FORTH ROAD BRIDGE

by

J. K. Anderson, M.A., M.I.C.E.,

J. A. K. Hamilton, G.M., B.Sc., D.Sc., F.R.S.E., M.I.C.E.,

W. Henderson, M.B.E., B.Sc., M.I.C.E.,

J. S. McNeil, B.Sc., M.I.C.E.,

Sir Gilbert Roberts, F.R.S., B.Sc., M.I.C.E.

and

H. Shirley Smith, C.B.E., B.Sc., F.C.G.I., F.I.C., M.I.C.E.

ACKNOWLEDGEMENTS

DESIGN AND SUPERVISION OF CONSTRUCTION

The Engineers for the Forth Road Bridge project were Messrs Mott, Hay and Anderson; Freeman, Fox and Partners were associated with them for the main bridge and approach viaducts. The Architect was the late Sir Giles Gilbert Scott.

Sir David Anderson was responsible for all work in connexion with the project until his death in 1953, after which the late Mr Vernon Bartlett, Sir Gilbert Roberts and Mr J. K. Anderson were the Partners chiefly concerned. The following individuals took a leading part in the various sections of the work.

Design

Main Bridge: Sir Gilbert Roberts, F.R.S., B.Sc., M.I.C.E.

W. C. Brown, Ph.D.; M. F. Parsons, B.Sc., A.M.I.C.E.; D. W. Smith, B.Eng., A.M.I.C.E.; A. E. Temple, B.Sc.

Approaches: J. K. Anderson, M.A., M.I.C.E.

C. D. Brown, B.Sc., M.I.C.E.; D. H. Fletcher; R. C. Marc, M.Sc.(Eng.), M.I.C.E.; I. W. Munro, B.Sc., A.M.I.C.E.; P. Sparrow, B.Sc.(Eng.), A.M.I.C.E.

Site Supervision

Resident Engineer: J. A. K. Hamilton, G.M., B.Sc., D.Sc., F.R.S.E., M.I.C.E.

Deputy Resident Engineers: S. M. Kirk, M.I.C.E.; B. G. Smith, B.Sc.(Eng.), M.I.C.E.; D. W. A. Bower, B.Sc., A.M.I.C.E. (Contract No. 2).

Section Engineers: A. C. Macdonald, A.M.I.C.E.; H. J. Sim, B.Sc., A.M.I.C.E.; T. M. Young, M.I.C.E., A.M.I.Mun.E.

* *Proc. Instn civ. Engrs*, 1955, **32** (Nov.) 321-512.

Assistant Engineers: G. Baillie, A.M.I.C.E.; D. S. Blackwood, B.Sc., A.M.I.C.E.; G. Bowie (*Student*); R. D. Broatch, B.Sc., A.M.I.C.E.; W. H. Brown, B.Sc.; I. F. C. S. Clayre, M.A., A.M.I.C.E.; G. Crighton, B.Sc.; P. J. F. Crossley, B.A., A.M.I.C.E.; J. Fraser, B.Sc. (dec'd); D. J. Gemmell, B.Sc., A.M.I.C.E.; J. Halliday; P. N. Holmes, B.Sc., A.M.I.C.E.; T. Johnstone, A.M.I.C.E.; H. Kneipp, B.E.; R. Page; R. G. Ramsay, B.Sc., A.M.I.C.E.; A. J. E. Rose, B.Sc., A.M.I.C.E.; F. C. Sandeman, B.Sc.; A. G. Simpson, A.M.I.C.E.; A. Szabo, B.Sc.; M. J. Walker, B.Sc.; H. L. T. Woodhouse, A.M.I.C.E.

Accountant: N. Blenkinsop, C.A.

CONSTRUCTION

The main Contractors and the members of their staffs principally concerned were as follows.

Substructure: John Howard & Co. Ltd

Chief Engineer: G. I. B. Gowing, B.Sc., A.C.G.I., D.I.C.

Agent: J. M. Donald, B.Sc.

Sub-agents: H. J. Jones, B.Sc., A.M.I.C.E.; B. J. Soanes.

Assistant Engineers: W. Brown, B.Sc., A.M.I.C.E.; D. Collier, A.M.I.C.E.; G. J. L. Coltart, M.S., B.A., A.M.I.C.E.; D. Findlay; G. Fitchie; A. Howard, A.M.I.C.E.; J. H. Hyatt, B.A., A.M.I.C.E.; J. Irvine, B.Sc., A.M.I.C.E.; A. Johnson; J. May, B.Sc., A.M.I.C.E.; J. Perkins; K. Sim.

Superstructure: A. C. D. Bridge Co.

Joint General Managers: J. Y. Danks, B.Sc., M.I.C.E.; D. C. C. Dixon, B.A., M.I.C.E.; W. J. Harper, B.C.E. (part time); K. E. Hyatt, B.Sc., M.I.C.E. (part time).

Office Staff: G. W. Morley, M.A., M.I.C.E. (Chief Engineer); V. Carnegie, M.I.C.E.; H. A. C. Reid, A.M.I.C.E.; T. J. Upstone, M.Sc., M.I.C.E.

Site Staff

Agent: H. Shirley-Smith, C.B.E., B.Sc., F.C.G.I., F.I.C., M.I.C.E.

Deputy Agent: J. Kinsella, M.Sc., B.E., A.M.I.C.E.

Section Engineers: D. C. Ross, B.Sc. (dec'd); H. S. G. Knox, B.Sc., M.I.C.E.; J. H. Hyatt, B.A., A.M.I.C.E.; C. H. Fairweather, A.M.I.C.E.; J. C. Kaye, B.Sc., A.M.I.C.E.

Assistant Engineers: R. J. Webb, B.Sc., A.M.I.C.E.; C. D. Strachan, B.Sc., A.M.I.C.E.; B. S. Egan, B.Sc., A.M.I.C.E.; H. A. G. Smith, A.M.I.M.E.; D. M. Dyson, B.Sc. (*Student*); J. L. W. Shirlaw, B.Sc. (*Student*).

Office Managers (part time): R. Mallinson; R. A. Richards.

Labour Officer: W. McPherson.

Foremen: J. Laverty, *B.E.M.* (steel); A. Fairless, *B.E.M.* (plant);
G. Marshall (yard); J. Aitken (welding).

Superintendent at Drem: R. K. Wilson.

Consultants to A. C. D. Bridge Co.: John A. Robelings Sons Corpn.

Chief Engineer: Blair Birdsall, *F.A.S.C.E.*

Site Engineer: H. W. Hills.

Approach Viaducts: Reed and Mallik Ltd

Director: R. F. M. Grant, *M.A.*

Agent: J. Kerr, *M.I.H.E.*

Sub-agents: A. Wearmouth; A. Hamilton, *M.A.*, *A.M.I.C.E.*

Messrs Head, Wrightson & Co. Ltd supplied and erected the steelwork.

Agent: O. G. Davies, *A.M.I.C.E.*

Messrs Sandberg were responsible for designing all concrete mixes and for supervising and testing all concrete work.

Northern Approach Roads: Whatlings Ltd

Directors: J. Robertson, *B.Sc.*, *M.I.C.E.*; J. Ross McGibbon, *B.Sc.*, *M.I.C.E.*;
D. C. Lindsay, *B.Sc.*, *M.I.C.E.*

Agent: R. Marshall.

Southern Approach Roads: A. M. Carmichael Ltd

Director: D. A. Carmichael.

Agent: J. Kirkpatrick, *B.Sc.*, *M.I.C.E.*

Particular acknowledgement is due to the work of Messrs E. E. H. Bate, *C.B.E.*, *M.C.*, *B.Sc.(Eng.)*, *M.I.C.E.*; J. Bickley, *B.Sc.*, *M.I.C.E.*; D. Ambrose; A. Emery, *A.M.I.H.E.*; and A. C. Paterson, *B.Sc.*, *M.I.C.E.*, *M.I.Mech.E.*, of F. R. Bullen and Partners.

Thanks are also due for the help and co-operation of the Chief Road Engineers, Scottish Development Department, Messrs J. Emlyn Jones, *O.B.E.*, *B.Sc.*, *M.I.C.E.*, and J. S. McNeil, *B.Sc.*, *M.I.C.E.*, and their representatives, Mr W. Henderson, *M.B.E.*, *B.Sc.*, *M.I.C.E.*, and Mr A. Sutherland, *A.M.I.C.E.*; to the City Engineers of Edinburgh, Messrs W. P. Haldane, *M.B.E.*, *B.Sc.*, *M.I.C.E.*, and F. R. Dinnis, *M.I.C.E.*; to the County Surveyors of Fife and West Lothian, Messrs T. McCallum, *M.I.Mun.E.*, and J. Stuart; and to the Burgh Engineer of Dunfermline, Mr A. Sinclair, *M.I.C.E.*

Mr A. Simpson gave great assistance in the preparation of two sections of the Paper and also supplied some of the photographs.

Discussion

Sir Alfred Pugsley (Professor of Civil Engineering, University of Bristol) said that close on 100 years previously the Institution had met to consider a Paper on the then newly completed Clifton Suspension Bridge. At that time this had not only the longest span in Great Britain but the longest in the world. Sir John Fowler, later responsible with Benjamin Baker for the Forth Railway Bridge, had then been President. The discussion had tended to concentrate upon the function of the stiffening girders in such bridges, and this had led to a subsidiary Paper by the then Astronomer Royal, Sir George Airey.

2. At that time, as a result of the work started by Telford and Brunel, Great Britain had been in the forefront of suspension bridge design and construction. After a long period of inaction, it was very pleasant to find, due largely to the Authors of the present Paper, that Britain was back again in the forefront.

3. He had first come into contact with the problems of the Forth Road Bridge through Mr Crosthwaite, who had done so much for the theory of suspension bridges, and Dr Frazer of the National Physical Laboratory who, with Mr Scruton, had conducted the early wind tunnel experiments. Dr Frazer, who had since died, had ably led British aeroelastic research work for some 20 years.

4. Sir Alfred drew attention to the co-operative spirit which had been shown throughout by the teams engaged on the bridge. In what was now the classic British manner for such great projects, they had successfully drawn in the help of many relevant experts, both academic and practical.

5. The bridge design contained a number of technical innovations, not least in the towers. By treating these not just as long columns subjected to very heavy end loads, but as vertical cantilevers having end loads with the top deflexions governed by the cables, the designers had cut out the irrelevant empiricisms in conventional column formulae and had saved a great deal of metal. This tempted one to wonder whether a little more could have been saved. He noticed, for example, that the diaphragms in the towers were at intervals of 14 ft 6 in., and he wondered whether the Authors felt that these could have been reduced in number or weight.

