, after a bridge had come into use. It had been made clear at the recent Stockholm[3] conference that the eventual crack pattern and crack widths depended not only on the actual steel stress adopted in the reinforcement but also very much on the distribution of reinforcement in the concrete, the size and congestion of bars, and so on (without even considering such matters as cover). With a detail of the kind shown (Fig. 17), where the steel was—in his opinion—too concentrated, the conclusion would be that the crack pattern would take an undesirable form, with greater crack-widths than if the reinforcement had been more spaced out, the latter providing a more desirable detail.

64. On the question of expansion joints, he agreed with Mr Berridge that the comb type of joint had very much to recommend it. The only problem there was perhaps corrosion on the vertical face. In the instance in question, details of the joint provided would be interesting.

65. The problem of expansion joints had not yet been entirely satisfactorily solved, but one thing seemed manifest, namely, that thin plate joints were unsatisfactory. If plate joints were to be used, they must be solid plate joints of considerable thickness, and if the gap was say, 1½ in. deep, whatever the width happened to be, that must be accepted.

66. With regard to costs, working it out at about £12/sq. ft, he wondered whether the Author could divide that into the cost on the approach spans (concrete) and the cost on the centre span (steel). It was difficult, but a general indication would be very useful.

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[3] Symposium on Bond and Crack Formation in Reinforced Concrete. RILEM, Stockholm, 1957.

Mr R. F. Bartholomew (Soils Engineer, Le Grand, Sutcliff and Gell, Ltd) referring to his own experience of site investigations in Iraq, said that the design of foundations for highway bridges in that country seemed to be very much dependent on the high penetration resistance which was generally encountered in the bearing strata. On the River Tigris he had obtained standard penetration figures for three bridge sites where the soils in question were alluvial sands, medium to fine grained, part of the great thickness of sediment laid down by the rivers. The penetration resistance at all three sites was extraordinarily high for such soils. At two sites in Baghdad the penetration resistance was up to 300, 400, and 600 blows per 12-in. penetration, and at a bridge site at Kut up to 80 blows, although the results at Kut might have been under-estimated because there had been upward seepage in the boreholes due to sub-artesian pressure. He believed that various patent types of pile had been used in such strata.

68. At Hindiya on the Euphrates, as described in the Paper, composite steel piles had been driven into dense and cemented sands with some difficulty. Could the Author give the standard penetration figures for, and some more descriptive information about, those soils? Presumably the sands were not of recent alluvial derivation but might belong to the Upper Fars series, which was a much older geological formation. It would be of particular interest to know if any connexion could be established between the standard penetration test results and the actual penetration curves in pile-driving.

69. The Abbassiyat Bridge site, with which Mr Bartholomew had been concerned together with the Author, was about 45 km south of Hindiya on the Shamiyah branch of the Euphrates, and also had a river bed which had been very recently formed. Recent alluvium and stiff Pleistocene clay overlay another geological formation (Bakhtiari) which was a little younger than the Upper Fars at Hindiya. It consisted of three definite types of soil. The first consisted of multi-coloured interbedded sandy loams and clayey sands, followed by clean coarse-to-medium sands and then limey clayey silty sands. Some of those soils were highly gypsiferous. It had been possible to confirm the exact nature of the beds by examining, some miles south of Abbassiyat, an outcrop of the strata in question in the Najaf Tar. The penetration resistance figures obtained had been very high. On an average, they had been, for the three strata in question, 130, 97, and 186.

70. He was sure that the consulting engineers, when they came to design the pier foundations in the river, would not be satisfied if their piles did not go below the interbedded sandy loams and clayey sands, the top layer of this series. The Author and Mr Bartholomew had considered that perhaps some sort of composite steel pile similar to that used at Hindiya might be suitable, but were doubtful about the possibility of either a caisson or a joist being driven through this top stratum. Jetting was unlikely to be particularly successful, because although the soil was predominantly sandy it was also very cohesive and did not wash away with jetting. They thought, however, that it might be possible to get the caissons down by removing the core of soil inside the caisson in short lengths. Mr Bartholomew meant really short lengths, because they had found that it was almost impossible to obtain the standard undisturbed samples; they could drive the core barrel for only a few inches before there was absolutely no further penetration. Had the Author had any further ideas about the way in which the Abbassiyat job might be tackled?

71. Mr Bartholomew had raised these points, which might seem a little irrelevant to the Paper, because he believed that it was not possible to have too much information about penetration tests, piling experience, and ground conditions in view of the present state of knowledge of the use of soil-mechanics data in designing piled foundations. Anything that the Author had to say about piling in Iraq should be of general interest. Mr Bartholomew was very appreciative of the Author's co-operation in Iraq. Even the carrying out of what would normally be regarded as a simple job in England such as a site investigation was quite difficult in that country.

Mr J. F. Pain (Manager, Bridge Department, Dorman Long Ltd, Luton) said it was clear from the drawings that the whole design had been laid out with the method of construction in view. That was a matter of practical importance in a country such as Iraq, where the working season on the rivers was relatively short. The two items which struck him most were the use of unit precast construction, which the contractor was able to get on with in the flood season in a yard, and the use of steel rather than concrete piles The average craftsman labour in Iraq was not of very high quality, and working on precast sections in a yard enabled the contractor's supervisory staff to make much better use of the craftsmen that they had than if those men were spread over piers in the river, and made it possible to concentrate the work in the river much more during the working season.

73. The precast shells, though they had been rather fiddly things to make, had not presented nearly as much trouble as the contractor had expected, and the concrete had gone in quite well. They had been of enormous advantage when it came to getting the piers constructed in a hurry.

74. The steel piles used in the foundations had considerable advantages over the ordinary concrete pile from the constructional point of view. They were much easier to handle; there was not the same risk of damage in dealing with them; and the butt welds between the different sections of the piles had not presented any trouble at all. The steel piles also cut out what was always a very troublesome business, the laying out of a pile bed and the long time required to mature the piles on the river bank. From the contractor's point of view, therefore, there was a saving in his time and of costs on the site, which would be reflected in the overall cost of the job. The Hindiya Bridge was the only case he had come across in which steel piles had been used for a bridge in Iraq, and it would be of interest if the Author would give some idea of their advantages and disadvantages from the design point of view and would say whether or not they were likely to be used in other structures.

