Bridges in Malaysia: Santubong Bridge, Sarawak; Penang Bridge; Sungai Perak Bridge

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Proc. Instn Civ. Engrs Structs & Bldgs, 1992, **94**, Feb., 117–125

Papers 9490, 9491, 9492, 9531

Discussion

T. H. Hanna, *University of Sheffield*
Could Mr McCabe comment on the value and validity of the Chin plot that is referred to in Papers 9491 and 9492: a method of extrapolating a pile loading test to give an ultimate load prediction? Could he also comment further on the philosophy of the design of the piled approach embankment? At the time of the design of that piled embankment, the mathematics of embankment design had not been worked out. In fact, they were finalized in 1984 by a final year student in the tripos exams at Cambridge.

2. I am amazed that raking piles have been used, and I cannot see the fundamental necessity for them. They are very difficult to drive, and I do not think one achieves any greater efficiency or lateral load resistance. It might be much better to use conventional piles.

3. I want to focus on §§ 89 and 90 of Paper 9490, which describe the piled embankment. A tripile is referred to; my belief is that most people are not familiar with the tripile system, and I think it would be useful to bring it to the attention of consultants and contractors in the UK.

4. Concerning the tying of the pileheads together in the piled embankment, I think it would be useful if a sketch of the pile cast could be given, because there have been, to my knowledge, two catastrophic failures of piled embankments less than a couple of thousand kilometres from this particular site in Sarawak; in one case, 10 000 piles failed in an embankment in the late 1970s. It would be useful to know how that piled embankment was analysed. I understand that there is some instrumentation which shows that the mathematics worked out by Hewlett, a Cambridge student, have been proven to be precisely right. How did the Author deal with the pile cap actually punching through the fill on top? This is one mode of failure that has to be anticipated and dealt with.

A. G. Simpson, *Mott MacDonald*
With reference to the Penang Bridge Project, I was interested to see the approach of Mr McCabe to the aerodynamics of the structure which seemed to be extremely economical. By making a comparison with the Dame Point Bridge, he was able to avoid the expense of a set of wind tunnel tests. I was surprised, however, at the relatively high speed—60–120 km/h—quoted for vortex shedding. I would have expected that to be lower. Has he, in fact, made any site observations or measurements, or is there any other instrumentation on the bridge itself?

6. I like Mr McCabe's neat arrangement of the cables at the tower junction, whereby the double cables in the side span were able to go outside the single one in the main span. Were all cables of the same size and form, and are they, in fact, anchored side by side in the side span at deck level or, as appears in Fig. 5 in Paper 9492, anchored at different longitudinal positions?

7. The protective works around the main tower were massive, but there was nothing around the approach viaduct. Was the water so shallow there that shipping was thought not to be able to reach those piers, or have special arrangements been made for repairing damage to either the piers or the spans?

M. S. Fletcher, *Sir William Halcrow*
With regard to Papers 9491 and 9492, I was interested in the permissible stress in the bars of the cable stays of Penang Bridge. The quoted figures are 0·55 under axial forces and 0·6 under axial forces plus bending. What was the origin of those figures? There has been a tendency in the UK to use lower figures, e.g. on cables made from prestressing strand, figures of less than 0·4 are generally adopted. It would also be interesting to know whether or not the 0·6 factor has been changed in the code of practice which was used for Penang Bridge.

9. Another point concerns the large man-made islands which protect the main towers. I am an enthusiast for using islands to protect bridges but appreciate that the design is quite difficult. Were any model tests performed, and could the size of the vessels and their velocity in the channel adjacent to the islands be given?

10. With regard to Paper 9490, one lesson arising from the Santubong Bridge is that the thermal gradients in the pile caps had a limiting criterion of 20°C and this was exceeded in a vertically downward direction into the flowing water.

11. Some years ago, I instrumented a 4500 m^3 continuous pour on a pile cap on the

Papers published: Proc. Instn Civ. Engrs, Part 1, 1990, **88**, Aug., 505–599

* *Deceased*

ground, and found that one could control the temperature on the surface with quilts and on the sides with insulated shutters, but again the 20°C criterion was exceeded in a downward direction. The lesson is that on the next big pile cap, polystyrene insulation on the underside is required. This is one of those parts of the bridge that can never be inspected afterwards.

12. I think the other message from Paper 9490 is that the need to supervise the site investigation is at least as great as the need to supervise the concrete. I have found that this is a very difficult message to put across overseas.