6. Bridges with such long spans had low natural frequencies comparable to the pitching and rolling frequencies of large ships, and were subjected to large deflexions. These characteristics had led engineers to consider the comfort of the bridge traffic; in America this had resulted in some interesting work which was now being supplemented at Bristol under the aegis of CERA. In Table 2.17 a peak change of slope at the towers corresponding to a gradient of 1 in 45 was quoted. The bulk of this was due to live load. He wondered what proportion of the span was covered to produce this change.

7. Mr Shirley-Smith had referred to some dynamic effects of wind on the bridge during erection. Great strength had had to be provided in the Forth Railway Bridge against steady winds. The new bridge provided for a steady wind of 110 mile/h involving the whole or any part of the bridge, considered aerostatically. To what extent, he wondered, should allowance be made directly in strength calculations for the dynamic effects of eddying winds.

Mr J. F. Pain (Consultant, Dorman, Long & Co.) said that the Paper under discussion formed a most worthy record of a great bridge-building achievement.

9. The economic possibilities of really long suspension bridges depended on the use of steel in its most efficient form as high-tensile wires, for their cables. The classical method of producing parallel wire cables evolved by Roebling over 100 years previously had undergone continuous evolution in all the major suspension bridges of America, but it remained unaltered in principle. This evolution had naturally been based upon American conditions of plant and labour costs which were not necessarily applicable in this country. The cables of the Forth Bridge had been produced under the advice of American engineers and by plant and procedures which they had recommended. He would be interested to know if the engineers directly responsible for the cable construction would recommend any major change if they had to do the job over again.

10. In spite of the wide American experience of parallel wire cables, there still seemed to be a lack of detailed knowledge of some of their properties. The Authors had drawn attention to the question of secondary bending stresses and the possibility of an increase in allowable working stresses if the secondary stresses could be more closely evaluated. In a structure where saving in self-weight was of paramount importance, this was clearly a subject for further investigation. As at present constructed, the cables were of uniform section throughout, although their stress was proportional to their inclination to the horizontal. If they could be more closely proportioned to the actual stress, a further appreciable saving in weight might be achieved.

11. The diameter of 0.196 in. of the individual wires was based on long-standing American practice, but if wires of larger diameter could be produced with the same physical properties at something like comparable cost, there was scope for a material reduction in spinning time. The cable was the only part of the superstructure built *de novo* on the site, and any change of method which reduced the spinning time was of great economic importance. The Americans were already investigating the possibility of pre-forming wire strands. The late Sir Ralph Freeman had been responsible for a 1100 ft suspension bridge in Rhodesia where 8 in. dia. parallel wire cables had been spun in ribbons of 37 wires at a time, largely by unskilled native labour. There seemed to be plenty of scope for further development in cable design and construction.

12. One point about the building of the superstructure which did not clearly emerge from the Paper was the very large amount of initial investigation and design involved in the preparation of the scheme of erection, and the plant and temporary work necessary to carry it out, particularly in a structure subject to large distortions during construction. In this work, which had been the responsibility of the Contractor's office, material help had been received from the Engineers.

Mr A. Goldstein (Messrs R. Travers Morgan & Partners) associated himself with all that had been said by the President and by previous speakers about the great achievements which were being discussed. It had occurred to him that some of the more general information in the Paper might have application to other work where both the forces of nature and the forces of human nature

had to be contended with, but he would address himself specifically to the enlightened target contract arrangements which had been used in the building of the bridge (§ 1.31 *et seq.*).

14. Special measures were clearly required to deal with the methodology for this work. It could apply to complex jobs, other than bridges, and it would be useful to hear a great deal and in detail about how this contract operated, the techniques used and the results.

15. He hoped the Authors would answer the following questions. First, what was the original target sum and the final adjusted target sum, the original and final profit fee and the final cost and profit adjustment? Next, could they help others concerned with the development of special forms of contract by giving the definition of overheads and of profits and explaining how the problem of plant charges, which were based on depreciation etc., were worked out and related to the cost of plant? This was a difficult problem; labour and materials were not quite so difficult.

16. The actual method of modifying the profit, etc. seemed 'unbalanced'. It was stated in § 1.36 that the Contractor was insured against absolute loss, and this may certainly have been proper on a job of this kind. But it seemed that in fact he was assured of a minimum profit of 70% of the original profit (negotiated) since there was a *maximum* abatement of profit of 30%. This meant that after a 15% increase in cost (and the 30% abatement of profit fee) *all* further costs were reimbursed and the 70% fee was not further reduced. It then seemed to become a cost-plus contract with correspondingly less favourable effects if indeed the costs did exceed 1.15 of the target sum (probably they did not, but the risk was there). This may have been in order, but if so, could not a similar arrangement have been made for the case in which the cost was below the target sum (e.g. 2% less cost leading to 1% profit increase, or conversely, a 50%-50% split on both increases *and* decreases)? This would appear rather more equitable. It would help considerably if the Authors could give some of the reasoning and explain the way in which it had operated.

17. There might be other ways in future work of dealing with target sums, possibly on a tender basis, where target sums and fees were the subject of tender. The Authors might explain their experience of the running of their procedure in a difficult and major job.

Mr K. E. Hyatt (Associated Bridge Builders Ltd) said that, having been fairly closely connected with the project in various capacities, he thought that it might be of interest if he took up a point which had been mentioned about the erection scheme for the suspended structure. It was rightly stated that a floating-out scheme had been considered in the early stages and had been turned down for various reasons, but, in the light of what had happened, he asked the Authors whether they might not have reconsidered this aspect. He felt that all had been unduly concerned about the conditions on the site and the difficulties of a large floating operation.

19. The proposal had gone so far that a site had been earmarked on the reclamation ground near the Rosyth dockyard for the construction of temporary slipways, erection bays and a general depot. Conversations had taken place with the Admiralty and tender documents had been printed on the basis of the use of Rosyth area.

20. From experience both on the Forth Bridge and on the Severn Bridge, he believed that six or seven months' construction time might have been saved if a floating-out scheme had been adopted. It would take some time to prove this assertion by figures and at present it was merely a matter of opinion, but there had been an assumed difference in cost between the two schemes which he thought would have been overtaken if the work had been carried out on this basis in spite of the very creditable erection speeds which had been reached on the site by the method employed.

21. There would have been a saving in overheads both to the Contractor and to the employing authority, and indeed to all concerned. There would have been considerable savings in labour in carrying out assembly in good conditions on a stage near ground level and not in the air. There would have been other considerations which would have had to be taken into account, and they could have saved on the secondary erection because they might have been able to put up heavier pieces. It was agreed that there was a distinct doubt because of the aerodynamic problems, and the possibility that part of the roadway deck might have had to be omitted had been noted. There had also been limitations on the loads on cable points at 4% of H in the main cable, but if erection had been begun in the middle, this would have soon been overcome and the H could have been built up effectively. They could then have proceeded to erect heavier pieces. It was a fact that although it appeared that weather was an important aspect in a floating operation, it was not such a consistently prominent factor as in piecemeal erection because a large assembly could be erected quickly and, if the weather deteriorated, one could wait without being too greatly concerned.

22. There had been other difficulties, including the presence of a rather prominent rock in the north side span which would in all probability have had to be removed. It was possible to argue that the method actually used was quite correct and could not have been improved, but he would like to hear the Authors' views on this subject.

23. Reference had been made to safety nets. It should be appreciated that a great deal of experimental work had been carried out by the company. There had been no previous standard for safety nets, as they were not in general use in British engineering although they had been used to some extent in the United States. It had been found necessary to conduct a great many experiments on different types of netting—nylon, terylene, coated, uncoated, knotted, unknotted—and tribute was due to the considerable work of the site staff in their efforts to get the best results. They had started with 600 lb braided cord and had finished with 1250 lb twisted cord. The factory inspectors were taking due notice of the results.

24. It was stated in § 4.99 that the porosity in the welds on the battled deck had been caused by the zinc spray. This might be true, but the permanent backing strip had probably also had an effect. Mr Hyatt thought that, in site welding, it must be admitted that double runs rather than single runs were necessary in cases of this kind. The use of the backing strip had many disadvantages but it was practical. It had to be attached during fabrication, before erecting the section, and it was there to place the next piece on. The alternative of sealing the deck joints from below was usually very costly.

25. Reference should also be made to the crash barriers and grillages. Perhaps if this type of work were carried out on a future occasion, it might be

felt that bolts should be employed instead of welding to fasten them to the bridge. Mr Hyatt thought that this method would be more advantageous if there was any damage to repair. The question of stagings, mentioned in the Paper, was very pertinent in this connexion. A great deal of under-deck staging work had been carried out for many operations and this had proved to be very expensive. This comment was made perhaps from the Contractor's point of view, but it was germane to structures as a whole, since all desired economy and all should seek to achieve it.

Mr C. D. Brown (Messrs Mott, Hay & Anderson) said that as a participant in some of these matters he wished to elaborate on one or two points of the Paper.

27. The first point to which he would refer was in § 1.22 relating to the wind gauge experiments which had been undertaken before detailed design work commenced. Three gauges had been operated, spread out over 3600 ft on the Railway Bridge, to measure the speed and inclination of the wind sheet. He thought that the figures given in the Paper referred to the wind sheet, because he recollected that for individual gauges, inclined winds of double the velocity stated might be expected.

28. Mr Pain had commented on §§ 2.9 and 2.10 in which the case for parallel wire construction was set out. Additional information which he did not think was contained in the Paper was that the portion of the cost of the special equipment for cable spinning and main tower construction at the Forth Bridge which was eventually charged to the Severn Bridge project, amounted to about £300 000. A sum of this magnitude underlined the benefits of building suspension bridges two at a time!

29. He wondered, however, if this might have inhibited the use of other materials and methods of construction for these items. He doubted whether it would make any difference in relation to the Forth Road Bridge project, but with reference to the method of cable formation, more recent developments in the prestressing and prefabrication of bridge strands led him to the view that, for the main cables, whether or not to spin must now be a very close decision.