75. Mr Pain had been puzzled by the thickness of the caisson shells round the piles, which were $\frac{3}{4}$ in. thick and 18 in. int. dia. The space between the caisson and the pile was filled with concrete, which seemed to afford protection for the pile, so what was the reason for $\frac{3}{4}$-in. thickness of shell? Was it provision for corrosion or intended to give structural strength?

Mr I. P. Haigh (Senior Engineer, Sir Alexander Gibb and Partners) remarked that Fig. 2, p. 3, showed that the H-piles had been driven down into beds of sand, and below them was more dense sand. Was there not some special risk of corrosion there, possibly from ground-water flow below the river bed?

77. With regard to the driving of the caisson piles, could the Author give any indication as to the accuracy with which those piles had been driven and the means adopted by the contractor for sealing the bottom shutter on the shells? Had it been necessary to survey each group of piles before the shutter had been fabricated and fixed to the shell?

78. Had any finishing treatment been given to the abutments, either by the use of coloured cements or by changes in the mix? He had not previously realized that the abutments were in fact imposing structures, and to users of the bridge might well prove to be the most prominent part of it.

Mr E. K. Bridge (Assistant Engineer (Bridges), British Railways, Southern Region), referring to the Author's statement that the piles on the abutments had been driven to a shallower depth than the piles in the river, said that on looking at the cross-section it seemed that the deep channel of the river was not under the main steel span but under the second span from Hilla bank. It would appear that the river had not settled finally into its course, and it was very likely, in those sandy soils, to continue to meander up and down across its bed. In that case, was not it likely that the deep channel might meander towards one of the abutments? In that event the shallow piling would be a disadvantage. Perhaps, however, the Author had had some reason for making those piles shallower.

80. Mr Berridge had said that the tied arch saved a great deal of steel. Mr Bridge

was not clear about this. There were two types of tied arch: (*a*) the type which Mr Berridge had illustrated, where all the bending of the footbridge was taken on the girder and only the direct thrust on the arch; and (*b*) the type of arch (which appeared to have been used at Hindiya) where the bending stresses were to a certain extent taken in the arch. Was that the way in which the Hindiya Bridge had been designed? The side girders appeared to be very shallow and would not seem to be able to relieve the arch of local bending. In any case Mr Bridge did not agree with Mr Berridge that tied arches of the former type were economical, being at the moment concerned with the question of using a tied arch of larger span than that in question, and having come to the conclusion that it would cost 10% more. He agreed that the appearance was better.

81. Mr Bridge frequently saw designs submitted by local authorities who were enthusiastic about prestressed concrete and who designed a footbridge of very long span with a floor of prestressed concrete beams. It was sometimes astoundingly shallow, but then they proceeded to add side walls to keep people from falling off. If one were going to build a parapet 4 ft high one might as well use its bonding resistance in the design calculations for the bridge; in any case, where there was a thin flexible girder and a parapet on the top, what happened to the connexion between the two was at least open to doubt.

82. Mr Bridge agreed that the comb type of expansion joint was excellent. He had had trouble with the flat plate on certain movable bridges, and the main trouble was the clatter of the plate as every vehicle went over it. He had tried putting Ferodo strips under it and using new bolts, but within a week the familiar clatter could be heard again. It was very difficult to keep a plate fixed down on a bridge carrying heavy traffic. He knew of a 100-ft-span road bridge built many years ago in Aberdeen where a great deal of money had been spent on roller bearings; the road surface had then been made solid with the abutments! But nothing seemed to happen, so presumably it was all right.

83. He noticed that there was no provision for future services in the bridge. Perhaps in view of the nature of the country services such as electric cables were not envisaged, but the country was developing, and it seemed surprising that some provision had not been made for services under the footpaths.

84. Turning to the very narrow girders, which had already been the subject of comment in the discussion, he would have expected that there would be at least a few transverse diaphragms to tie one girder to another; that would, he thought, have stiffened the structure and helped to relieve the deck from some of the work of transverse distribution.

Mr A. G. Gullan (Chief Supterintendent, Designs, Air Ministry Directorate-General of Works) appreciated the point of Mr Bridge's reference to a meandering river, and, referring to Table 2, asked how much was involved in £15,000 worth of revetments. It was quite conceivable that the Euphrates would shorten its course upstream. Owing to its very flat hydraulic gradient, 1 in 10,000, and its old characteristics, it kept its length practically the same from its source to Basra, so it was liable to cut back, as it had done through the centuries, and in course of time it might miss the present site altogether and turn out 10 miles to the south, or even to the north. To keep the course in its present channel, therefore, he felt that the revetment must be carried upstream and downstream for a considerable length, and it would be interesting to know what had been done in that respect.

86. Reference had been made to a "wet season." Mr Gullan would prefer to call it a dry season. The wet season happened up country and the rains came down quickly, but in Iraq there was very little rainfall. It was the rain coming from Turkey, upstream, that made it necessary to provide against sudden spates arising in the Euphrates at very short notice. The authorities, however, kept excellent records right up the Euphrates, so that warnings could be given of storms coming down.