13. With reference to Papers 9490 and 9531, the two balanced cantilever bridges are a sophisticated form of concrete construction. The techniques used to be purely European but they have spread tremendously quickly worldwide. In 1984, I published a paper which indicated the learning curve of teams using cantilever shutters on this type of bridge. It is interesting that on these Malaysian bridges a similar learning curve has been achieved. I think this is a great credit and a significant change in the world construction scene. Do the Authors think that Malaysian contractors would now be able to build these bridges without foreign input?

J. J. Hart, *Mott MacDonald Group*
From Fig. 6 of Paper 9491, the stand-off distance for an impacting vessel is not very large when compared with the size of the pile cap. With the fill placed around the pile cap and the piles, it would appear that the force from an impacting vessel will transmit through the fill to the foundation. Were any comparisons made with the magnitude of likely forces arising from seismic effects?

15. Looking at the seismic spectrum, it would appear that for a structure with a reasonable response time, the seismic forces are quite low—in fact, the *g* force is probably down to 0·03 or 0·05. Could it be that the seismic effect would be less than the effects of shipping impacts?

16. Another question relates to the superstructure, namely the compound behaviour of the stays and the surrounding shroud tube. I followed from Paper 9491 the intention of the transfer of load from the participating shroud through the rivets as they are termed. Was any consideration given to the use of short shear connectors? In addition to the tests described in the Paper, were any tests made for pull-out in terms of crack control on the concrete anchorages surrounding the shrouds?

17. The relatively low wind speeds at which vortex excitation can occur would appear to correspond to the upper range of the operational occupation of the structure by traffic. Is a limiting windspeed forseen for traffic using the structure?

P. J. Andrews, *Member*
The bridges described have used mainly local materials and local labour. They have been designed probably with the government directing but with the use of the people in mind, and they have been designed using these people's skills. I think it is vitally important that engineers are sympathetic to the environment and the economy of the country in which they are operating.

19. With reference to the Sungai Perak Bridge, I was interested in the pile tests there because I think it was stated that the piles adjacent to the test pile were used to do the monitoring. How was a firm reference point obtained? I would have thought that in a pile cap so close together, all kinds of unusual drawdown would occur. I also think that there were ducts inside which plastic sleeves were put, which stiffened the steel liners that went through. Why were the plastic ducts not used alone?

20. With regard to the Santubong Bridge, there was a problem with the grouting at the end where the tendons actually moved in. How did that affect the grouting that was in position for the tendons, as I assume the tendons were grouted at that point? What tests were carried out to find out, and were other measures required to check the grouting elsewhere? On the Penang Bridge, the classic way of grouting or of getting concrete not to bubble was to start at the bottom and to allow the concrete to push the air out rather than to push the air around a curve, which would leave pockets of air behind.

21. There have been instances in which prestressed bridges have collapsed as a result of corrosion in prestressing tendons. How much management of the future maintenance of the bridge was built into this design? That is an essential part of whole-life costing and indeed of getting bridges of that scale to function. One cannot simply go back and replace a bearing unless that bearing has been designed to be replaced.

R. E. Williams, *Mott MacDonald Civil*
A common feature of all three bridges is the need for extensive piling to transfer foundation loads through weak near-surface deposits. At Sungai Perak, foundation conditions were particularly adverse because of the great thickness of alluvial and marine sediments and because of the difficulty in obtaining reliable geotechnical data.

23. Investigation within the main channel was a hazardous business, with water depth up to 20 m, a 4 knot current and the occasional oil tanker. One deep borehole was put down at each pier to a depth of approximately 80 m, but bedrock was not conclusively encountered, although rock has been proved at about 85 m depth on each river bank.

24. The ground profile is relatively consistent across the whole site (Fig. 2 of Paper 9531) but the upper thick clay stratum was interspersed with bands of clay and stiff peat or lignite, two of which appeared to be laterally continuous. The principal uncertainties that remained on completion of the site investigation with regard to the sand stratum were the frequency and thickness of clay and peat layers, and the relative density, as there was a significant spread in the reported SPT *N* values.

25. On the basis of this ground information, the pile arrangement adopted for the bridge (Fig. 4 of Paper 9531) consisted of

(*a*) 386 mm square 35 m long driven precast concrete piles of 750 kN capacity for the viaduct piers
(*b*) 1·8 m dia., 45 m long open-ended tubular steel piles for the ring beams
(*c*) 1·5 m dia., 45 m long open-ended tubular steel piles for the main river piers; these piles have a working load of 5200 kN.

The performance of the piles for the ring beams and viaduct piers was generally in accordance with the design predictions, but this was not so for the piles supporting the main river piers.