30. In addition to the information on the tower mentioned in § 2.107, he wished to give some additional data about the statement on the 1 ft allowance made to cover the lack of straightness and deviation from the vertical in the dead load condition. This allowance had been deliberately made rather large, and it might be of interest to give the measured end result on completion of the bridge. The worst error was about $5\frac{3}{16}$ in. This error corresponded fairly closely to a predicted value based on the following design assumptions:

- (a) *Dead load*: 2% error in concrete density, $\frac{1}{8}$ in. error in concrete thickness; 1% error in all other dead load in any one span;
- (b) *Temperature*: an error of $2\frac{1}{2}^{\circ}\text{F}$ in the measured temperature of the back stay and side span cable strand when the first strand was finally adjusted in the saddle;
- (c) *Elasticity*: a 2% error in the assumed elasticity of the cable material;
- (d) *Setting out*: a $\frac{1}{2}$ in. error in setting out the verticality of the tower when adjusting the first strand of cable based on an error of 10 s of angle at a distance of 650 ft.

31. He drew attention to the points set out in § 4.81 and the arguments advanced by Mr Hyatt for erection in large sections. The Tamar Bridge and the Severn Bridge had both been erected in large sections, and Mr Hyatt had expounded the technical arguments and voiced some queries which also existed in Mr Brown's mind. Under all the circumstances now known would not large section erection have been advantageous? The inference of § 4.81 might be that the local working sites were too small to permit this. If so he would be interested to know the minimum size of working site which the Authors considered would be suitable.

32. He referred to Appendix 5.2 which gave details of some of the bridges to the approach roads of the project. The tables showed the cheapness of flat-slab reinforced concrete construction, the Masterton Viaduct being a particularly good example of this. However, he pointed out how misleading it could be to compare the cost/sq. ft of small approach bridges. For example, both the Burnshot Bridge and the Echline Bridges were roughly similar in facilities and total cost, but the unit costs falsely suggested that the Burnshot Bridge was much cheaper because the embankment side spans of that bridge were included in the deck area.

Mr R. B. Wood (A.C.D. Bridge Co. Ltd) said that, as the last member of the site staff still on the roll of the A.C.D. Bridge Company, his thoughts turned naturally to the problems of construction and particularly to the suspended structure. Mr Pain and Mr Hyatt had mentioned some of the difficulties in the construction and he wished to refer to one of the main problems, the provision of sufficient portable and/or movable stagings and platforms required for the wide variety of operations involved.

34. Temporary stagings had been required for erection, bolting-up, welding, painting, and, in the side spans, shuttering and concrete work. They had also been required for work on electrical and various other services. The problem became more acute as the bridge deck panels and crash barriers were built in during the second pass erection, and the variety of jobs under the deck spread out from one end of the stiffening trusses to the other—a distance of $1\frac{1}{2}$ miles.

35. There was no doubt that the provision of permanent travelling carriages would have been of great assistance in the completion of the suspended structure, particularly during the six months before and the six months following the opening of the bridge to traffic. Would the Authors not agree that travelling carriages, and a suitable system of rails on which to run them, should have been included as part of the bridge structure itself, so that they could have been erected and used during the later stages of the construction of the suspended structure? The Contractors would have found a great deal of use for them, and their erection as part of the suspended structure would have been a much cheaper business than would be the case now.

36. He drew attention to the installation at Drem Airfield, East Lothian, for the protective treatment and storage of bridge steelwork. It might not be appreciated that this site lay in a mainly agricultural district where, although there was some unemployment, there were no trained men, and in fact very few skilled men of any type, available for the grit-blasting and zinc-spraying processes. To carry out this work a considerable amount of very heavy and quite new metallization plant had been installed. However, within a reason-

ably short time it had been possible to train virtually unskilled labour so that a remarkably high standard of grit blasting, zinc spraying and painting had been achieved. He knew of no other plant of this size and capacity which had been used in connexion with an outside site.

37. The airfield had also been used as a collecting and storage area for the whole of the bridge steelwork. Again, it might not be appreciated that it lay roughly 30 miles from the south end of the bridge, a long distance considering that Edinburgh lay between the two points. Moreover, it was 75 miles away from the north end of the bridge. With hardly any storage space on either bank, the delivery of 10 000 tons of steelwork to each end of the bridge, to reach the right place in the bridge structure at the right time, had been a formidable task. It must be remembered that a great number of the loads of steelwork had been abnormal ones, requiring police escorts. It said much for the personnel of the storage area and for those in charge of the transport arrangements that this operation had been carried through with hardly a hitch.

38. As Mr Brown had said, it might perhaps have been worth considering whether it would have been better to open a storage site rather nearer to the bridge. Inevitably, the difficulty of the long transport run to the north bank via Kincardine Bridge would always have been there, but at least the initial 30 miles would have been cut out.

Mr H. S. G. Knox (Senior Engineer, Associated Bridge Builders Ltd) said that as one connected with the planning and execution of cable construction at the Forth Bridge site he would like to draw further attention to some of the features which were new and to the success which had attended their adoption.

40. This had been the first time that a cable arranged as a vertical hexagon had been spun in the saddle. Up to the time of the construction of the George Washington Bridge, all cables had been arranged as horizontal hexagons, but with the 36 in. dia. cable there considerable difficulties had been caused by excessive tilt and slack of the top strands in sun, and slow normalization at night for adjusting.

41. After this experience, the same contractors, the John A. Roebling's Sons Corporation, had proposed a vertical hexagon for the Golden Gate Bridge cables. This, as the Authors had mentioned in § 2.31, allowed the use of vertical spacers for aeration. But this cable, and subsequent vertical ones, had been spun out of the saddle, i.e. strands had been built up and seized out to one side, and then lifted into the saddles. Meanwhile, the saving in equipment made possible by spinning the strands in their final position in the saddles had been shown, but only on horizontal hexagon cables of up to 24 in. dia.

42. For the Forth Bridge, the Contractor had wished to combine the advantages of both processes and this had been achieved by Roebling's suggestion, accepted by the designers, of using rectangular section strands through the saddles, with a stepped bottom to the groove instead of the normal circular trough, and permanent vertical spacers between the rows of strands. This had not only made spinning in the saddle possible but had also been found to produce better wire placement in the tower saddles. The voids ratio after spinning had been reduced to 16% against the previous 20–21%, with negligible slump during addition of the suspended structure. The desired shape to the top of the cable had also been easily achieved, whereas with the horizontal hexagon cables spun in the saddle there had been considerable

difficulty in forming the top strands correctly, followed by an unpredictable slump of perhaps an inch or two with, finally, a rather flat top. Permanent top spacers had often had to be machined at a very late stage in construction to suit the actual shape.

43. In the spans, the original purpose of the vertical hexagon had proved vital in the higher latitudes of Scotland. In the summer, with daylight almost to midnight, the strands had not reached sufficiently uniform temperature for adjusting until about 1.30 a.m., and had started to change again with sunrise at 5.00 a.m. This had given only just enough time for adjusting; with a horizontal hexagon, proper adjustment might have been impossible.

44. The cable formers which had been provided primarily for aeration, had also proved indispensable in controlling the strands of the partly-spun cables in the ever-present high winds at Forth. The Authors had described, in §§ 4.58–4.60, what had happened when severe storms hit the cables with four or five strands spun, before the cable formers could be used. If this had happened with a large number of unseparated strands the damage would probably have been irreparable, but once the cable formers had been placed in position there had obviously been no risk of this. Apart from security in storms, the cable formers had also allowed efficient adjusting and safe tying-in of wires to be maintained in the almost constant 20–35 mile/h winds, to an extent not otherwise possible when spinning in the saddle.

45. He was therefore convinced that, while it had been shown in lower latitudes and much less windy climates that it was possible to spin medium-sized horizontal-hexagon cables in the saddle to an acceptable standard, experience at Forth and subsequently had shown that the vertical hexagon arrangement was essential for suspension bridges in northern Europe.

46. He congratulated the designers on one point in the tower design to which he thought insufficient attention had been drawn, and for which they had not taken sufficient credit. This was the use of an embedded steel base section in the piers of the towers. He knew of the months of very laborious work which went into lapping down the concrete for the conventional American base-plate. The embedded base section with its screw jacks could be set and levelled to within 0.01 in. in one day, then held securely and surrounded by the concrete of the pier top. The result was a good machined surface on which to erect the towers. This had been a significant advance in suspension bridge design.

47. Mr Knox wished, however, to disagree with the statement in § 2.42 that the double suspender socket was entirely satisfactory and an improvement that might be used in future. While there was probably no reasonable alternative at Forth, as the change to four-part suspenders was made too late to alter the details of the stiffening trusses, and while satisfactory manufacture and erection of the suspenders was achieved, the double sockets were the cause of considerable difficulties and additional expense.

48. In manufacture, the specified length accuracy was of course no greater problem than normal, as the ropes were pre-stretched and marked singly for cutting. The difficulties arose in socketting and reeling; the latter was specified to be 'on the bight', a commonly used procedure avoiding the need for any work from the river in suspender erection. The double sockets, however, involved greatly increased handling on the pre-stretching range to get the two unequal lengths of rope alongside each other, socketed together, and

doubled for reeling; in the reeling itself great care had to be taken to avoid twists and to keep the two parts of each half in step with one another.

49. Erection of these suspenders, which weighed up to 4 tons, was not easy, particularly in regard to holding back the bights safely and equally while running them down the footbridge, and when landing them together on to the cable band grooves. While no major accident occurred during this work there were several minor ones, which could easily have had serious results, and the operation was clearly attended by more risks to men and material than any other part of the cable erection, requiring care and supervision to an extent difficult to ensure consistently. There was also little doubt that two single bights to each cable band could have been erected separately in less time than was required for the composite four-part suspender.

50. The third difficulty presented by the double socket arrangement was to ensure that the ropes were connected to the suspended structure in the same state of twist as when measured at the rope works. It could be shown that serious variations in total length, and, therefore, in distribution of load, might occur between the parts of the suspender or between neighbouring suspenders, if ropes differed more than about a quarter of a turn relative to the measured condition. To prevent this, it was the convention to paint stripes along the ropes during the pre-stretching and measuring operation. It was not practicable to land heavy suspenders during erection on the cable band with the stripes in any particular position. It was difficult to check the stripe throughout a long, hanging, and probably twisted, group of ropes and there might also be subsequent rotation in the grooves before final connexion. Where individual sockets were provided this would present no problem as the stripe could be surveyed immediately before connexion to the truss and the socket twisted accordingly. With the double socket arrangement any correction involved re-lifting the suspender from the cable band groove and rotating both halves simultaneously. There was also the risk of one socket being turned 'inside out'; if this was not realized the stripes could not be made straight throughout.