87. A considerable part of the deck concreting appeared to have been carried out in

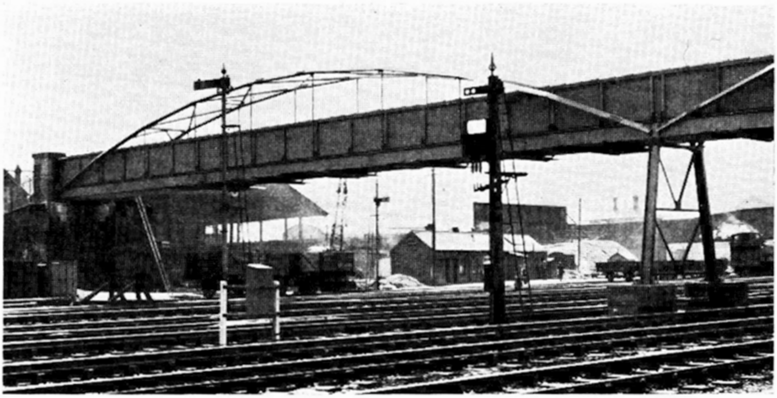

FIG. 19.—100-FT TIED-ARCH SPAN OF FOOTBRIDGE NEAR CARDIFF

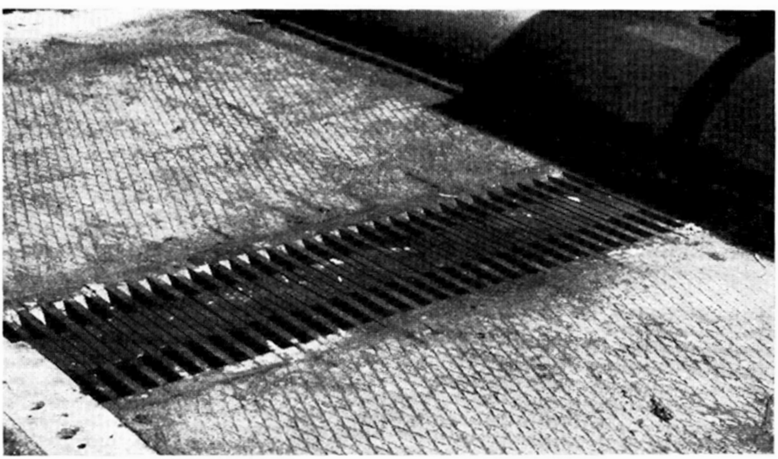

FIG. 20.—COMB EXPANSION JOINT IN A ROADWAY

the very hot season, July and August. How had curing been dealt with and what protection had been used? Had cement been supplied from the local Baghdad works or had it been imported? The local cement was liable to change its characteristics when big dust storms occurred. There was little that could be done about it, and it led to trouble. What concrete mixes had been used, and what was the size of the aggregate? In § 40 reference was made to the difficulty experienced with sand from the Habbaniya Lake area. In Mr Gullan's experience every contractor would offer to provide sand free from gypsum and would show it to the engineer, but it never came from the pit from which he would deliver the sand. It was essential, therefore, to see the source of the sand before accepting a quotation or estimate. Gypsum was very prevalent everywhere, both above and below the Habbaniya Lake. The lake had varied considerably in depth in the past 15 years.

88. It would be of interest if the Author would give the specification of the paint which he had used.

Mr W. T. F. Austin (Civil Engineer, Messrs Freeman Fox and Partners, Consulting Engineers, London) observed that the virtues of the comb expansion joint had already been extolled, but there remained the question of drainage. A frequent source of bad corrosion in bridges was the expansion joint, and an effort should be made to produce expansion joints which were adequately drained or watertight. Too much importance could not be attached to this. There were a number of types of joint on the London bridges. The sliding plate type was used on Waterloo Bridge, and that seemed all right to drive over. The joints on the approaches to Tower Bridge were rubber tubes with rubber-bitumen jointing compound on the top, which solved the problem of a waterproof joint and was ingenious. He had heard recently of another type of deck expansion joint which consisted of vertical steel plates across the roadway slab, sandwiched between strips of rubber, to which they were bonded. They were jacked together in the direction along the slab to form a flat waterproof joint. The jacking force required might be an embarrassment, but, where it was not, this would be an ideal joint for small movement; with large movements it would be inconveniently long.

90. Steel H piles were little used by British engineers, but were common in the United States of America, where special sections were rolled similar to those used by the Author, except that the web was generally the same thickness as the flanges. The corrosion of the steel piles had been mentioned. The Americans were satisfied that they did not corrode unduly provided they were always immersed in water which was not saline, or were completely below ground-water level, and Mr Austin had seen pictures of piles pulled up after 30 years which were in good condition. In one, where the steel H piles had been driven in clay, the section which came up was a rectangle, the spaces between the flanges and web being filled with clay. This indicated the friction developed. Steel H piles were very useful when a heavily loaded pier was supported on hard rock below a bed of sand, where it was very difficult to drive a large number of reinforced concrete solid piles without compacting the sand and making it almost impossible to drive the last few piles through the sand to reach the rock. He had heard of a case where so many reinforced concrete piles had been driven for the foundations for a rolling mill that the effect had been to raise the whole ground level on the site by 2 ft and to push a canal wall into the canal.

91. In the Hindiya Bridge the number of expansion joints had been kept to the minimum by making the deck continuous over the intermediate piers. Those piers were fairly flexible, but in cases where piers were stiffer the temperature movements on the concrete deck were an embarrassment. It was stated in § 10 that "Thermal movements in the concrete bridge deck were subsequently recorded over a period of 6 months from summer to winter and, with respect to variation of ambient air temperature, they conformed closely to customary calculation with a coefficient of 0·000006." Mr Austin had heard of many different customary methods of calculation; would the Author amplify that statement? Did the Author mean that the concrete deck reached the maximum temperature during the day or merely the average daily temperature during

the summer and the average daily temperature during the winter? The sections were very thin. In the case of thicker reinforced concrete sections it was often the practice to assume that concrete sections never suffered the full variation of air temperature.

Mr T. J. Upstone (Chief Designer, Bridge and Contracting Department of Dorman Long (Bridge and Engineering) Ltd) raised the question of vibration in the hangers of the tied arch. The centre tied arch of the Hindiya Bridge, he said, had very slender hangers, and it had not been unknown in the past for such hangers to vibrate under wind forces. The Hindiya Bridge had now been built for some time: was any trouble being experienced in that respect?

The Chairman, confessed to a feeling of disappointment such as he had had when reading other Papers on a similar topic. It was due to the assumption on the part of the Authors that everyone knew exactly how the design of the structures was carried out. He sometimes wished he could see into the mind of the designer of a bridge and know what kind of risks the designer had been prepared to take or what coverage he gave himself against risks which he was not prepared to evaluate.

94. There must have been all sorts of problems in the design of the Hindiya Bridge. For example, the tie in the centre arch span appeared to be a fairly stiff beam and interaction between this beam and the arch would present an interesting problem. What assumptions had been made in that connexion? What stresses had been used in the design?

