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DISCUSSION

Ahmad Shah Bridge, Malaysia

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Messrs D. W. Smith and A. R. Cusens, University of Dundee

We have been reminded recently¹¹ that a bridge is a hydraulic structure. At the site of this bridge two examples, the old bridge and the new, showed the importance of this fact. The Paper gives very useful information as to how the problem was tackled in designing the new bridge. Could the Authors enlarge on the determination of extreme bed scour at the river pier (§ 25)? It would also be useful to know more about the old bridge (§ 3); its type, level, span, depth opposing water, and mode of failure in 1961 and 1971.

65. The designers of the new bridge were indeed fortunate if they started out with such definite and clearly stated terms of reference as are mentioned in the introduction. Considering the conditions of the site, these terms of reference are very demanding, but without 'ifs' and 'buts'.

66. In view of events since 1969 it would be difficult to exaggerate the value of an example of a large steel box girder bridge, successfully completed, after having been designed over a period of only 6 months at the critical time mentioned in Table 1. The designers are to be congratulated on the skill and courage with which they took in what was useful in the nascent new rules, but avoided consequential doubts or delays. The stress comparisons given in Fig. 17, after little change had been found to be necessary as a result of the Merrison Interim Design Rules, give a valuable insight into the effect of those rules. It is almost impossible to obtain this insight in general terms by a simple comparison of codes. The good work in keeping within fabrication tolerances is also well demonstrated by Fig. 20. It would be interesting to know whether there was much inconvenience, cost or delay arising out of the necessary inspection and checking.

67. The absolute values of the temperatures mentioned in § 52 are, of course, higher than would be obtained in a more temperate climate, and there must surely have been times when working within the box girders became impossible. The differences in temperature between the sunny and the shaded side were also more, but apparently not grossly more than could occur in Britain.^{12, 13}

68. Was fatigue considered to be a problem? Were the transverse stiffeners fillet welded to the tension flange plate as well as to the longitudinal stiffeners?

69. After a careful discussion of the problem of slab thickness in §§ 33-34 (too often neglected, in view of the high cost of carrying the weight), the final answer does not seem to be clearly stated. It would be useful to have several additional dimensions relevant to Fig. 15: slab thickness, rib size, concrete cover, reinforcement diameter, and length of haunch. Is the deck provided with a waterproof layer below the surfacing?

70. The derivation of the live load shear distribution between the longitudinal ribs and the slab of the concrete T-beam system, described in § 34, appears to be based on a

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semi-empirical approach of debatable accuracy. It provides a marked contrast with the sophisticated and costly three-dimensional finite element technique used to determine diaphragm stresses. A simple two-dimensional finite strip or finite element analysis^{14,15} would have solved the rib/slab problem cheaply and easily.

71. What was the spacing of the high tensile screwed rods used to eliminate cracking of the concrete slab of the box girder edge? Was the restraint provided taken into account in the design of the slab?

72. The statement about avoidance of paint intercoat adhesion problems in § 56 reads strangely. Does this mean that the first site coat went on too soon to avoid subsequent damage, and that the later coat was needed mainly for appearance?

73. As this was a site where broken rock caused hard driving of piles, can the Authors say whether the use of universal columns in certain locations proved an adequate solution? Was the driving of these piles (§ 24) reasonably easy? Presumably the Authors were satisfied that there was no corrosion risk to the steel piles, as paint was applied only to reduce friction. Romanoff indicates that steel piles are normally safe below water table,¹⁶ but the exact position and level of the concrete caps is not clear from § 18 and Fig. 7.

74. In these days it is exciting to hear of a job completed without exceeding the tender sum. In view of the recent rapid changes in exchange rates it would be useful to have Authors' approximate translation of the cost into pounds sterling at the relevant date.

Mr M. S. Fletcher, Sir William Halcrow and Partners

I was particularly interested to see the maximum bridge temperature and temperature gradients quoted for the Ahmad Shah Bridge. These matters are well documented for bridges in the UK. For overseas bridges the designer still has the responsibility of choosing appropriate figures.

76. Experience gained during the construction of Al Garhoud Bridge, Dubai, UAE, has confirmed that in the hot areas of the world very high bridge temperatures actually occur. Al Garhoud Bridge is a five-span continuous structure, 160 m long,

Table 5. Values of temperature difference

	Air shade temperature, °C	Top flange temperature, °C	Temperature difference, °C
<i>Dubai</i>			
Al Garhoud Bridge	32	52	20
Al Garhoud Bridge	32	51	18
Al Garhoud Bridge (hessian covers over steel)	—	—	8
<i>Malaya</i>			
Ahmad Shah Bridge	—	65	33
<i>United Kingdom</i>			
DoE Tech. Memo BE 1/77 (40 mm asphalt surface included)	38	—	24
Steel box girder bridges Interim Design and Workmanship Rules, service conditions			
Unsurfaced steel deck	—	—	29
Surfaced steel deck	—	—	24

built over Dubai Creek. The superstructure is formed from steel box girders 1.5 m deep, acting compositely with a concrete deck. Approximate measurement of temperature were taken on the steel girders after erection and prior to placing the concrete deck slab, and the comparative values are given in Table 5.