51. In conclusion Mr Knox strongly recommended that in any future bridges the truss connexion details should be arranged so that single sockets could be used. He was sure that this would produce an overall economy in material and erection costs.

Mr O. A. Kerensky (Messrs Freeman, Fox & Partners) said that the unavoidably abridged Papers could not even begin to describe the work that went into the £12 million project, the first suspension bridge of any size in the United Kingdom for nearly 100 years.

53. As was explained in the Paper, the 3300 ft span Forth Bridge superstructure, as built, was very nearly the same as that designed for the 3240 ft span Severn Bridge in 1945-55. The Severn Bridge, now completed, was a break-through into a new era which could probably not have taken place without the work that went into the first project.

54. In 1945 when the work on the Severn Bridge started there were no engineers in the United Kingdom who had experience or knowledge of designing and constructing large suspension bridges. A start therefore had to be made from scratch. As in every other research project, the first step was to survey the existing practice and knowledge. The late Sir David Anderson and the late Sir Ralph Freeman jointly and separately instigated enquiries and research.

55. Several American experts were contacted and most of the available literature was studied and summarized in numerous memoranda on different aspects of the problems in design and construction of suspension bridges, such as construction of deep foundations, design of anchorages, design and construction of towers, methods of erection of superstructure, design and methods of making cables, design of steel battledecks and composite concrete and steel decks, loadings, aerodynamic investigations, and so on.

56. With the knowledge of what had been done in the case of some 20 major bridges with central spans varying from 1200 ft to 4200 ft, the newly formed team, taking nothing for granted, began to examine each premise from first principles, and to search for better ways, if these were possible.

57. One of the first snags was, in fact, the loading. In 1945–46 the Ministry of Transport standard loading had certain values which, if applied to four lanes of the 6000 ft long bridge, would have required almost all of the heavy lorries in Scotland in order to produce the load. A similar situation existed in the United States, where practically every large suspension bridge was designed to a special specification produced for that bridge. For example, in the case of the George Washington Bridge, the stiffening trusses were designed for half the loading for which the cables and the towers were designed, on the ground that they were only secondary members.

58. Such a procedure was undesirable and Freeman and Roberts offered to the Ministry a loading which was then being developed by them for the Humber Bridge. This produced a lighter loading, but not as light as those which were eventually produced jointly with the Ministry of Transport and the B.S. 153 Committee, now known as the HA and HB highway loadings. The minimum load of 70 lb/sq. ft was reduced to 40 lb/sq. ft and there were still further reductions on the lanes in excess of two. The Forth Bridge was the first major bridge designed to the new loading, which became available in about 1953.

59. The simple rules then existing for estimating the effects of 90 mile/h winds on medium sized bridges were also quite inapplicable to a structure where wind effects were a major factor, and model static wind tests were carried out at the National Physical Laboratory. The present rules in B.S. 153, if applied to the Forth Bridge superstructure, would give total wind effects almost identical to those determined by the tests at the National Physical Laboratory, because the rules were based on that work. Wind effects were also related to speed, inclination and extent of the wind front and the duration of critical gusts. These data were not available for either Severn or Forth Bridge sites. Recording instruments were set up in 1947 at Severn and later at the Forth Bridge site. The measurements continued until 1954 on the Severn site and until 1957 on the Forth site. Special anemometers designed by Mr Hartley of the Meteorological Office, which recorded speed, direction and inclination of the wind, were installed in 1951 after three years of development and trials. The recording charts, which eventually filled a room, were carefully studied and analysed and the characteristics of winds at the two sites were then determined.

60. Much had already been said and written about the aerodynamic instability of suspension bridges. The Joint Engineers, with the help of the National Physical Laboratory^{D1} and the Ministry of Transport, picked up from

^{D1} The references appear on p. 187.

the point reached in the investigations on the Tacoma Bridge, and together with Professor Farquharson^{D2} advanced the theory and knowledge of almost six years of research. The Forth and the Tacoma Bridges were the first bridges designed to deal with the wind and not just to resist it. It was interesting to consider the elementary conceptions prevailing in this field only fifteen years earlier.

61. Having decided on the loadings, the next step was stress analysis. The early elastic theory which assumed equal pull in all hangers, i.e. an infinitely stiff hardening girder, was replaced in 1889 by the deflexion theory developed for a single span by Melan, and in 1904 extended to three spans by Moisseiff for the design of the Manhattan Bridge. Since then different mathematical treatments were applied to the theory by various workers in this field, such as Steinman,^{D3} Timoshenko,^{D4} and in 1939 by Southwell and Atkinson^{D5} whose theory Crosthwaite modified and perfected in 1947.^{D6} This method took into account the effect of hanger extensions and the variation of the moment of inertia of the stiffening trusses, and permitted the use of variable loadings of any length. But even at this stage all known methods dealt only with plane structures, i.e. a cable with a truss hanging from it. The torsional stiffness of the braced box was universally ignored in stress analysis, as were the deflexions due to shear. Crosthwaite^{D7} suggested a relatively simple correction to take account of these and the late Dr Frazer checked the torsional effects for the 3000 ft suspended structure by a rigorous mathematical analysis which took him nearly two years and was completed in 1951. This became of particular importance when reduced lane loadings were introduced in 1953.

62. The Forth Bridge was the first bridge in which all these factors were taken into account, resulting in a significant reduction in the weight of the stiffening girders, combined with additional safety. This was, perhaps, one of the biggest advances in the design of the classical suspended structure, and so far it had received only scant recognition.

63. There were two other major breakthroughs. One was the design of the all-welded battled deck and the other the design of the towers. The prototype battled deck, which was designed and tested in 1946-49, was the first of its kind. It was inspired by American practice and research, although the Germans actually were the first to build modern battled deck bridges.

64. The design of the towers was the result of many years of thinking and arguing. Usually towers accounted for more than half the weight of the structural steel in the bridge and maximum efficiency in their design was therefore vital. Approximate comparisons were made for steel and concrete towers, fixed base and pin-ended towers, HTS and mild steel towers, welded construction versus rivetted or bolted, and of course for many different arrangements of cells, but the biggest advance was made in the actual method of design, which was explained in the Paper. Mr Kerensky believed that the Forth Bridge towers were the first to be designed from first principles and not as struts subjected to transverse loads. This, the adoption of welding, and the omission of cover plates and of the usual multitude of internal webs and diaphragms, resulted in very considerable saving (some 30%) in weight.

65. Then there was a debate as to whether cables should be spun by the 100-year-old Roebling method or erected by the method used for the Otto Beit Bridge.^{D8} Both methods were theoretically satisfactory. Twisted wire

ropes were ruled out from the start as theoretically unsound and uneconomic at such spans. Eventually the choice was left to the Contractors. Perhaps the Authors would say something about this. It was interesting to read in the *Engineering News Record*^{D9} that spinning might be on the way out and that parallel wire strands could now be produced and transported to site in the required lengths.

66. Detailed comparative studies were also made of:

- (a) loaded versus unloaded back stays (complete designs);
- (b) concrete deck versus steel deck;
- (c) light weight concrete deck versus steel deck;
- (d) aluminium deck versus steel deck (at that time the aluminium industry hoped that the price of aluminium would fall, but even using a very optimistic price, which never materialized, aluminium did not compete);
- (e) economic panel lengths;
- (f) 0.196 in. dia. wire versus 0.144 in. dia. and even 0.25 in. dia. wire (at that time the industry's capacity to deliver the regular amounts was seriously in doubt);
- (g) economic sag combined with required rigidity;
- (h) erection by floating out versus cantilevering at deck level, including complex calculations of cable flexure and deflexions.

67. After various basic decisions had been taken the detailed design of all the parts was begun.

68. Suspension bridges were structures whose design could not be simply adjusted if any of the loadings were changed by even a small percentage, because the rigidity of the bridge and the stresses in the stiffening girders depended on the pull in the cable as well as on the configuration of the whole structure. The Severn-Forth Bridge had been analysed about ten times (a process which took months each time) and every element was designed at least four times and sometimes many more. For example the 78 ft span cross girders shown in Fig. 2.16 were the outcome of 20 different designs made in 1948, in which girders of different configuration were designed in B.S. 15, 548 and 968 steels, using welding and/or riveting. This, of course, was before the grip bolts were even heard of.

69. The results of model tests on welded chord members described in §§ 2.147-9 alerted the B.S. 153 Committee to the question of safety as stipulated in the then existing rules for design of welded compression members, and after further tests at Cambridge University an amendment to B.S. 153 was issued in 1964 as an emergency measure. Further work in this field was being supported by CERA, and it was hoped that even better rules would result.

70. Over a period of almost exactly 20 years many people were involved in the evolution of this great project, from the humble chairman who for seven years attended the wind recording instruments in all weathers and at all times of night and day, to the leaders of the team, Sir David Anderson, Sir Ralph Freeman and Sir Gilbert Roberts. Some had passed away and others had taken to other fields, while a few lucky ones were able to share in the final accomplishments. The achievement, however, would not have been possible without the leadership of the few and the devoted toil of the many.

FIG. D1: CHEPSTOW BRIDGE: TRUSS GIRDERS

Mr P. S. A. Berridge (Consultant: Donovan H. Lee & Partners) confined his remarks to the construction of the towers (§§ 2.12, 4.6 and 4.7) and the stiffening trusses.

72. Three-quarters of a century separated the two Forth Bridges. The contrast between the riveted construction of the railway bridge and the clean lines of the towers of the road bridge was very marked. The 1890 bridge was built up of tens of thousands of pieces of steel, mostly too heavy to move without lifting tackle, and each part was straightened, bent, punched, reamed, drilled, bolted, riveted, unbolted, and riveted in place as the structure grew. The President had drawn attention to the meagreness of the records in the *Proceedings* of the Institution, and had expressed the hope that the records about the new bridge would be complete. And, with this in mind, Mr Berridge hoped that some description of the shopwork and how the welded steelwork of the road bridge towers had been fabricated would be included in the records. The avoidance of distortion during fabrication and the machining of the ends of the huge boxes had obviously called for skill and clever planning in the shops. Work described as 'a more critical operation' (§ 2.12) and in which there was 'distortion resulting from the stress relieving action of the vibrations set up by the milling cutter' (§ 4.7) deserved a fuller description.