95. Again, in §§ 40 and 41, reference was made to certain tests on a concrete beam, during which the deflexions had been measured. The type of loading was not explicitly stated but whatever it had been it would be interesting to be able to compare the measured deflexions with the designer's estimated values. It was stated that a 40% overload had been applied, but working stresses were not given. The Chairman asked if any attempt had been made by the Author to compare test behaviour with that calculated and he emphasized the importance of such correlation.

The following contributions were received in writing.

Mr R. W. Brims (retired) was particularly interested in the steel-piled foundations, which were similar to the protected steel piles used in part of the Tyne Commission Quay, North Shields,[4] for which he had been Engineer to the contractors.

97. The protecting steel tubes had been driven first, without a shoe, and then cleared out with a small "orange-peel" grab. These tubes were of mild steel $\frac{1}{4}$ in. thick and 2 ft 9 in. I.D. They had usually sealed themselves, and could be visually inspected inside, but occasionally water had percolated in at the bottom. Since the grab had had only a small clearance inside the tube, it was almost certain that no silt had been left adhering to the smooth inside face. After pitching and driving the "cruciform" steel piles inside the tubes, any water had been blown out, steel reinforcement inserted, and the tubes concreted up. The protection of these steel piles had been fully considered by the late Sir Frederick Palmer, Past-President of the Institution and Consulting Engineer to the Tyne Improvement Commission, and since Messrs Dorman Long and Company Ltd had made and supplied the steel piles to his specification, they would know of the precautions taken against corrosion at that time.

98. At Hindiya, as described in § 25, the bulk of the material inside the tubes, or "caissons" as the Author had somewhat confusingly called them, had not been cleared out until after the 12-in. × 12-in. steel piles had been driven, and this clearing must have been not only tedious, but also extremely difficult to achieve. Why had this procedure

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[4] R. F. Hindmarsh, "Tyne Commission Quay, North Shields". Min. Proc. Instn civ. Engrs, vol. 230 (1929–30, Pt II), p. 25.

been accepted by the resident engineer? If a tube of 18½-in. I.D. had been considered too small for a grab, the various tools used in well-boring were nowadays available for dealing with any material.

99. According to § 7, the ¾-in.-thick steel tubes were supposed to carry part of the load; it seemed preferable to use thinner and cheaper tubes and consider them simply as a means of providing a reinforced concrete protection to the piles against corrosion. No indication was given in the Paper of what proportion of the total load had been assumed to be carried by the tubes in frictional or other resistance.

100. Fig. 10 showed a precast concrete shell, more usually called a cylinder, being lowered over three pile-tubes with a bottom shutter in split halves attached. While this was ingenious, it was not clear how anything like a watertight connexion could have been made under water with the three tubes, of which two were battered, and which could hardly have been positioned at all accurately. Had some rough caulking by diver been done before the concrete seal was placed?

Mr R. C. Harvey (Messrs Rendel, Palmer & Tritton, Consulting Engineers, London) thought that, regarding the design, the form selected and in particular the bow-string navigation span was not only functional but appropriate. There were a number of bow-string steel girders as well as tied arches in existence in the Middle East and in his opinion those types of span were very suitable for use in a part of the world where curves were a dominating feature in much of the architecture.

102. Hydraulically, the design appeared good for a river which had high flood velocity and a bed which was liable to scour.

103. Furthermore, the design included the maximum amount of prefabricated work, which had resulted in reduced in-situ operations and temporary works such as coffer-dams and concrete shuttering. From the figures of cost given in the Paper the economy of the design was apparent, and the Consulting Engineers were to be complimented on their choice of type for the structure.

104. Secondly, regarding the structural features, the piling was a very interesting feature of the work. Steel bearing piles were useful foundations in many circumstances, and the driving results given by the Author and shown in Fig. 9 of the Paper were interesting. Mr Harvey's experience agreed with the Author's inasmuch as he had found that the steel bearing pile (H section) was driven more easily with an automatic hammer with comparatively rapid blows than with a slow-acting heavy hammer. He thought that the choice of the 9B3 McKiernan-Terry hammer for the piles on the entire bridge had been a good one.

105. When the driving resistance was high had there been any indication of splitting of the toes of the piles? He had found that splitting sometimes occurred at the junction of the web and the flange. It seemed that the toes of the piles had not been reinforced to enable them to stand up to hard driving.

106. The test loading of the piles had been mentioned but had any re-driving tests been carried out during the work? Mr Harvey had found that re-driving was a useful test which could be carried out very conveniently, since it could be done first thing in the morning to the last pile of the previous day's work, which had "rested" one night.

107. Mr Harvey thought that the design of the pier bents had been intended to permit flexibility along the centre-line of the bridge in order to accommodate the expansion between the abutments and the piers of the navigation span. This had not been mentioned in the Paper, would the Author say if this was the case? Had any side pulling tests been made on the pile bents, before the 46-ft precast girders were erected, in order to ascertain their flexibility?

108. The thickness of 20-in.-dia. pipe used at the river-bed-level section of the piers had been criticized but Mr Harvey thought that the designers had been wise in providing ¾-in. thickness of material there since it was possible that in times of flood there would be some attrition by the sand in suspension.

109. Regarding the type of pile used, had the designers considered the use of steel pipes of suitable diameter for driving within the 20-in.-dia. starter tubes? For the

driving conditions envisaged it appeared the use of the open-ended steel-pipe pile would have some advantages over the H or joist section.

110. Some revetment work had been done at this bridge. Would the Author give particulars regarding the type used?

111. The 6-in.-dia. bottom-opening skip, referred to in § 26, sounded like a very practical and useful tool. If it had any special features, would the Author include a sketch of it in his reply?