26. The tender design was based on piles driven through the first clay layer to −45 m OD within the medium dense to dense sands, the soil plug being cleaned out to within 5 m of the base and the pile being concreted for the full depth. The first pile load test on the south pier was conducted without the installation of the concrete plug so that the magnitude of skin friction resistance could be evaluated (Fig. 1). The load–settlement plot shows a sharp break at the point where the skin friction load was exceeded. Back calculation gave similar skin friction parameters to that used in design. However, the second test conducted after the pile had been fully concreted showed that little, if any, end resistance had been mobilized.

27. In view of this unexpected result, four options were considered. Base grouting was rejected as being too uncertain. The use of a plug inducer was given serious consideration, but was rejected again because of uncertainties in the performance. This left two courses open: either to drive to bedrock, which was the safe option but would have significant implications on both cost and programme, or, alternatively, to ignore end bearing and to drive to a depth such that the entire load could be generated by skin friction alone.

28. Since it appeared that the skin friction parameters had been confirmed during the first test, the latter course was adopted and the second pile test was conducted at the north pier on a pile driven to −60 m OD into what the site investigation had indicated was dense sand. However, contrary to expectations, little increase in driving resistance occurred between −45 and −60 m OD and, in the event, the response to the pile under load was similar to the test at the south pier. The estimated skin friction values, based on the API (American Petroleum Institute) guide-lines, are shown in Fig. 2. It is evident from the load–settlement curve that the predicted increase in skin friction from the dense sand beneath −40 m OD did not materialize.

29. In view of the critical importance of achieving a reliable foundation for the balanced cantilever construction, the only feasible option at this stage was to extend the piles to bedrock. By doing so, it was possible to up-rate the working load from 5200 kN to 7800 kN, thereby eliminating four piles from each group. The test pile on the north pier was redriven to approximately 85 m below datum, and its satisfactory performance was confirmed by a further pile test.

30. In retrospect, it is evident from the driving records (Fig. 7 of Paper 9531), that the

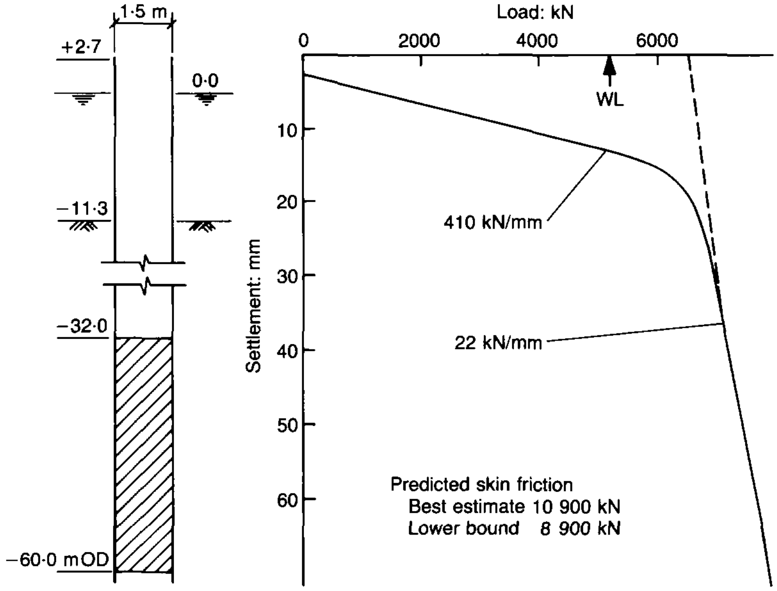

Fig. 1. Test pile 2: north pier

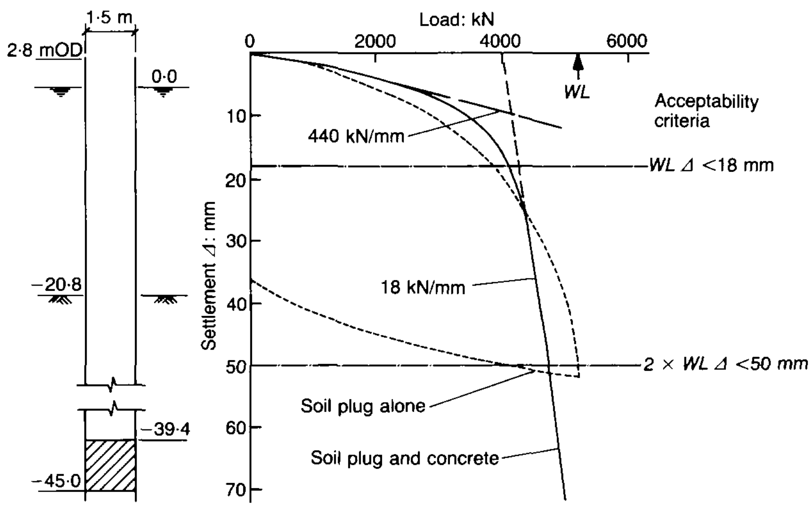

Fig. 2. Test pile: south pier

density of the sand stratum below −40 m OD was less than that suggested by the site investigation. The skin friction resistance was substantially lower than anticipated from the API recommendations, and also the strains to mobilize the end resistance were substantial.