73. The machining of large pieces of welded girderwork *after* fabrication was always expensive. The very same problem had had to be tackled in fabricating the box sections of the top chords of the truss girders for the railway bridge at Chepstow in 1962. There, the six units in each truss were 12 ft wide, nearly 4 ft deep, and the largest, 60 ft long, weighed $34\frac{1}{2}$ tons (Fig. D1). Accuracy of fit between the parts of these 300 ft long horizontal struts was just as important as at the towers on the Forth. But the ends of the units had *not* had to be machined. The faying surfaces of the flanges of adjoining units had been brought into very close contact by temporarily tightening high strength bolts, at close pitch, before fabrication. The units had been assembled and

FIG. D2: CONNEXION OF WEB MEMBERS AND GUSSET PLATES: GERMAN PRACTICE AND SYSTEM AT CHEPSTOW

built in pairs on an erection bed, levelled to give the correct camber to the units, in the workshop. On completion of the first pair, the second unit had been moved along to where the first had been, and the third unit was built against it. Then, the third had occupied the place of the second, and so on; the process ensured that the fit of the following unit matched the one previously completed. Any significant distortion due to welding had been corrected before the units were unbolted, and the ultimate alignment in the trusses had been very good. By avoiding the machining of those internal flanges, the contractor had saved not only the costs of milling and turning the awkward units end-for-end to offer them up to his machine, but also the need for trial assembly as would have been imperative after post-fabrication machining.

74. He observed from Figs 2.16 and 4.19 that the web members of the stiffening trusses had been fitted in between the gusset plates which were already welded to and formed part of the box-section chords. The arrangement appeared to follow German practice (Fig. D2) in which there was a risk of loss of efficiency in the joints because the tension put into the bolts had first of all to bend the gussets inwards to close the erection clearances before clamping the parts together in the friction joint.

75. In the arrangement used at Chepstow, the web members did not even touch the chord gussets, much less have to be entered in between them. In fabrication, the width of the web members and the dimension over outsides of gussets had had to be kept constant; but that had been an easy matter, and the flange and web butt-straps had made a simple and efficient connexion.

76. Throughout the construction of the Forth and Severn Bridges, he had been privileged to visit the works and discuss problems with Consultant and Contractor. Such free exchange of ideas was not always so forthcoming. Perhaps it stemmed from a privilege accorded members of the Institution. In any case he wished to express his gratitude for the help given with such kindness and frankness.

Mr S. B. Kirk (Messrs Mott, Hay & Anderson) said that he wished to comment, first of all, on the construction of the anchorages. Though, admittedly, it might have raised certain difficulties and probably required the re-spacing of the central sections of the $4\frac{1}{2}$ in. diameter mild steel tunnel stressing ducts, he considered that there was definitely a case for forming access shafts in the concrete filling to the four anchorage tunnels during their construction, and possibly their retention after construction for inspection purposes.

78. Although he understood that it was not normal British practice, these should be left open for operational purposes as long as possible, and, being parallel to the centre line of the tunnels, they would themselves be at 30° to the horizontal. The formwork could be faced with steel corrugated sheeting in the same manner as was used throughout the construction of the upper section of the main piers. This would have provided a useful key for duck boarding in the invert of the access shafts themselves, and, eventually, a good key for their own concrete fill, and, coupled with the vertical access shafts originally sunk, would have helped to create a useful source of ventilation.

79. At the time the scheme was drawn up, it appeared that small tunnels of approximately $5\text{ ft} \times 2\text{ ft } 6\text{ in.}$ could have been provided, but the Contractor felt that the presence of the required formwork would have interfered with his use of the pneumatic placer in the concreting operations. This might be a valid criticism, but he would like to have the Authors' views as with such a big grouting programme the possibility of having control checks at all stages was invaluable.

80. He considered that the construction of the north and south piers was a wonderfully executed piece of work, and wished to add only two points. First, one of the side issues not touched upon in the condensed Paper was that while blasting the area required for the cofferdam the divers had to remove part of an old iron ship, which was never properly identified. Approximately 200 tons of debris were removed from the rock before the area could be certified as clear for placing the cofferdam steel sheet piles.

81. Secondly, another difficulty, mentioned in § 3.67, was that in the icy-cold black waters of the Forth, the working divers and the Resident Engineer's inspecting divers had tremendous difficulty in determining when all the very hard boulder clay was removed from the surface of the whinstone (dolerite) rock. Once that tenacious material was split open section by section by pneumatic spades, high pressure water jets and airlifts did the rest. The divers' attention to detail, as well as the good quality underwater concrete work, was revealed when dewatering took place.

82. In respect of the south and north piers, it might be of interest that tests carried out at Herriot Watt College, Edinburgh, revealed that the crushing stress for sandstone taken from the south pier foundation and whinstone taken from the north pier foundation was 790 and 1260 tons/sq. ft respectively.

83. With regard to the question of concrete mixes and control, he considered that it might be of value if in future a central laboratory could be established on the site well in advance of the various contractors coming on to it. The Paper described 12 contracts, and probably for 10 of them separate testing establishments were required. One central laboratory, or one laboratory on each side of the river, would have been of great value in handling the testing for all the contracts throughout the work.

84. With regard to materials on approach roads, the Main Contractor for these roads had elected to use Portland blast furnace cement to B.S. 146 instead of ordinary Portland cement, as allowed by the specification, both initially in his structural concrete and as a constituent in the cement stabilization of the spent oil shale sub-base. The contract specification for stabilized sub-base called for 7 day strengths of 250 lb/sq. in., but with blast furnace cement it was found by experiment that, because of its slow hardening properties, 200 lb/sq. in. at 7 days was sufficient to meet strength requirements, since at 28 days the strengths of ordinary Portland cement and blast furnace cement were about equal. The main advantage, apart from the economy afforded the Contractor, was that this cement was considered to have superior characteristics for resisting attack by the sulphates present in the spent oil shale, generally in the form of sulphur trioxide, which in turn should help prevent attack of the underside of the road base of cement-bound granular material.

85. There was co-operation with the Road Research Laboratory in a major experiment with edge markings of roads. The outcome could not be used on the Forth Bridge contract, but he hoped that it would help a great deal in the future.

Mr C. D. Crosthwaite (Messrs Freeman, Fox & Partners) said that, speaking as one who was associated initially under Sir Ralph Freeman with the design of the Severn Bridge as it then was, before it became that of the Forth, from the beginning in 1945 until 1952, he wished to add a little to what Mr Oleg Krensky had said.

87. First consideration had been aerodynamic stability. It was only four years after the Tacoma failure, and it was felt that until basic shapes were agreed there was not much point in designing the structure. Some elementary tests, which he had carried out in a home-made wind tunnel with section models, highlighted the radical instability of the H-section plate girder stiffening truss cum deck—the Tacoma section—and the very strong positive damping from some form of lattice girder associated with longitudinal air gaps.

88. When scientific aerodynamic testing was later carried out by the National Physical Laboratory, the first point to be established was whether it was acceptable to test section models supported on springing systems representing the combined stiffness of the cable and stiffening trusses, or whether there were effects which would only reveal themselves when a full model of the bridge was tested. If the latter were the case, suspension bridge designers would be faced with serious difficulties, as it would not be practicable in time or in money to try out many variations on a full model. Fortunately correlation was established between the behaviour of the full and the section models. At that time they had many discussions with Professor Farquharson, of the Engineering Experimental Station, Seattle. His advice was invaluable and they maintained their association with him throughout.

89. One of the points that came out very strongly from the tests was the markedly improved behaviour under torsional vibration of a section with top and bottom laterals. A suspension bridge with full top and bottom bracing between the chords had a quite different order of torsional stiffness from one with one plane of bracing only. These considerations led to a consideration of the very limited effects of eccentric loading on chord stresses to which Mr Kerensky had referred. Having established that torsional stiffness was greatly increased by full bracing, there was a natural desire to reduce the depth of the stiffening truss. No very cogent reasons for not dropping below 27 ft 6 in. in depth could be produced but it was felt that as the trusses were already substantially shallower than accepted practice they should not be further reduced.

90. Another matter to which much thought had been given in the early days was structural damping. There were obvious advantages in making the battled deck participate with the stiffening truss top chord, but it was felt that the structural damping from the expansion joints at each cross girder would be beneficial. They were reluctant to dispense with any available structural damping in an endeavour to make Severn-Forth outstandingly stable against all forms of wind-induced oscillation.

91. It was thought that wind-induced oscillations were unlikely unless 'wind-sheet' effects on a substantial scale could be anticipated. In other words, unless wind blew at a relatively constant speed, direction and inclination for more than a very short period of time along a substantial length of suspension bridge, there was little likelihood of more than random oscillations. It was to test out the extent of 'wind-sheet' that the Sharpness Railway Bridge tests were put in hand.

92. Finally, coming to structural analysis, he had realized when concerned with the Severn Bridge that the classical theory was unsatisfactory in its presentation. He decided that a method was needed which could be handled readily and which at all stages would reveal how the structure was supposed to behave. In the classical Melan analysis, the designer became trapped behind ponderous mathematical expressions which suddenly gave an answer that had no logical justification. A step-by-step method which could be examined as one went along was needed. That was the thinking behind the analysis methods that were adopted and used for every aspect of the interaction of the cable and the stiffening truss.

Mr C. Scruton (Aerodynamics Division, National Physical Laboratory) said that he could claim a very long, if intermittent, association with the aerodynamic problems of the design and the construction of the Forth Road Bridge, for it was in 1946 that the National Physical Laboratory was first requested by the Ministry of Transport to undertake an investigation into the wind-excited oscillations of suspension bridges, with the special purpose of assisting in the design of the Severn Bridge.

94. The various stages in the development of the design had been given in Fig. 2.1 in the present Paper. There was, in fact, no very significant improvement in the inherent aerodynamic characteristics of the section as between the through deck design of (c) and the top deck design of (d). The all-important feature of the top deck design, from the standpoint of aerodynamic oscillations,

was that it permitted the top laterals to be incorporated. This produced a very large increase in torsional stiffness, and a substantial increase in the torsional frequencies. No theory was available at that time for calculations of the torsional frequencies of a suspension bridge with a substantial torsional rigidity of the suspended platform, but Dr Frazer had developed one which took into account the discrete framing of the structure.

95. Speaking from memory, he believed that the increase in torsional stiffness of the isolated suspended platform was of the order of 100. The increase in the torsional frequency of the bridge as a whole was much less than that: it was increased by a factor of about 2. This substantial increase meant that the critical wind speed for oscillations also increased by a factor of 2. It also increased the ratio of the torsional to the bending frequencies in corresponding modes, thus reducing the possibility of coupled oscillations. Since the frequencies of oscillation of suspension bridges of the proportions of the Forth Bridge played such an important part in the design he would rectify their omission in the Paper by giving the values for the fundamental nodes.