Mr G. W. Morley (Dorman Long (Bridge & Engineering) Ltd, Iraq) said that the design of the Hindiya Bridge had been very economical, and had lent itself to very rapid construction. On the basis of area of road surface the estimated cost had been £14/sq. ft and the actual cost somewhat less. The use of steel broad-flange-beam piles driven through steel-tube caissons had allowed pile-driving to commence as soon as the materials were delivered to the site. This had been in contrast to the time lost, at the start of reinforced concrete piling, in waiting for the casting and curing of the piles. Handling of steel piles had also been much easier; they could be lengthened or shortened with ease, as mentioned in § 22 of the Paper. The precasting of the pier shells and bridge beams had been entirely independent of river conditions, the progress of the piling, or any other site work, and had again led to rapid progress. In this connexion the agreement of the consultants to the contractors' suggestion that the lower shells should be provided with a slotted hole in their base to accommodate all three piles, instead of individual holes for each pile, had materially increased the speed of construction, since it had enabled all piles to be driven before the placing of the shells. In the original design the two raking piles in each shell would have had to be driven through the shell while it was supported on the centre vertical pile alone.

113. While the design of the abutments had permitted rapid progress on the structure of the bridge, one criticism of the abutment screen could be made. The sheet-piles shown in Fig. 7 were purely bearing piles and could not be used to protect the foundation of the screen from the river during its construction. The Kerbala abutment screen had been under construction in the winter of 1954 behind the protection of a low earth bund when this had been destroyed by the unusual and unexpected rise in the river in December and January. For the Hilla abutment the Consultants had agreed to move the line of the sheet-piles to the outside of the foundation, thus allowing them to be used as a coffer-dam face which had been subsequently cut down to slab level.

114. The method of erection proposed in the Specification had been for construction to proceed from both banks simultaneously, using cranes moving out on the completed work as it progressed, and the design had been specially adapted for this method. However, because the cross-section of the river had indicated that there was always sufficient water between abutments to float pontoons carrying derricks, the contractors had elected to use floating plant. It had been the flexibility inherent in the use of floating plant which had contributed most to the successful completion of the contract within the permitted period. Section 18 of the Paper described how the water level had varied in an 11-day cycle and how the river-bed level had been 3 ft higher than expected. While adding to the difficulties neither of these conditions had proved insurmountable with floating plant, and indeed it had been decided to make use of the water-level cycle to float the centre arch into position instead of building it in place on piled falsework, as originally intended by the contractors. The steelwork had therefore been assembled on trestles on the pontoons during the 1955 flood, and it had been known that the water level would be suitable for floating in on a falling river at intervals of 11 days during the summer. In fact advantage had been taken of the first of these levels. Approximately 3 months' erection time had been saved in this way, since piling of the falsework could not have started earlier than the date chosen to float in.

115. When the contract had been let no double-acting hammer larger than the 9B3 McKiernan-Terry had been available. This had been thought too light to drive the broad-flange-beam piles and it had been decided to use 4-ton semi-automatic

hammers. Followers and false leaders had been provided to guide these, but it had transpired that they could not stand up to the large forces set up by the resilience of the steel piles. Leaders and followers had been thrown back 6 in. clear of the pile heads after each blow and the leaders had soon disintegrated under this treatment. It had become necessary to use the 9B3 hammer and it was immediately found that it would drive the broad-flange beams up to 5 ft farther than the 4-ton semi-automatic. Driving, however, had often been very severe and it could be seen from Fig. 9 that resistances of up to 130 blows per inch had been encountered. The aim had been to achieve 80 blows per inch at the desired penetration. A 10 or 11 B3 hammer would have been more suitable, and would be used on another occasion.

116. Placing of the lower shells had often been very difficult, as described in the Paper. At these times these 12-ft-high shells had been accurately lined and levelled with their tops 4 in. below water where they could not even be seen. However, the speed of construction in better conditions was illustrated by the fact that ten lower and ten upper shells had been placed in position, sealed, and filled with mass concrete in the first 10 days of February 1955.

117. The success of this contract had largely been due to the spirit of co-operation which had existed between the consultants and the contractors.

Mr A. R. B. Edgecombe (Head of Hydraulics Division, Central Laboratory, George Wimpey & Co., Ltd, Hayes) observed that the construction of the Hindiya Bridge had added a fourth to the number of bridges across the main course of the Euphrates, omitting the Abbassiyat Road Bridge over the Shamiyah branch of the river, and it was interesting to compare the overall spans of these bridges.

TABLE 3

Bridge	Completed	Overall span
Falluja Road . . .	1930	277 m (910 ft)
Mussayeb Road . .	1938	194 „ (638 „)
Hindiya Road . . .	1955	174 „ (570 „)
Barbooti Railway . .	1925	176 „ (579 „)

119. When designing the Hindiya barrage in 1913 Sir William Willcocks had estimated the maximum flood of the Euphrates at 4,000 cumecs, counting on the barrage to pass a maximum discharge of 2,840 cumecs. Records had confirmed the latter but had shown the former to have been underestimated. It was considered that the barrage could safely pass 3,000 cumecs. This was the order of discharge which could and did pass the Hindiya barrage and would pass through the Hindiya Bridge.

120. Sir William Willcocks had given the Hindiya barrage a width of 237·5 m between the right abutment and the outer lock wall on the left bank. This had been before Mr Gerald Lacey expounded his regime theory, but Sir William's estimate of the width had been very close to the regime width.

121. Another factor had now entered the case inasmuch as the new works at Ramadi would deflect floods on the Euphrates in excess of 2,500 cumecs into Lake Habbaniya.

122. Mr Edgecombe was fully aware that deltaic rivers could be throttled down below the regime width at bridge crossings if such a course proved economically warrantable, but would be interested to know how the overall span of the Hindiya Bridge had been arrived at.

Mr Rowley replied briefly on behalf of the Author and his comments were incorporated in the Author's written reply.

The Author, in reply, said that he realized the Paper lacked detailed information on certain aspects of the design. While regretting this criticism he stated that he had, quite deliberately, confined the Paper to those aspects of the design which he thought were uncommon to the general run of bridge construction.

125. With regard to the choice of the arch for the centre span, he believed that the gap could have been closed more economically (e.g. an alternative tender for a plate girder had been offered at a saving of approximately £3,000) but, as stated in the Paper, the arch had been chosen as a suitable feature for the surroundings. Amongst other possible alternatives had been a suggestion that the arch be constructed in reinforced concrete, but in the circumstances and in the time available, the simple mild-steel riveted construction had been considered the most advantageous.