C. T. Harris, *Ove Arup and Partners*
Could Mr Buckby comment on the decision regarding the position of the abutments with respect to embankment length? It was necessary to pile the embankments and there is reference in the Paper to flooding. Was there any justification for extending the approach embankment spans, which incidentally are, at 28 m, perhaps shorter than they might have been?

32. Turning to the large provisional sum for the piles, does Mr Buckby have any comments, particularly with respect to the assessment of the contractor's programme?

33. Why was transverse prestressing of the top flange necessary on a cantilever length of approximately 2·5 m?

N. L. Sadler, *Cass Hayward & Partners*
My first point on Paper 9490 relates to the agreement between the observed and predicted deflexions, particularly in the main span of the balanced cantilever construction. The measured deflexions were taken immediately after the post-tensioning and I wonder if any subsequent measurements have been taken, particularly in the middle of the main span, to determine the more long-term correlation. What levels of tolerance were actually achieved at deck level, in view of any variation in the correlation? Was it necessary to make special provision to ensure that the final road levels were actually as required?

35. Was a structural steel form of deck considered during the early stages in each case before tender? This is not a question without background, in that one is aware that the question of steel construction is being increasingly considered in south-east Asia. The point of the question is that on both the Santubong and Sungai Perak Bridges there appear to have been problems experienced during the piling stage, which may have resulted in delays on the Sungai Perak Bridge, and which caused delays on the Santubong Bridge. With the in-situ concrete construction, the piling is critical to the programme and progress of the job, and perhaps the delays would have been minimized by the use of steel or other form of prefabricated construction.

N. Meadows, *Taylor Woodrow Construction*
With regard to Paper 9531, I was interested to see that steel circular props were used in the temporary works. While there is an excellent load-bearing member acting on the temporary works, are the tubes filled, for instance, with concrete or with sand? If they are filled, what happens to the prop afterwards; if they are not filled, how should the bearing stresses at each end be dealt with?

37. If one has circular props then there is a situation where one has to brace, and bracing members are usually of a rectangular section, so there are problems with trying to tie bracing into a circular section. Could Mr Stanley convince a contractor that circular props are perhaps the best way forward?

P. L. Hugo, *Mackintosh Bergh & Sturgess, Pretoria*
Paper 9490 does not refer to a preliminary economic evaluation of the bridge project and its approach roads. Now that the installation has been in operation for two years, could the Author give some indication of the average daily traffic and its composition?

39. With regard to Paper 9491, could the Author indicate the nature of the preliminary studies on sea traffic between Penang and the mainland which led to the decision to build the six-lane bridge? The bridge has been open for five years. Have the traffic projections been realized?

40. With reference to Paper 9492, the cross-section is described as providing for three lanes in each direction, separated by a median strip. There appears to be no provision for emergency stops or for inspection, except by using one of the working lanes. What provisions have been made for dealing with vehicle breakdowns on the 13·5 km of bridge?

Sir Frank Gibb, *Ross–Gibb Consultants*
As these works were carried out in the open sea, what was the effect of the sea conditions on the piling? Were calm seasons chosen to do the various parts of the work?

A speaker
The 900 m^3 of grade 40 OPC concrete on the pile caps for the Santubong Bridge was quite an achievement. However, in retrospect, if Mr Buckby were starting afresh now, would he design the piers differently so that he could do it in two pours of half as much? Then all the problems with temperature and so on would not occur. If he had another chance, would he redesign the pier caps?

Mr Buckby and Mr Sim
Professor Hanna raises a number of points on the design of the piled embankments. The first refers to the precast concrete tripiles. The name Tripiles is a patented reinforced-concrete pile of triangular cross-section, which is produced by Pilecon Engineering Bhd, Malaysia. The approach embankments were constructed by a

Sarawak based contractor working in association with Pilecon under a separate contract to the bridgeworks. The tripile embankment design was proposed by the contractor as an alternative lower cost design to the traditional bakau timber piles that were specified by the Engineer and are used commonly in Malaysia.