Type of oscillation	N Fundamental c/s	First overtone
Symmetric flexure . .	0.149	
Antisymmetric flexure . .	0.125	0.293
Symmetric torsion . .	0.260	
Antisymmetric torsion . .	0.351	0.702

Had the top lateral bracing not been fitted, the torsional frequencies would have come down very much closer to the bending frequencies.

96. Much had been said about the troubles caused by wind during erection, and it might be asked whether those troubles could not have been foretold. In 1957 there was some interchange of views between the Consulting Engineers and himself with regard to the possibility of oscillations of the towers. He had then written to the Consulting Engineers saying that he was of the opinion that some consideration should be given to the possibility of tower oscillations being set up during the construction period before the cables were installed. He had continued that he felt quite certain that there would be some tendency for oscillations of the towers to be excited by the wind, in both bending and torsional modes, and that whether oscillations would occur would depend on the amount of structural damping. Model tests in a wind tunnel would indicate the wind speeds at which oscillations would occur, and the amount of structural damping necessary to prevent oscillations. This information, however, was not very useful for prediction purposes unless the damping value for the actual structure was known.

97. He had therefore suggested that recourse to previous experience of the behaviour of towers under construction might provide a more useful guide. He had not come across any reports of tower oscillations during construction of the Mackinac Bridge towers, the legs of which were of a cruciform section generally similar to that proposed for the Forth Bridge. This suggested that unless the Forth towers were of such a construction as to provide very much reduced structural damping than that of the Mackinac towers, no trouble was

likely to be experienced. Such a difference might arise if the Forth towers were to be of welded and the Mackinac of riveted construction.

98. He had concluded by saying that he thought that some provision should be made for adding damping to the towers, and he was assured at the time that the device provided had been designed for that purpose.

99. The birdcaging of the cables was not foreseen, but he thought that they were more fortunate with the main span erection. He thought that the credit for that must be given to the Contractors, who had asked for tests to be made before the erection started, so that it was possible to recommend the erection procedure for the steelwork described in § 4.94, and which had apparently proved successful.

100. One of the lessons learnt from the Forth Bridge was that it was not sufficient to establish the aerodynamic stability of the completed bridge, by which time it was at its strongest and had reached its maximum stiffness, but that wind effects on the bridge and its components during the erection stages must also be considered. In the case of the Severn Bridge a comprehensive series of tests had been carried out. He had been given to understand that so far there had been no trouble there from wind, although he thought it must be admitted that the wind conditions on the Severn site were probably much more favourable than those on the Forth Bridge site.

Mr T. J. Upstone (Dorman Long (Bridge & Engineering) Ltd) said that in the design of the suspension bridge the Engineers utilized two types of roadway deck—a steel deck for the main span and concrete decks for the side spans. As could be seen from Table 2.5, this resulted in the side span deck weighing some $1\frac{1}{2}$ tons/ft more than the main span. It was also found as a direct result of the difference in weight that the cables could be spun with the main towers vertical instead of having to be strained back temporarily towards the shore. However, the difference in deck construction had a marked effect on the erection of the suspended structure, as it imposed a more severe limit on the amount of main span deck which could be erected without over-straining the towers towards the river. Also, concreting the side spans had to be carried out in phase with the completion of the main span steel erection.

102. If the side span decks had been of the same steel construction as the main span, their erection would have phased-in much better, and the steel erection of both main and side spans would have been completed at about the same time. The complication of erection using two types of deck offset to a large extent the financial saving given in § 2.55, and it would be interesting to hear from the designers what allowance was made for the resulting differences in erection when deciding to adopt a concrete deck for the side spans.

103. The erection of the suspended structure of suspension bridges was always accompanied by the problem of distortion of the suspension cable. With the structure partially erected across the span the loaded portion of the cable supported the weight of the deck, partly by an increase in tension, but mainly by distorting to a greater curvature. Unless hangers were provided with considerable adjustment, as at the Tancarville Bridge, the bridge deck must also accommodate itself to the excessive cable curvature. It was necessary to know the form and extent of the distortion in order to decide how to articulate the stiffening trusses to permit them to take up such shapes. It

was also necessary to keep the longitudinal deflexion of the tower slope towards the main or side spans within limits as erection went ahead. From experiences with the Otto Beit and Menai Bridges, calculations of this type were known to be repetitive and very tedious, and it was realized that this was ideal work on which to use a computer. The problem was taken to the late Professor W. Merchant and Dr D. M. Brotton, of the Manchester College of Science and Technology, who succeeded in producing workable computer programmes. The programme, developed for an unstiffened suspension cable, gave the tension, the distorted shape and other information for any span, loaded non-uniformly, when the unstressed length of the cable and the span were known.^{D10} A further programme gave similar information for a stiffened suspension cable, together with the bending moments in the stiffening girders.^{D11} Using information from the computer, it was possible to determine the limitations which stress and distortion would place on any proposed erection sequence.

104. Fig. D3 showed the horizontal tension as determined by the computer plotted against the position of deck erection. The main and side spans were calculated independently. For each erection position the tension was calculated for the main towers vertical and also for each tower top deflected to its specified limiting position of 30 in. to the shore or to the river.

105. With the side span erected from the main tower to panel point 30 (point A) the horizontal tension in the side span would be 4000 tons with vertical towers. The main span would have to be erected to panel point 68 (point K) in order to balance this tension. If the main span was not built so far out, the excess of tension in the side span would pull the tower top towards the shore until a balance was obtained. The limiting position with the tower tops 30 in. towards the shore was represented by point B in the side span with a horizontal tension of 3300 tons and by point E in the main span which would be erected to panel point 58 (point J). The corresponding limitation of erection in the main span with the tower tops 30 in. towards the river was panel point 79 (point L) with a horizontal tension of 5100 tons. So with side span erection at panel point 30 (point A) main span erection must be between panel points 58 and 79 (points J and L).

106. An erection sequence was worked out and Fig. D4 showed the horizontal tension for each stage of erection plotted against the position of the erection in main and side spans. Now, however, the position of erection in each span was related and balanced and the amount the towers had to be out of plumb to achieve such a balance was given. The towers were leaning 2 in. towards the shore when erection was begun but after erecting 9 panels of steelwork, each side of the tower, the inclination of the towers had reached the practical limit of 26 in. towards the river. It was later necessary to stop the erection in the main span after 18 panels had been erected to allow the side span steelwork erection to be completed without exceptional difficulties being encountered at the connexion to the side towers. After that concrete was cast on the side span decks in a predetermined sequence as erection of steelwork continued in the main span.

107. The calculated vertical distortions of the deck steelwork as erection proceeded outwards from the main towers were plotted as shown on Fig. D5. The distortion was greatest when three or four panels were erected and in this condition the butt joints in the chords of the stiffening girders would be open

Fig. D3: Deck Erection Scheme A: Horizontal Tension in Main and Side Spans Plotted Against Position of Deck Erection

FIG. D4: STAGES OF ERECTION OF SUSPENDED STRUCTURE: PHASES I AND II

FIG. D5: DEFLECTIONS OF STIFFENING TRUSS RELATIVE TO DEAD LOAD DURING FIRST PHASE OF ERECTION

FIG. D6: NORTH AND SOUTH TOWERS: RECORD OF VERTICALITY

about 9 to 10 in. and the diagonal connexions would be out of line by 6 to 8 in. It would be seen from this that the reverse camber of the deck steelwork persisted in each individual span until that span had been virtually completed. It was, therefore, not possible to make the open and misaligned joints until a very late stage in the steelwork erection. The erection of the last panels of steelwork closed all these joints practically simultaneously and the opportunity had then to be taken of bolting them up as soon as possible.

108. A critical feature of the deck erection was the amount the towers were deflected from the vertical position. Fig. D6 showed the tower verticalities plotted against the erection of steelwork in the main and side spans and the casting of concrete in the side span. The general agreement between measured and calculated figures was good. The completion of side span concrete, completion of main span steelwork, addition of handrailing and protective grids and the surfacing of the whole bridge brought the towers practically to their full dead load vertical positions.

Sir John Howard (John Howard & Co. Ltd) said that Howards secured the sub-structure contract valued at £2½ million on a very tight schedule of completion, because it was vital that the work should be completed on time to allow the superstructure steelwork to proceed. They had had no form of target price contract.

110. The main work centred round the south pier, situated about 1250 ft from the south bank. It was decided to approach the working site by a causeway constructed of rubble rock and an open pile structure. After the contract was signed, this was strengthened so that the much heavier lorries which the steel contractor would require were able to get right up to the site of the pier.

111. The original form of the foundation for the south pier was rectangular. After a few extra borings on the site had been taken, Mr Gowing, the Chief engineer of the company, approached the Engineers to see whether they would agree to a foundation of a different shape for the south pier, and a circular foundation in the form of a figure-of-eight was developed, with two big monoliths. It was originally expected that compressed air would be necessary for sinking these, but it was hoped that it might be possible to eliminate the use of this.

112. The Engineers were adamant that they would only agree to a modification in the shape of the foundation provided that it did not increase the cost or extend the completion date. As it happened, the foundations were sunk without any mishap at all and without compressed air having to be used. An external lubricating system was used to get through the very stiff boulder clay to the rock and the two monoliths were sunk without any real difficulty. He thought that it was the first time that that method of sinking monoliths had been adopted.

113. There was little comment to make about the north pier foundation, except that the form of construction of the cofferdam was reinforced by internal tremie-type concrete work, which was found to be very successful.

114. With regard to the anchorage tunnels, on the south bank two 25 ft dia. tunnels were excavated and backfilled with concrete which was forced into the tunnels by concrete pumps. The north bank tunnels were 35 ft square and were constructed in the same way. Great care had to be exercised in the

construction of both these tunnels because of the very inferior rock encountered. A good deal of stiffening and support by steel rings was adopted as the tunnelling proceeded.

115. The total hours worked on the whole contract amounted to 1½ million man hours. The total number of men who passed through their books during the construction of the work was 1830 while at the peak period they employed about 360 men, with 45 other categories of staff. The total amount of excavation for the whole job was 60 000 cu. yd. The amount of concrete placed was 80 000 cu. yd and 26 000 super yards of shuttering were used as well as 1500 tons of mild steel.

116. Another interesting point was that in most of the concrete work flyash was used with the cement to prevent any undue heating in the big masses of concrete placed.