126. The type used of roadway expansion joint was a combination of the sliding plate and comb types, i.e. the sliding plate and back plate were tongued and interleaved. While this joint was not entirely self-cleansing, it had the dual advantage of smoothness to traffic and safety for animal hoofs.

127. The movable bearings for the arch span comprised a phosphor-bronze plate sandwiched between two steel plates and were designed so far as possible to prevent ingress of dust and grit. No particular facilities for lubrication were provided.

128. The contractors might have suggested the use of dowels instead of holding-down bolts for positioning the centre span had they felt that circumstances warranted the alteration. There were several methods of erecting a truss and it was often convenient to have well-fixed bolts for anchoring down temporary or permanent sections during erection.

129. The articulation of the beam and pier system was questioned in §§ 55 and 61. Fig. 21 showed a section through the joint. The beams were continuous over the piers by tension steel in the deck and by concrete in compression in their soffits, which were carried through the thin spline walls surmounting the transverse box-girders. There was no compression steel. The spline walls were attached to the transverse girders with nominal reinforcement. Although the system appeared virtually monolithic the

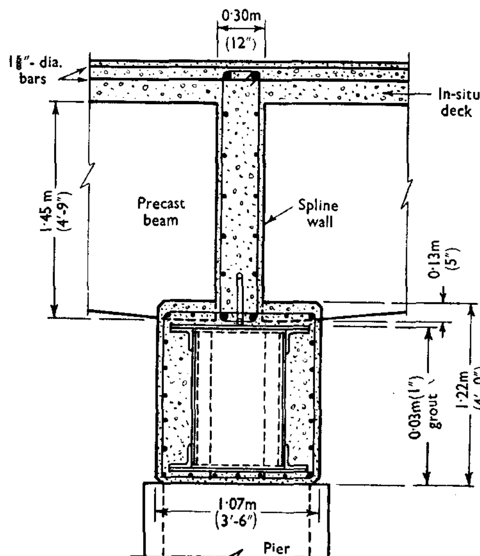


FIG. 21.—SECTION THROUGH CROSS-BEAM AND SPLINE WALL

attachment of the beams and spline walls to the tops of the box-girders could actually be considered as a pin joint with certain minor limitations. In the Author's opinion it was justifiably assumed by the designers that a hinge would form at the bottom of the spline wall by a slight horizontal crack between the soffit of the beam and the transverse girder on the one side and a slight plastic yield due to compression in the same place on the other side. With the formation of this hinge and the inherent flexibility of the pier as a cantilever almost no resistance was offered to temperature movement in the bridge, and consequently the temperature stresses would be very small.

130. No difficulty had been experienced in placing concrete in the precast beams. A carefully graded mix with a 3-in. slump had been used and there had never been any honeycombing to contend with. Special care had been taken in the accuracy of reinforcement fixing giving $1\frac{1}{2}$ in. outside cover and $2\frac{1}{4}$ in. between bars.

131. As stated in the Paper, the bridge had been designed for construction with the use of 10-ton derricks. This had involved careful selection of beam size for a reasonable length of span. The beam size chosen gave a satisfactory balance between weight, length, and number.

132. The maximum positive moment in these beams gave a working stress of 1,140 lb/sq. in., and the maximum negative moment at the haunches gave a working stress of 1,250 lb/sq. in. The beam test had been calculated to cause a stress of 1,580 lb/sq. in., which was 40% greater than the positive working stress and 26% greater than the negative working stress; the percentage of overstress on the material was therefore 26% and not 40% as indicated in the Paper, and the Author regretted the inexactitude. The theoretical deflexion had been calculated as 18 mm in the test load as against 11 mm creeping to 14 mm, which was recorded in the test. Admittedly the test had been a light one but it had been intended to do no more than demonstrate the adequacy or otherwise of a single suspect beam amongst many satisfactory units.

133. Mr Walley was correct in his assumption that the whole expansion of the bridge was taken up at the centre piers. The abutment anchorages had been designed to take any earth thrust plus tension due to movement of the bridge as well as any traction or braking effects. No rollers had been provided at the other piers and the fixity of the cross-members to the pierheads was regarded as nominal.

134. Basically the design loading was Standard Ministry of Transport loading, extended slightly to meet certain military requirements of the Iraq Government. Traction loading was also to Ministry of Transport standard.

135. The Author regretted that he could offer no opinion with regard to crack patterns which might eventually arise. Mr Goldstein should bear in mind that during by far the greater proportion of the life of the bridge the stress in the bottom steel, which resisted the positive moment, was only 8,000 lb/sq. in., or approximately half that which applied in full live-load conditions.

136. All the main-beam steel had been specified and delivered in lengths for use without splicing. The Author agreed with Mr Goldstein that splicing under such circumstances would have prejudiced the satisfactory placing of the concrete and would have turned a good design into an impracticable one.

137. With regard to the further division of costs given below, the Author contended that all such analyses should bear relation only to the bridge itself, complete with

	Total cost £	*Cost/sq. ft*
Single concrete span	5,200	2·5
Pier bent	8,800	4·3
Total of concrete span and pier bent	14,000	6·8
Centre span	38,000	8·3
Centre pier bent	18,000	3·9
Total of centre span and pier bent	56,000	12·2
Abutments (two)	68,000	2·7
Total of bridge and abutments	263,000	10·3

handrailing, electrics, etc., but excluding approaches, revetments, and ancillaries. Therefore, on this basis and for purposes of economic comparison, the cost of the bridge and abutments worked out to £10·3/sq. ft. He was impelled to add that he believed the contractors' rates for the work were fairly well distributed but very competitive for Iraq.

138. Mr Bartholomew had asked for more statistics on pile-driving results and penetration tests. Fig. 9 indicated the useful information available though the Author had admitted in § 22 that correlation was unsatisfactory. There seemed to be rapid changes in sand density on a horizontal plane which defeated comparison with piling results unless the pile was immediately adjacent to the borehole. The Author felt that it was outside the scope of the Paper to present other information not relevant to the matter under discussion.