77. There appears to be little doubt that had the measurements on Al Garhoud Bridge been made in the summer, when air shade temperatures of 50°C are recorded, a figure for unshaded steelwork equal to or in excess of the 65°C recorded in Malaysia would have been measured.

78. The figure of 33°C temperature difference is higher than other figures used in Britain and higher than those recorded in Dubai. This seems to indicate that with increase in exposed steelwork temperature there is also an increase in temperature difference. Please could the Authors comment on this and state the air shade temperature at the time the 33°C difference was measured.

Mr J. E. Gray, Inst. Technology MARA, Selangor, Malaysia

Regarding §§ 18 and 24, how and at what stage of construction was the 'poor sub soil' discovered, and could the Authors elaborate on its nature? Was there any difference between the penetration of the steel piles and the Frankipiles? How was the penetration determined?

Mr C. H. Teoh, C. H. Teoh dan Rakan

It is noteworthy that a steel box girder bridge of modern design should be constructed in a relatively remote area of a developing country to a tight programme without excessive problems and difficulties. Some of this can be attributed to the appreciation of local conditions and combining this knowledge with advanced international practice. For instance the steel panel sizes were specifically related to the mode of transport (particularly the rail sector), which in turn assisted in the formulation of the bridge proportions.

81. Temporary works were fabricated locally on site from available sections in Kuala Lumpur and from Singapore.

82. The erection pontoon was brought up river from the mouth at Kuantan, requiring considerable initiative in negotiating rapids and shallow places with the use of small temporary dams. This aspect was personally supervised by the contractor's Managing Director Mr Ng Teow Yong and is something of which he can be justly proud.

83. It was extremely helpful to station an experienced steelwork inspector in Osaka during fabrication and trial assembly. Subsequently the expatriate inspector was resident on site during steelwork assembly and his detailed knowledge greatly assisted the Japanese erection crew and the Malaysian workmen.

Messrs Lee and Wallace

We are indebted to **Mr Smith** and **Professor Cusens** for raising a number of interesting questions.

85. Without clear terms of reference it would not have been possible to prepare a design for this bridge within the time allocated. The contract also benefited greatly from the fact that apart from the changes necessitated by problems in the approach embankments (§ 18) and the addition of roadway lighting at a late stage, no significant changes were requested by the Public Works Department (PWD) during the construction period.

86. The contract value at March 1972 exchange rates was approximately £1.6 million. There were no price fluctuation clauses which, due to the inflation which subsequently occurred, was to the advantage of the PWD.

87. As far as we are aware no details of the earlier bridge have been published, and the following notes are taken from information put at our disposal by the PWD. It was

built at a cost less than one tenth that of the present structure to a design which accepted the economic disbenefit of its being unservicable, by submergence, for limited periods, and also the risk of damage by floating debris. It was not a unique structure.

88. Following the decision that it should be a submersible bridge, the behaviour of the river was studied over a number of years with particular regard to the large amount of debris which was known to be carried during spates. It was concluded that the concentration of debris did not build up to serious proportions until the water level had risen about 7 ft above its normal level. The quantity tended to increase to a peak as the upper reaches of the catchment became flooded. It then diminished significantly while the water level at the bridge remained high and little debris was present on the falling river. This last observation conflicted with earlier recorded impressions of local people that the maximum concentration of debris occurred on a falling river.

89. From records for a 10 year period it was deduced that a bridge with a deck 10 ft above normal water level would be closed on average 13 days/year while one 15 ft above would be closed on average only 4 days/year (and in some years not at all). The latter degree of interruption was considered acceptable and the deck level was fixed at +100 ft OD.

90. The bridge comprised 17 spans of approximately 40 ft, all but one of which were built from precast prestressed concrete beams. The span at the Temerloh end was of Bailey bridge construction which could be withdrawn if required for navigational clearance. The piers supporting the deck were 2 ft thick concrete walls carried on a single row of 10 precast reinforced concrete piles driven to a set in the alluvium of the river bed. Deck and piers were designed to present a minimum obstruction to the river in spate.

91. The bridge was virtually completed when it was first submerged to a depth of 3 ft. The river had risen very rapidly and debris containing a high proportion of cut logs built up on the upstream face and also under some spans. Despite the river falling for a few days to +87 ft OD the debris remained trapped. Then there was an abnormally rapid rise in the river which submerged the deck again. The maximum depth of debris piled up against the bridge was estimated to be between 20 and 30 ft and its weight thought to have been about 10 000 tons. Next day three piers and four spans were carried away.

92. Eyewitnesses standing on spans near the breach stated that there was no noise or disturbance of the bridge when the spans were lost. The only indication that the bridge had failed was that the debris started to move through the gap. It was subsequently deduced that the most likely cause of the breach was the scour resulting from the constriction of flow caused by the debris. The piers appear to have toppled under the horizontal and uplift forces and carried the deck with them. As indicated in the Paper (§ 22) the piles had not been driven to rock.

93. Four further spans, adjoining the first gap, collapsed in a generally similar manner in 1971 but with the river at a level of +126 ft OD.