Mr D. A. Stephenson (Reed and Mallik Ltd) said that he had not joined the firm of Reed and Mallik Ltd until after the completion of the approach viaducts to the Forth Bridge, but on behalf of his firm and its chairman he wished to refer to the happy relationship that existed on site during the construction period. The successful outcome of the project was, he thought, in no small measure due to the atmosphere of friendly co-operation and understanding that existed under the excellent supervision of Dr Hamilton and his staff.

118. He wished to ask a question about the actual live loading on a suspension bridge. It would appear that the actual live loading could be recorded comparatively easily by measuring strain in the cables, deflexion at the tops of the towers, or the inclination of the cables at the towers. Did the Authors consider that such methods of recording were practicable, and, if so, that they would provide useful data for future bridge design?

119. Finally, he wondered whether the Authors had any information available as to the final outcome of the target cost contract for the superstructure. Was the target, in fact, achieved?

Dr A. J. Dutt (Senior Structural Engineer, Special Structures, Greater London Council) congratulated the Authors on their Paper. Proper assessment of wind loading for ensuring aerodynamic stability of the bridge, and thereby developing a safe yet economical design was one of vital importance in suspension bridge structure. He wished therefore to raise a few points on this and other related aspects.

121. In structural engineering, wind loading had been regarded as a quasi-static loading on certain types of buildings and bridges, i.e. static forces acting on a structure at rest, like the horizontal wind force taken into consideration in structural design. The severe warping of the Tacoma Narrows Bridge which finally led to its collapse demonstrated, however, that even under the action of a fairly steady wind, periodic forces might develop to start vibration with progressively increasing amplitudes which might reach dangerous or even catastrophic proportion. These vibrations had been recognized as self-excited oscillations resulting in aerodynamic instability of the vibrating structure.

122. A truss-stiffened suspension bridge under the action of horizontal

steady wind could be subject to the following phenomena:

- (a) motion in the direction of wind causing lateral deflexions;
- (b) vertical oscillation due to vortex shedding from the leading edge of the chords of the truss;
- (c) torsional oscillation due to the absence of dynamic symmetry of the section about its principal axes, in relation to the oncoming wind force;
- (d) flutter, in which effects of Karman vortices initiated a vibration after which the behaviour was associated with different phenomenon such as flutter. This was a major problem in aircraft structures but could also be important in suspension bridges.

123. These problems could be solved in the following ways, respectively:

- (a) by providing adequate stiffness in the truss to resist lateral bending;
- (b) by eliminating large girders and providing a system of deep-trussed stiffened type of suspension which would reduce the vortex shedding phenomenon to a secondary though not insignificant role;
- (c) by adopting a suspension system with dynamic symmetry about the principal axis in relation to the oncoming windstream as far as practicable and providing adequate torsional stiffness (this could be achieved by providing top and bottom lateral trusses and locating the deck level at the top region of the truss-stiffened girder);
- (d) by determination of the critical velocity of flutter and the flutter frequency. A theory relating to this phenomenon had been developed by the late Dr Bleich.^{D12}

124. However, the natural wind velocities were seldom steady. Fluctuations in magnitude and direction and the influence of the shape of the structure caused time variations of the actual forces resulting in a very complex type of loading, and amplification of load effect occurred. The solution to this problem lay in the analysis of random forces and the associated periodic forces due to gust on the suspension system, and was sufficiently important to warrant its inclusion in the aerodynamic behaviour of suspension bridges. Although some theoretical and experimental work in the wind tunnel had been carried out there was a lack of field investigation which might have given a clearer picture of the actual aerodynamic response of the suspension bridge under the action of gusty wind. Such field investigations should be carried out on different suspension bridges so as to provide quantitative information which would be useful for the design of future suspension bridges. This would also give an opportunity for correlating the theoretical and prototype results for specifying dynamic load factors. This should give a safe, yet economical design for a bridge. It might be mentioned that a double amplitude of 45 in. caused concern on the Golden Gate Bridge in the U.S.A.^{D13} in spite of extensive wind tunnel tests.

125. It had been mentioned in the Paper that a double amplitude of 7 ft 6 in. was observed on the main tower in its free-standing position even at low wind speeds of 20–25 miles/h. Dr Dutt had observed similar high amplitude oscillations while carrying out research with Professor Hendry on the aerodynamic response of free-standing structures under the action of natural gusty wind. The reasons were that the fundamental period of the tower in its free-standing condition was rather long and the gravity and downward loads were

small, and the stresses were almost entirely due to the effects of wind load. High amplitude oscillations disappeared with the erection of cables and superstructure when the towers were under heavy downward force. Dr Dutt would like to know if such oscillations were foreseen by the Authors or if they knew of their occurrence in other suspension bridges. It would be of interest to know if the observed oscillations agreed with the calculated deflexions.

126. It was stated in the Paper that 60% of the total wind force on the span was transferred to the cables by the lateral inclination of the hangers. Could the Authors elaborate on this point? Another point worth mentioning was the generation of vortices on the cable because of its cylindrical shape; there would be vertical oscillation of the cable which could be significant. It would be of interest to know if such oscillations were observed in the wind tunnel tests and on the actual bridge. Such oscillations might be more significant in the upper region of the cable where it would experience changes in wind velocity almost instantaneously.

127. Dr Dutt asked if bolted or riveted joints had been considered in place of welded construction in the chord members and towers. Bolted connexions had the particular advantage of increasing the value of structural damping to a large extent. This also minimized the possibility of fatigue in the joints.

128. He enquired if there was any possibility of a portion of the tower being submerged in water during high tide combined with gale as had been experienced in the river Thames during the first week of December 1965. In that case the towers would be subject to the combined dynamic effects of gust and wave impact.

Mr W. I. J. Price (Road Research Laboratory) proposed to direct his remarks to the parts of the Paper dealing with the roadway deck. It was shown in §§ 2.139–2.143 that, although the actual deck section used was not tested, the stresses likely under the design loading were within permissible limits. There were, however, other important criteria in addition to strength to be taken into account in the design of such a structure, namely, durability of the surfacing and fatigue of the welded connexions. The former was dealt with fully in the Paper, but there was no mention of the latter.

130. He had little connexion with the earlier experiments referred to in §§ 2.59–2.68 but had been involved in the final selection of the surfacing system. In the interim period there were changes in the form of the deck structure which substantially modified the interpretation of the results of the earlier experiments. The final recommendations which the Authors accepted with misgiving were considered the most suitable in the light of British and German experience.

131. The introduction of the rubber bitumen layer conferred several advantages, including better adhesion between surfacing and deck, better water-proofing of the deck and a reduction of the effect of differential thermal movements. In § 4.116, it was recorded that the asphalt cracked over the stringer in the near-side traffic lane, during the first winter of 1964/65. Sections of the surfacing were subsequently removed and there were signs that water had penetrated to the steel in a few areas, and a whitish powder similar to the corrosion products of zinc was evident (Fig. D7). It was as well, therefore, that a rubber bitumen membrane was laid to impede the movement of water

FIG. D7: DECK PLATE WITH ASPHALT REMOVED

from the cracked area, even though it did not entirely prevent penetration of water through the crack itself.

132. Fig. D7 showed the deck where a 6 ft wide strip of surfacing was removed over the whole of one panel where the crack had developed. The line of the crack was apparent some 6 in. to the right of the weld in the deck. The mastic was relaid on this trial section during the summer of 1965 and it appeared that a crack had again developed in the same position in the new mastic during the cold weather of December 1965. This partly invalidated the explanation for the formation of the crack given in § 4.116.

133. There was no alternative explanation which was entirely satisfactory. The transverse stresses measured on the surface of the steel were higher in the region of the trough stiffeners than over the stringers. It was possible that the strain gauges, having a gauge length of $\frac{3}{8}$ in., did not record localized peak stresses. The measured transverse distribution of wheels showed that fewer wheels ran along the stringer than along the trough stiffeners to each side. If local yielding of the plate, referred to in § 4.116, had taken place on more than one occasion, the provision of a surfacing to withstand the resulting movement as well as the usual traffic stresses, without cracking, was likely to be a very stringent requirement. The formation of a joint over the line of maximum change in plate curvature seemed to be the best practical answer.

134. With reference to the fatigue problem, how important did the Authors consider it to be and how was it dealt with?

Mr J. Kinsella said that having been associated with the erection of the bridge superstructure there are a few comments he would like to make.

136. It might be worth recording the part played by the site workshop. This was a building 60 ft by 40 ft in area and in addition to the usual small equipment contained a lathe, a radial drill, power hacksaw, screwing machine, welding bay and blacksmith's shop and had an electrician's shop attached. In addition to routine plant and equipment maintenance this shop turned out a vast quantity of staging frames, the movable platforms for the deck welders, the steelwork for the deck shutters, the cross-heads for the diagonal wind ties, etc. The part which it played in sorting out the teething troubles with the new and untried cable spinning, compacting and wrapping equipment was highly commendable.

137. Referring to the figures for cable spinning progress (§ 4.61) it was interesting to put these in another way. If one looked at the number of wheel trips actually made in a given period and the number which could have been made in that period after allowing for direct weather loss and plant and equipment failures, and expressed this ratio as a percentage, which might be called the *labour efficiency*, the following figures were obtained:

For the period November 1961–February 1962: labour efficiency 35%		
” ” ”	March–April, 1962:	” ” 55%
” ” ”	May–August, 1962:	” ” 80%

138. The increase in efficiency was partly explained by increase in familiarity with the process though there was ample time for learning in the early stages. What was far more significant was the effect on these figures of the better weather. The slowing down of production caused by the rigorous conditions of the winter of 1961–62 over and above direct weather losses could not be over-emphasized.

139. Cable band bolt-tightening was an extremely important part of the job. Due to progressive relaxation as the cable load built up it was necessary to re-tighten the bolts at intervals. All load-carrying bands were tightened at least four times while those on the steeper part of the cables had an extra tightening to make a total of five. Initial tightening was done to stretch each bolt 90–100 thousandths of an inch and so take it beyond the yield. Subsequent tightenings were done to restore the relaxation with an additional allowance for permanent set. The relaxations for the side span cable band on the east side adjacent to the north main tower given in Table D1 were fairly typical.

140. This covered a period from October 1962 to May 1964, during which time the suspended structure was erected and the wrapping done. It was

TABLE D1: CABLE BAND BOLT RELAXATIONS

Measurements made	Relaxation. 10^{-3} in.
Prior to 2nd tightening	25–28
” ” 3rd ”	30–50
” ” 4th ”	10–30
” ” 5th ”	12–17
Some time after 5th tightening	1–7

interesting to note that in addition to the effects of increase in cable load, bedding-in of wires and so on, it was considered that plastic flow of the zinc galvanizing on the wires due to the high cable band pressure was also significant.