139. The Author was surprised that Mr Pain should have been puzzled at the thickness of the tubes. The whole of Mr Pain's site organization had been delighted at their sturdiness. The thickness was largely a matter of design judgement to ensure driving without buckling and to afford a very long life against possible corrosion. Cases were known to the Author of two other structures in Iraq founded on reinforced-concrete-filled steel tubes of similar diameter, with walls only $\frac{3}{8}$ in. in thickness, and buckling of these tubes during driving had caused difficulties.

140. The composite pile—tube and joist—was regarded as being the most suitable for the strata which it was expected to encounter at Hindiya. This type of combination pile had also been used for the Samawa Bridge.

141. The steel piles were more expensive than the concrete piles, but it might have proved impossible or dangerous to drive the latter through dense strata. The medium and dense sands would probably have made jetting necessary and there would also have been the very hard clay layers to contend with.

142. Mr Haig was concerned about corrosion on the joists. Experience, particularly American, did not seem to bear out the fact that it was likely to occur at such a depth. Cathodic hydrogen was unlikely to be disturbed or otherwise removed by bacteria and a reasonable resistance to prolonged attack should be built up.

143. The caisson tubes had been driven with remarkable accuracy. In a very few instances it had been necessary to increase the clearance at the bottom of the shell by hacking away the inside rim and increasing the theoretical clearance, which was 4 in. at the sides and $1\frac{1}{2}$ in. at the ends. The shells had all been placed exactly in line both along and athwart the bridge, but the resulting pier bent may have been sometimes an inch out of square with the centre-line. This error had been taken up in the superstructure. The rapidity with which half the piers of the bridge had been placed and filled in a period of 2 weeks was proof that not much trouble had been encountered in fitting.

144. It had not been necessary to survey the piles each time for preparation of the bot om shutter, which was so cunningly contrived by the contractors' agent. Each side flap had been formed of a frame and three independent plates with hemispherical holes. Each plate had been attached by one bolt to the frame and could move laterally with respect to the other two. When there was some doubt about reasonable closure the gap had been plugged with bagged concrete. The initial 2-ft layer of tremied concrete never failed to seal the base against leakage of water.

145. Considerable thought had been given to treating the abutment pylons with "colourcrete", and it was decided that they would look better in natural concrete. The pylons were of precast blockwork treated with brushed acid on the faces, after casting, to show the aggregate. This had been found most difficult to perform and the result was not as perfect as one would have liked. The surface did not brush away either regularly or to any great depth. Perhaps the 25% solution of hydrochloric acid had been too weak.

146. Mr Bridge is probably correct in his belief that the deep channel in the river-bed would wander from side to side. The Author believed that it was as close to one

abutment as it was likely to go and that it was now tending to move across to the centre. The main abutment piles had been driven 30 ft below bed level into strata where the 4-ton single-acting hammer, dropping 4 ft, would drive them no farther, and they had stood up to the load tests extremely well. The front curtain of sheet-piling carried very little load and existed mainly to prevent the river from scouring away below the footing slab. It had not been considered necessary to provide any better protection or to drive these piles any deeper.

147. With reference to § 80 the arch rib had been designed to take all bending stresses due to unequal distribution of live loads.

148. Unfortunately, the provisions made for services were not indicated in the diagrams. In fact, however, ducts had been provided for future power and telephone cables and hangers under the deck for pipes. Transverse diaphragms between the beams would have complicated the construction of the work which had been designed for simplicity of execution. The design had rendered them unnecessary.

149. Mr Gullan had referred to the possibility of the river changing its course and of breaking away from its channel at some considerable distance up or downstream. While this was not unlikely, it would affect the whole economy of the locality and it could hardly be expected that a bridge contract should have to include for guarding against such an eventuality. A bridge was designed to cross a river, not to keep it located on its course except insofar as it might encroach upon the abutments. At Hindiya the river was flanked by the town on each side and both the municipality and the Irrigation Department were interested in ensuring that there was no encroachment on local amenities. The revetments extended an average of 300 ft upstream and 150 ft downstream of the bridge. Necessary extensions were either already in place or were now being built.

150. For curing, the deck had been covered with hessian and sprayed with water. Cement had been brought from Baghdad and had been of good quality and consistent. For most of the work aggregate of $\frac{3}{4}$-in. maximum size was used. The nominal mixes had been $1:1\frac{1}{2}:3$ and $1:2:4$ respectively, but the exact mix used had depended upon the grading of the aggregates. Taking into account the possible diversity of sources, the Author had found that the most effective way of controlling sand and aggregate quality was rigid inspection on delivery.

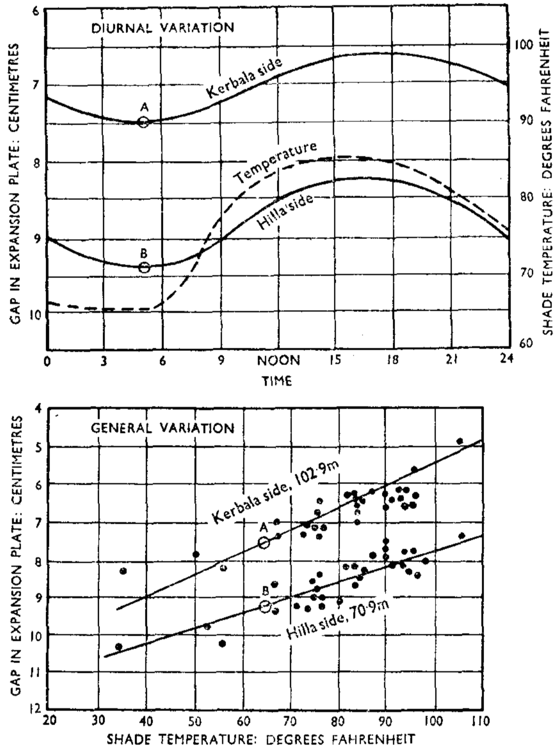
151. Zinc chromate primer had been used followed by three coats of proprietary paint.