44. The piles are currently produced in three sizes. The two larger sizes (designated T35 and T25), used on the approaches to the bridge have working load capacities of 52 t and 37 t respectively. They are easily cast in V-shaped steel moulds and have a light prefabricated reinforcement cage which can be handled by two men. The steel end plates are anchored into the concrete body of the pile and have three holes to allow adjacent lengths of pile to be bolted together quickly during driving. The piles are generally cased in 5 m lengths for ease of transport and handling, but are also produced in shorter lengths to allow the optimum driven length of pile to be achieved. The tripiles for the Santubong approach embankments were driven by $1\frac{1}{2}$ ton and 2 ton drop hammers to depths of 30 m through soft silt to highly weathered mudstone. Two piles in each embankment were load tested to 1·5 times their working load. The maximum settlement recorded was 25 mm.

45. *Professor Hanna's* second point refers to the layout and tying of the pile heads to prevent lateral failure. The pile lengths in each embankment varied uniformly from a maximum of 33 m immediately behind the bridge abutments to 9 m at the low end of the embankment. The piles were spaced at 1·8–2 m centres: the outer 2 or 3 rows were raked and the inner piles were driven vertically. A reinforced concrete pile cap was cast on each pile. Lateral failure of the piles supporting the higher section of embankment was prevented by tying the pile caps together laterally and longitudinally with lightly reinforced concrete tie beams.

46. With regard to the analysis and design of the piled embankment, the spacing of individual piles was determined by the loading imposed on a single pile by the embankment fill and live loading. The pile caps were sized using the recommendations of the Swedish Road Board. These are intended to ensure that the embankment fill material arches between pile caps and does not fail in punching shear under the action of the pile load. The pile caps themselves were designed against failure in punching shear and bending. The pile length was determined by geotechnical considerations for safe working load and for limits on the immediate plus long-term consolidation settlement of the pile group.

47. On the instrumentation of the embankment, at his own expense, the Contractor installed pressure cells above the pile caps and between the pile caps, deep settlement markers and surface settlement markers. Monitoring was carried out over a period of one year, and confirmed that the vertical pressure in the embankment between the pile caps was being relieved by arching action; the actual settlement of the embankment was everywhere less than the calculated grouped settlement of 47 mm behind the abutment and 168 mm at the low end of the embankment.

48. *Mr Fletcher* observes correctly that the design thermal gradient in the 900 m^3 concrete pours for the pile caps to the main piers had exceeded the limiting criterion of 20°C in a downward direction. His suggestion that the temperature gradient could be better controlled by placing an insulation layer below the underside of the pile cap is useful, but I would not recommend the use of polystyrene sheet unless it was backed by plywood or a protective membrane which was sufficiently robust to prevent damage from occurring during steel fixing and cleaning out.

49. His question on the ability of Malaysian contractors to construct cantilever bridges without foreign input is interesting. Local contractors in Sarawak do not yet have the experience or skills to take on a project of this type without external assistance, but they are able to provide good subcontract labour. In peninsular Malaysia, there are four or five national contractors of sufficient size and experience who could successfully undertake a project of this type without external assistance, but most would probably recognize that they would benefit from limited specialist inputs.

50. *Mr Andrews* asks how a firm reference point was obtained for monitoring settlements of the test pile. Level datum reference points were established on either temporary piles driven for the purpose, or permanent piles that would not be subject to load changes during the period of test. In either case, the reference pile was located outside the zone of influence of the test pile.

51. His second point refers to the PVC rigid liners used in prestressing ducts. They are used only in the segment under construction where their function is to prevent collapse of the corrugated metal sheathing and to limit accidental grout ingress during concreting. They would be too rigid to follow the curved profiles of the tendons. As far as we are aware, PVC semi-rigid spirally wound ducts have not been used for prestressing, but provided that they were sufficiently robust to withstand site conditions and rigidly fixed in position without suffering flotation during the placing of wet concrete, we see no objection to their use.

52. With regard to local spalling failure of the webs at the ends of the anchor spans during stressing of the continuity tendons (see § 64), the failure occurred during stressing, and at this time none of the adjacent tendons in the

web had been grouted. It was therefore a relatively straightforward operation to de-tension and remove the continuity tendons affected by the web failure, before rectifying the failure and installing new strands. There were no problems with grouting, other than those which were experienced and resolved in the trials carried out in advance of permanent works grouting.