94. The depth of material overlying bedrock at the new river pier was of the order of twice the calculated Lacey mean depth for the 1 in 500 year flood, and was of uniform quality. It was decided therefore that no further scour calculations need be made but that the pier stability should be checked, assuming the overlying material to be entirely scoured away. The foundation cylinders were socketed into bedrock (§ 23) but rock anchors were not found to be necessary.

95. We agree, of course, that a finite strip analysis of the concrete deck would have given a more correct solution, but such a program was not immediately available to us and our three-dimensional finite element program was not particularly suitable for this particular problem. Before the 'semi-empirical' solution of the shear load distribution quoted in the Paper was adopted, a number of computer grid and manual solutions had been completed and the stage reached where the areas of bending reinforcement necessary were known but it was desirable to confirm that the maximum shear in the rib would be significantly reduced by distribution through the slab haunch

between the ribs. The calculation quoted in the Paper was considered to be adequate for this purpose.

96. Deck details were as follows:

Slab thickness	5 in
Rib spacing	24½ in centres
Rib width	8½ in tapering to 7½ in
Rib depth	10½ in
Reinforcement top cover	1 in
Reinforcement bottom cover	1½ in
Length of haunch	16 in

There was no waterproofing layer between the surfacing and the deck slab.

97. The pairs of high tensile screwed rods referred to in § 36 were in the cross girder end connections only, i.e. at 7 ft 11 in centres. They were prestressed before the deck slab was cast so that there would be no separation of the steel box/cross girder interface under live loading to crack the slab. They did not provide any direct edge restraint for the slab.

98. Fatigue did not govern the design although the use of stud shear connectors on the top flanges reduced them to Class F. Transverse fillet welds were generally avoided on tension flanges by the introduction of intermediate plates. The cross-section shown in Fig. 14 is taken at a position where the bottom flange is in compression. Unfortunately space limitations did not allow us to include additional sections and Fig. 16 is too small for the stiffener and joint arrangements to be properly seen. In boxes where the bottom flange is in tension the longitudinal T stiffeners were bridged by transverse channel sections although the vertical web stiffeners were carried down and welded to intermediate plates on the bottom flange which were only welded longitudinally.

99. Mr Teoh has commented briefly on fabrication inspection in Osaka. The British inspector, although not Japanese speaking, was able to work closely with the Contractor's inspector. He was resident for about 7 months. During a short period when work was proceeding in two shops several miles apart he was assisted by a local inspector. The Japanese fabricators appeared slightly apprehensive about the specified tolerances, but in the event he did not seem to find it excessively difficult to work within them. On the other hand they did not appear to like 'British Standards', so it may be concluded that they had incurred additional costs in meeting them.

100. At site, temperature conditions did make work inside the boxes very difficult at times. Adequate ventilation was never provided by the Contractor which made painting, in particular, very unpleasant even at night. This is almost certainly one reason why a large amount of remedial painting was required inside the boxes. Much of the final bolt tightening was also done at night.

101. We cannot agree with Mr Fletcher that temperature differential increases with the temperature of the exposed steelwork. Model experiments at Temerloh and measurements on the steelwork confirm that temperature differential is influenced by the shape of the box. The heat soak effect of the web must be related to plate thicknesses and areas. The shade temperature was fairly constant and was not recorded with the particular maximum differential quoted but would have been about 30°C. Normally there was very little wind at the assembly site.

102. The UC piles were driven without difficulty and the required set was achieved at a penetration of about 50–60 ft which is above the level of broken and decomposed rock. The bottom 25 ft of the steel pile were left unpainted and corrosion is not expected to be a problem in this ground.

103. There was a delay of about 18 months between the application of shop and site paint coats. This was very much greater than had been expected, and although the initial coats stood up well to handling an extra finishing coat was recommended:

(a) because of the increased hardness and smoothness of the epoxy undercoat;

- (b) to minimize moisture penetration of the finishing coats which would be induced by the increased polarity of the coating.

104. In reply to **Mr Gray**, a pre-tender borehole showed a 20 ft thickness of very soft organic clay at a depth of about 15 ft under the west approach viaduct. This layer was not present in another hole about half way along the approach embankment. The embankment, which was a maximum of about 45 ft high, was therefore kept under observation and some movement had been noted prior to the slip arch type failure. In the Authors' opinion it would have been preferable had all works connected with the bridge been under the responsibility of the Consulting Engineers, rather than the design, supervision and construction of the embankments being separate from those of the bridge.

105. We do not have a completed record of the depths of the Frankipiles but these were generally in the range 40–50 ft compared with the steel piles which were 50–60 ft. Both were designed by normal formulae to have a capacity of 100 tons and were test loaded.

106. In conclusion it should be mentioned that the bridge was opened to traffic on 24 October, 1974. The bridge has the honour to derive its official name from HRH The Sultan of Pahang who performed the ceremonial opening on 17 May, 1975.

Corrigenda

Caption Fig. 12: for 'viaduct on west side' read 'viaduct on east side'. § 37, last line: for 'Fig. 15(a)' read 'Fig. 14'.

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