152. Mr Austin had referred to thermal movements. The Author had meant that the bridge responded closely to all changes in ambient temperature. If the temperature rose 20°F between morning and afternoon then the bridge extended by $20 \times L \times 0{\cdot}000006$. The higher temperature of the deck surface from direct sunlight presumably balanced out the cooler temperatures within the body of the deck and girders. Fig. 22 indicated the diurnal and general variations encountered. The divergence of points from the line on the second graph might be partly due to the lagging effect when readings were taken on a rising and a setting sun. The temperature of the concrete had never been taken. All measurements were compared with shade temperature.

153. In reply to Mr Upstone, the Author stated that no vibrations had been noticed or reported in the arch hangers from any cause.

154. The Chairman had referred to the absence of general information relative to the assumptions made in the design of the bridge and as an example he had cited the centre span. The Author had purposely omitted much reference to this, believing that it was a fairly commonplace structure. The arch span had a trough decking supported on cross-beams which transmitted the deck load direct to the hangers. The tie-beams were formed from pairs of 17-in. × 4-in. channels with plated webs. Their depth was therefore $\frac{1}{66}$th of the span and the stress in them caused by deflexion of the whole truss would be correspondingly small. The adopted working stresses were those specified in the British Standard for mild-steel bridges.

155. With regard to § 95, the Author thought he had presented adequate information to check the theoretical deflexion. He believed this to be 18 mm compared with the



Slope of lines is 0·000006 x L cm/°F
Kerbala side includes the steel span

FIG. 22.—EXPANSION OF DECK

values of 11 mm rising to 14 mm which had been measured under test. He had avoided offering any correlation, for the derivation of the customary calculation was open to dispute.

156. Mr Brims had referred to the procedure adopted in clearing the pile tubes. The contractors had provided for breaking up the core and jetting, together with arrangements for removing the contents by air lift. They had experimented in the United Kingdom and believed the arrangement would be satisfactory. That this equipment had been unable to cope with the hard clay was unfortunate for them and had put them to a great deal of extra work and expense. Provided such work was done to the satisfaction of the engineer, it was scarcely necessary to have it stopped pending receipt of other equipment which might have been equally unsatisfactory. On an overseas job such difficulties had to be overcome with the use of available equipment at hand, despite the possible expense in labour and time.

157. All four compartments in each tube had been sounded prior to concreting. After placing the sealing plug the tube had been pumped out and could be visually inspected for cleanliness. There was no harm done if the walls of the bottom few feet of tube, which contained the plug, were not clean. The reasons for the thickness of the tubes had been previously given in this reply and the Author believed that the

remainder of Mr Brim's points had been similarly covered. It was incidental perhaps, but the shells had not been called cylinders, because they had no constant diameter.

158. In reply to Mr Harvey, whereas there had been no indication of splitting of pile toes, the Author could not say definitely that it had not occurred. If a pile were to split he assumed that there would be some difference in the reaction of the pile to driving, such as an alteration in the sound of the hammer blow or in some other form of unusual behaviour. There had been nothing obvious and the splitting might never have occurred. There had been no strengthening to the toes of the piles. Admittedly the ground was dense in layers, but since there had been no indications of boulders or large gravel, it had been felt that such measures were unnecessary.

159. A fair amount of re-driving had been done, owing to the necessity of extending some of the piles in situ. This had frequently happened the following morning and it had always meant a "hard" start. But two piles with sets of 50 and 53 blows per inch had remained for 3 weeks before re-driving had been carried out. Each of these piles had been re-driven for 14 in. at sets of between 100 and 200 blows for every inch. Mr Harvey had mentioned that he thought re-driving was a useful test. The Author felt that unless the engineer had had long experience of driving particular piles through particular strata, he should be uneasy of comparing such results with other tested piles which had been more normally driven. He felt that a truly comparative set would be measured only when the pile was actually on the move after being continuously driven from ground level or from soft strata.

160. The pier bents had, as Mr Harvey had suggested, been made flexible to accommodate the expansion of the deck. Calculation had indicated that the force required would not be very great. No side pulling tests had been carried out but it had transpired during construction that they were capable of swaying fairly easily under pressure from the pontoons.

161. Mr Harvey had also referred to the possibility of using smaller tubes inside the larger ones, instead of the joist piles. No serious thought had been given to this possibility and the designer's main alternative had been an extension in the driven length of the 18½-in.-dia. tubes while omitting the joists. Where even harder strata were located nearer the river-bed, on another bridge, this system was now being adopted. The Author was not sure whether Mr Harvey would have expected the inner tubes to be concrete-filled or left with a solid soil core. The core would doubtless compact very well but it might limit driving, for after a few feet it would descend with the tube. Further penetration would then necessitate some excavation.

162. On the Hilla side the revetment was in block limestone set in mortar, but on the Kerbala side brick had been used to match with existing work.

163. The 6-in. bottom-opening skip had been a simple affair consisting of a pipe about 5 ft long with lugs at the top for handling-wires. A central rod, with an eye at the top and a plate at the bottom, had provided the mechanism for restraining the concrete inside. The skip had been lowered by the rod at the end of a wire and when founded it had been lifted again for a few inches. The pipe-handling lines had then been belayed and the rod wire slackened off, thus permitting the concrete to depress the plate and flow out. A rich pea-gravel mix had been used. The skip had been handled by a small air winch.

164. Mr Morley had referred to the proposal in the Specification that the bridge be built by simultaneous construction from both banks with derricks at deck level and he had postulated that the contractors' method of construction by floating craft had contributed largely to the timely completion of the contract. Although the Author agreed with Mr Morley in this matter there were others who might consider his statement a little sweeping. Different contractors had as many approaches to construction as consultants had to design. Others might have made a success of working from beam level and the bridge had been designed to accommodate this method without hindrance to the opportunity of building from pontoons. The same considerations applied to Mr Morley's comment in § 112 regarding the driving of tubes through the shells rather than erecting the shells over the tubes.

165. The Author found Mr Edgecombe's remarks very interesting. The river was a little restricted below its normal regime width where it flowed through the town of Hindiya and municipal requirements were such that the bridge had had to conform with the existing embankments. The bridge piers now offered greater restriction to flow at this point but it was believed that their presence would not cause any excessively disruptive effects to the regime of the river-bed in the locality.