53. *Mr Andrews* also refers to post-tensioned bridges which have collapsed because incomplete grouting has led to corrosion of prestressing tendons. The success of grouting depends on the level of supervision exercised on site. Grouting trials are essential before permanent works grouting starts in order both to identify and to resolve any problems of mix design and grout flowability, especially where grout is being injected in temperatures of 30–35°C; but close supervision of the progress and volume of grout injected into each duct is equally important. Even then, it is impossible to say that a duct is 100% full of grout. Mr Buckby has sectioned grouted ducts from closely supervised trials carried out for other major bridges and has found small pockets of entrapped air on the crest of vented ducts. Radiography is not successful in thick concrete sections and is so limited in thin sections with regard to the extent of the area that can be examined as to render it useful only for detecting voids where they are already suspected to exist.

54. *Mr Andrews's* final comment on the need for engineers to pay more attention to maintenance in the design and detailing of bridges is fully endorsed. The lessons learnt on this from the rapid construction of motorway bridges in the UK in the 1970s are, or should be, only too evident to every bridge engineer.

55. *Mr Harris* asks about the position of the bridge abutments and the span length on the approach viaducts. Given the deep deposit of soft silt over which the bridge and embankments were constructed, we believe that the length of embankment adopted was about right in relation to the height of embankment to be supported and to the relative unit cost of piled embankment compared with the cost of the approach spans. However, there were other physical constraints, namely a drainage channel, flood protection bund and access road which determined the abutment positions. The 28 m span length was adopted to suit the lifting capacity of locally available carnage. Longer spans would have involved heavier lifts and would have proved difficult on the soft ground conditions at the site.

56. The provisional sum for piling was a peculiarity of the method of measurement used by the Sarawak Public Works Department. However, this did not affect the Contractor's ability to programme the works at tender stage, because a bill of quantities was provided for the bridge piling and the final measurement for piling was within 5% of the tender quantities.

57. The provision of transverse prestressing in the top flange should be addressed to the designers of the bridge, T. Y. Lin International. In our view, it could have been omitted on a cantilever slab, 2·5 m long, and would have saved one day in the construction cycle of each segment. However, practice varies from country to country, and as the original specification made no provision for waterproofing the bridge deck, transverse prestressing could well have been justified by the designer for durability reasons as a method of preventing water ingress and corrosion at longitudinal flexural cracks in the top flange. In the event, a cold applied bituminous waterproofing material was applied to the top flange before the asphaltic cement wearing course was laid.

58. *Mr Sadler* comments on the long-term deflexions and construction tolerances achieved on the bridge deck. Meaningful measurements of long-term deflexions due to creep and shrinkage on the completed bridge deck cannot be obtained unless levels are taken under identical conditions of mean bridge temperature and thermal gradient. Similar conditions can be verified only if sufficient instrumentation is provided in the bridge. The necessary instrumentation was not provided in the case of the Santubong Bridge. The tolerances on level achieved during cantilevering are shown in Fig. 10 of the Paper.

59. After the continuity tendons had been stressed, the as-constructed profile of the bridge deck was surveyed and the design finished road levels were adjusted where necessary to achieve the minimum depth of wearing course consistent with an acceptable vertical alignment. Small areas of regulating course were laid before machine laying of the 40 mm asphaltic cement wearing course.

60. *Mr Sadler* refers to the consideration of steel as an alternative material for the construction of the bridge deck. The policy of the Malaysian government at that time, and currently, was to make maximum use of local labour skills, materials and components produced in Malaysia. A steel plate or box girder bridge would have required the import of large quantities of structural steel sections or steel plate, and the unnecessary employment of foreign labour. The possibility of a steel bridge was therefore precluded from the outset, but, in our view, its exclusion was correct in terms of the Malaysian national economy, and at this site it did not prevent the designers from realizing the most economic form of construction. The nine-month delay in the completion of the bridge, which was caused almost entirely by the prolonged delays in piling, would not have been reduced significantly, if at all, if the deck

had been composite plate girders. The concrete box girder superstructure of the three river spans, including the hammerheads, was constructed in just nine months. Irrespective of whether a concrete or steel superstructure is being used, piling of the main river spans is almost always on the critical path for major bridge crossings over navigable waters.

61. *Mr Hugo* notes the absence of any reference to an economic evaluation of the bridge project and the approach roads. The economic evaluation of the bridge and approach roads was made by the Sarawak State Government who financed the project, and was not within the consultant's terms of reference. The scheme provided the sole highway link between Kuching, the state capital, and a recently completed international hotel and beach resort. The economic evaluation almost certainly relied on the realization of a substantial increase in income from tourism, greater employment opportunities, land development and benefits from improved agricultural usage. We do not have survey data on traffic volume and composition, but from our own observations, the traffic volume on weekdays is very light, but increases at weekends to an estimated 4000–5000 vehicles/day, which is wholly local traffic composed mainly of private cars, motor cycles and light vans, attracted by the new facilities offered at the beach resort.

62. *The last speaker* asks if, in retrospect, we would cast the 900 m^3 concrete pours for the main pile caps in two separate lifts. Our reply is definitely not. The restraint afforded to early thermal contraction of the second lift by the previously cast section would induce excessive cracking in the second lift. Control of early thermal cracking in large concrete elements placed in a single pour is now well understood, and if planned and managed properly is very successful.

Mr McCabe

In reply to *Mr Hugo* (§ 40), vehicle breakdown areas or layby areas (4 m × 40 m) are provided along each side of the bridge at a spacing of approximately 1200 m.

64. In answer to § 39, preliminary studies indicated that peak hour traffic on the ferries would grow to 3000–4000 passenger cars by 1990. This was far beyond the capacity of the existing ferry system. Additional capacity would necessitate substantial investments in additional ferry terminals and road systems at the terminals. Although the existing ferry has remained in service, it is my understanding that traffic projections have, in fact, been realized.

65. In answer to *Sir Frank Gibb*, the sea in the harbour is generally calm because the area is well sheltered from ocean waves and swells. Except during the short-lived thunderstorms, the piling work was affected little. Precast box forms were used for the footings at water level. After the piles were driven and surveyed, the forms were cast with the holes in the bottom slab located accordingly. The form was then set over the piles, the space between the form and pile plugged and the footing cast. This greatly assisted the works in the harbour.

66. In reply to *Mr Fletcher*, at the time of the design of the Penang Bridge there were limited specifications governing the design of cable stays. Practice at the time was to use allowable stresses of $0{\cdot}6\ F_y$ for steel tension members. Taking $F_y = 0{\cdot}86\ F_u$ for the high strength bars results in an allowable design stress of $0{\cdot}55\ F_u \div 0{\cdot}86 = 0{\cdot}64\ F_y$. This allowable stress is justified by

(*a*) a high fatigue strength of the cable bars
(*b*) a high degree of corrosion protection
(*c*) cable bars being individually stressed and thus bar forces known accurately
(*d*) complete analyses for calculations of bending stresses at cable anchorages
(*e*) a very large design live load.

Additional checks on the cables were made for

$$1{\cdot}4\ \mathrm{DL} + 2{\cdot}2\ \mathrm{LL} > 0{\cdot}9\ F_u$$

$$1{\cdot}4\ \mathrm{DL} + 2{\cdot}2\ \mathrm{LL} > F_y$$

Current American practice for cable stays is to use an allowable stress of $0{\cdot}45\ F_u$, with discussion on increasing the allowable stress for bar cables.

67. Concerning the man-made island protection, no model tests were performed. Island geometry was based on theoretical calculations and experience with other structures. Ship collision design was based on a 15000 DWT vessel traveling at 4 knots. It was recommended that piloting be required for large vessels.

68. With regard to *Mr Hart's* comments, it may be said that a very high percentage of the larger vessels enter and leave the main ports through the north channel and thus never pass under the bridge. The channel depth at the bridge is 8–9 m and the vessel size is limited. In addition, compulsory piloting was recommended for vessels of over 5000 DWT. A risk assessment resulted in a design vessel of 15000 DWT travelling at 4 knots. In comparison with seismic forces, collision forces were approximately five times greater. The islands were sized to stop the vessel before impact with the pier. Certainly, some of the force from the impacting vessel will transmit through the fill to the foundation. However, this force will be small in comparison with the large resistance of the foundation.

69. Short shear connectors were not considered because the rivets had to pass through the pipe wall to increase bond resistance on both the inner and outer pipe surface.

70. As mentioned in Paper 9491, both static and dynamic tests were made on the live load bond anchorage. These tests proved the satisfactory performance of the anchorage. No attempt was made to identify any local cracking around the pipes for crack control because this was not a concern.

71. To my knowledge, a limiting wind speed is not foreseen for traffic using the structure.

72. To answer *Mr Simpson's* first question, I was at the bridge site for approximately two months during which one half of the structure was completed and supported at the end pier. Winds through most of those days were fairly steady, and I did not notice any movement at that time. I was on the Dame Point Bridge when it was at the side span closure stage, and one could visibly notice the relative movement between the end pier and the bridge cantilever. The frequency was in close agreement with the calculated frequency, and the amplitude was small.

73. Nothing has been done to monitor the movements from wind to this point. The main problem that arose during the construction was a cable vibration problem. This had occurred before the grouting of the cables; since then vibration has not been noticed.

74. The first pair of cables—cable 1 on the main span side—is side by side; on the end span side it is one after the other longitudinally for that segment. From there on, there are two cables per end span segment, anchored in a horizontal plane at the tower, and at different longitudinal positions in the edge girder. On the main span side, there is a single cable anchored in the centre of the edge girder. The cable sizes did vary. Cable types A, B, C and D were used in the main span, and A (cable 1), E and F in the end span.

75. The water depth at the approaches was approximately 8 m. It is certainly possible for larger ships to reach these piers, but the probability that they will is extremely small owing to navigation controls and recommended compulsory piloting. For protection against smaller vessels, the piers were designed for collision forces of 1000 kN (bow collision) and 500 kN (beam collision).

76. In reply to *Professor Hanna*, my review of Professor Chin's plot indicates that it shows good prediction in some soils and poor predictions in others. Current practice is to evaluate ultimate load prediction using dynamic analysers. The use of piled embankments is over 60 years old. To the best of my knowledge, the Penang embankments are performing satisfactorily. The use of raking piles does increase lateral resistance. I believe that the benefits derived from their use outweights the smaller increase in driving difficulty.

Mr Stanley

Professor Hanna refers to the use of raking piles. At Sungai Perak Bridge, the piles under the main pile caps were indeed raking but the inclinations of 1 in 40 and 1 in 25 were primarily to ensure that the group had a positive spread. Having carried out the horizontal load tests on the vertical piles under the protective ring beams, we established that the horizontal resistance was indeed significant. In theory, therefore, I would agree that the piles under the main piers could well have been vertical. From the practical point of view, it was good to have a positive outward spread that resulted in the loads being applied to a larger base area.

78. In reply to *Mr Fletcher* about the ability of Malaysian contractors to build balanced cantilever bridges, it is worth noting that Malaysia has quite a history of balanced cantilever construction. In 1974, two bridges were constructed at Jerantut and Kuala Lepar across the Pahang River, with spans of 116 m, and two other bridges were built on what is known as the East West Highway. Another bridge was completed over the Perak River in 1985, but the Sungai Perak Bridge, with a span of 160 m, is the longest so far. The superstructure was essentially constructed by Malaysians with very little help. The main prestressing contractor VSL Malaysia fabricated and supplied the travelling formwork and provided all the jacks, with only the back up of their Swiss colleagues. Therefore, the answer is yes, the work can be done by Malaysian contractors; however, the large diameter piling needed special equipment. If it had not been for the availability of the large floating pile driving barge, the handling and driving of the piles and the pile cap construction methods using the heavy precast concrete units would not have been possible.

79. I would also agree with *Mr Fletcher* about the importance of the supervision of the site investigation, but would reiterate what *Mr Williams* says about the hazards and difficulty inherent in carrying out a sophisticated investigation in a deep, fast flowing tidal river (§ 23).

80. *Mr Meadows* refers to the use of circular steel props. At Sungai Perak Bridge, the contractor chose to provide the fixity needed at the pier during the cantilevering stages by using offcuts from the steel piles. The 1500 mm diameter of the steel tubes resulted in little difficulty when fixing the bracing members and were several times larger than they needed to be. The steel props were filled with concrete at their tops to ensure that loads were spread evenly into the steel walls.

81. *Mr Andrews* asks about the reference datum for the pile testing and the use of plastic tubes inside the prestressing ducts. One pile

was indeed jacked against four others, and another two independent but adjacent piles were used to support the equipment which recorded the settlement. All the piles were monitored using standard levelling methods to confirm the settlement datum.

82. The 6 m long plastic tubes were used to stiffen the duct during construction, making it easier to fix in a straight line with a minimum of wobble. During concreting, the plastic tube helped to prevent grout from leaking into the duct and causing blockages. The tubes were removed and re-used for each cantilever unit. Plastic tubes would not have been suitable as permanent stressing ducts because of the problems of fixing them within the reinforcement cage and perhaps because there could be bond problems with the concrete and subsequent grout. In § 21, *Mr Andrews* also refers to future maintenance; this was one of the major considerations in choosing concrete as opposed to steel.

83. In reply to *Mr Sadler*, it was clear before detailed design that steel could have had advantages in producing a lighter superstructure and a reduced piling requirement. The piling and foundation problems at Sungai Perak Bridge would not have been solved by the use of a steel deck, and the delay would have been reduced only slightly. The advantages of in-situ concrete—using local materials and requiring less frequent maintenance were both a practical and an economic argument in the circumstances.