141. A good deal had been said about the safety nets and their usefulness. One point worth emphasizing was the confidence they imparted to men moving relatively freely about the steelwork to deal with troubles, in emergency and storm conditions. The collapse of the north main span safety net frame was a tragic occurrence. It was significant to note that there was a minor structural failure—for no clear reason—associated with the removal of the south main span frame, whereas both side span frames were lowered without incident. One wondered if the more pronounced movements of the main span steelwork during erection might have induced some weakness in the frames attached to it.

142. Reference had been made in the Paper (§ 4.86) to the difficulties in closing the bottom chord joints of the stiffening truss, mainly because of misalignment at the butts caused by wind movements. It should be mentioned that the closing of the bottom laterals after the chord joints had been made was also extremely difficult because the bottom chords were invariably marginally too wide apart. Straining gear of 20 ton capacity was devised across them and sets of this gear combined with progressive heavy drifting of a series of joints successively brought the joints fair. The straining gear generally reduced an initial lack of fit of the order of $\frac{1}{8}$ in. by $\frac{1}{4}$ in. or so, leaving the rest to be dealt with by the combination of drifting and straining. On one or two occasions advantage was taken of wind deflexions but of course winds which were significant generally precluded work on the stagings.

143. The method of shutter heating adopted was of interest. As a guide to its effectiveness the following figures, taken after the hot water had been circulating under a shutter for some 6–8 hours, were a fair average.

Air temperature	30°F
Temperature of water in tank	150°F
Temperature in boxed-in pocket under shutter cladding	40°F
Temperature under the tarpaulin covering the reinforcement and shutter ready for concreting	33°F

144. Finally Mr Kinsella mentioned some figures for the positions of the main and side tower saddles when surveyed shortly before the opening of the bridge. This was at a stage approaching full dead load, the only omissions being the main span footway, cycle deck parapets and surfacing, and the side span footway. When converted to the standard 60°F the north main tower was leaning 5.4 in. to the shore and the south main tower 4 in. to the shore. Again at 68°F, the north side tower saddles were out of plumb to the shore by the following amounts:

north-west	$\frac{1}{2}$ in.
north-east	$\frac{3}{8}$ in.
south-west	$\frac{1}{16}$ in.
south-east	$\frac{1}{2}$ in.

145. The addition of the missing dead load would have improved all these figures substantially but in any case too much reliance could not be placed on them due to the difficulty of estimating the temperature of the wrapped cable. However, they were a guide to the remarkable accuracy that could be obtained.

Mr D. H. Little (Messrs Walter C. Andrews & Partners) said that at £5/sq. ft the viaduct cost seemed to him to be very low. The main span worked out at £22/sq. ft, which also seemed low. He wondered whether the Authors could enlarge on this point and indicate whether all the latest techniques which had been embodied in the project had led to economies when compared with other structures of this nature.

147. His main concern was the north pier. In the early days Mr Hamilton had asked the Admiral Superintendent of Rosyth dockyard if he might be allowed to use naval divers to examine the rock at the north pier underwater point. At that time Mr Little had been in charge of civil engineering works at the Dockyard and the Admiral had consulted him about the request. As a result Mr Little was shown the drawings of the main pier. The discovery that it was to be founded on underwater concrete had been a severe shock, as he himself would never use underwater concrete unless there was absolutely no other alternative.

148. He had explained this as forcibly as he could to the Admiral Superintendent, and suggested that it would not be difficult to plant two rings of sheet piling on the rock round the foundations and fill between with underwater concrete. With this as a dam it would be possible to pump out the foundations and examine the rock in the dry, and the foundation concrete could also be placed in the dry. He could assure those present that if half a dozen divers had been sent down to examine the rock, three of them would have said there was none there at all.

149. The Admiral had accepted Mr Little's advice, and naval divers were not made available. Mr Little would like to ask the Authors if this had had any bearing on the decision to amend the design.

The **President** (Mr J. A. Banks) remarked on Mr Shirley-Smith's reference to speed on the job and to accidents, and said it was of interest to compare the number of accidents with what had occurred during erection of the Forth Railway Bridge.

151. When the Railway Bridge was constructed, the safety provisions adopted were advanced for the period. Nevertheless, in a comprehensive article on the Bridge published in *Engineering* in 1890, it was said that between July, 1883, and Christmas, 1889, there had occurred 57 fatal accidents, 106 accidents which required removal to the infirmary in Edinburgh (in which number some of the fatal accidents were included, the men having died after admission) and 518 minor accidents which required the attendance of a medical man.

152. On the Forth Road Bridge, four men lost their lives in the course of construction of the main bridge, which was the comparative figure, and three lives were lost on the approaches.

153. One could not too greatly emphasize one's admiration for the care that had been taken by all concerned with the project to ensure safety and, though unfortunately there were some accidents, one had the gratification of knowing that everything that could possibly have been done in the interests of safety had been done.

The following contributions were received in writing:

Mr W. Wright (Senior Lecturer in Civil Engineering, Enfield College of Technology) asked if any use was made of aerial photographs of the site and approach roads.

155. In § 3.1 reference was made to a difference of 0.1 ft in two measurements of the distance across the Forth. Did this have any serious consequences? Was a further check made on this distance after completion of the bridge to indicate which of the two methods of measurement was the more accurate?

Mr G. W. Morley (formerly Chief Engineer, A.C.D. Bridge Co.) wished to comment on certain aspects of the erection of the superstructure of the bridge, namely cable spinning and aerodynamic effects.

157. When describing the main cables, the Authors stated that 'no parallel wire cables of great span had been erected on any bridge except by the process of cable spinning'. This was true in regard to large spans, but in 1938, the Otto Beit Suspension Bridge was constructed with parallel wires and a main span of 1050 ft. There the wires were laid by a different method involving the pulling across of complete strands of flat ribbons of wires.^{DB} Mr Morley asked whether, as a result of their experiences at Forth, the Authors would recommend the Roebling spinning method for future suspension bridges in similar situations or would attempt to develop the method used at Otto Beit.

158. At Forth some 33% of cable spinning time was lost because of weather conditions and it was found impracticable to adjust wires in winds over 30 mile/h. He believed that on many occasions spinning had to stop because wire adjusting could not continue, although the men were still willing to work and the actual laying of wires was still possible.

159. The spinning process at Forth involved the making of 116 402 separate adjustments, viz. five to each wire and three to each strand of each cable. For each one of these adjustments reasonable weather conditions, with winds under 30 mile/h, were required. At the Otto Beit Bridge only 80 adjustments were made, viz. no individual wires, and strand adjustment in the main span only. There was clearly an enormous difference in the number of periods of suitable weather needed for making adjustments for the two methods of laying wires. Mr Morley suggested that the Otto Beit method might show great advantages in time over the Roebling method in situations where bad weather conditions were likely.

160. The equipment used at Otto Beit was very simple and rather crude, but there would seem to be plenty of scope to make it more sophisticated while still retaining the adjusting advantages outlined above. The method suffered from other disadvantages, but it was suggested that these could be overcome. The Authors' views on this matter would be welcome.

161. The aerodynamic oscillations of the north tower when the wind was comparatively light had not been anticipated by the Contractor and no provision had been made in the erection scheme to counteract them. Fortunately the Consulting Engineers quickly developed a damping system which proved entirely effective. The Contractors then learnt from Mr Scruton of the National Physical Laboratory that some years previously he had informed the Engineers that in his opinion such oscillations were likely to occur. Had this

opinion been passed on to the Contractors some form of damping system could have been incorporated in the erection scheme and a period of some anxiety and danger to the Contractor would have been avoided.

162. In the Paper, and in the discussion, attention had been drawn to the shape of the suspended structure when partially erected. On the Forth Bridge the joints in the bottom chords of the stiffening trusses were open as much as 14 in. at times, the diagonal members could not be connected at the lower gussets, and the bottom laterals had to be left open at their central inter-sections. One great inconvenience of the method of erection adopted was that when the suspended structure began to approximate to its final shape all these joints closed at the same time, and the Contractor found that he needed large quantities of stagings and men to complete the connexions within a very short period of time. When second pass erection was begun, it produced a deck cross section which was aerodynamically unstable unless the suspended structure had its full final torsional stiffness. To achieve this the bottom lateral connexions had to be made simultaneously with the adjacent stiffening truss joints. This added still more to the number of points to be staged at the same time.

Mr V. F. Lindsey (formerly Civil Engineer, National Coal Board Opencast Executive) wrote that in § 5.30 it was stated that tests had established that blasting on the scale proposed would cause no damage to a building at a distance of 300 ft.

164. During excavations on opencast coal sites in Northumberland, the measured amplitude of ground vibration caused by blasting seemed to conform to no pattern which could be related to observed phenomena, e.g. weight of explosive, distance and direction of seismograph from explosion, nature and dip of strata, weather conditions, etc. The locations of buildings for which claims for damage were investigated similarly formed no logical pattern.

165. In view of this experience it would be interesting to know if the conclusions drawn from the Forth Bridge tests were fully vindicated.

Mr B. G. Smith (Severn Suspension Bridge) commented on the 'target' type of 'cost-plus' contract for the erection of the bridge superstructure, described by the Authors in §§ 1.31–1.36. This was a negotiated contract and, with the advantages briefly mentioned in § 1.36, it was most suitable and worked well.

167. Competition in the construction industry was intense and, though this might be very good in some respects, larger and larger claims at the termination of contracts seemed to be the accepted practice. Mr Smith wondered if the 'target' type of contract might be usefully tried on other contracts let by competitive tendering. Tenders could be invited for target prices, which could still be made up from prepared bills of quantities, if necessary, and the actual work then 'costed' as was done at the Forth Road Bridge.

168. Whatever form of contract was let, the relationship between the ultimate cost and the contract price depended largely on the adequacy of preparation and on preliminary investigations, on the part of both engineers and contractors. If the advantages to be gained from competitive 'target' type contracts were to be realized, these preparations and investigations would have to be just as thorough, perhaps even more so, than with the standard type of contract now used. On the other hand, the attitude of employers and engineers

to claims would have to be more rigorous than at present and the contract documents so worded as to eliminate many of these. There were obviously many pitfalls which would require thought and the possibilities for variation in the 'target' condition were numerous. Did the Authors know whether contracts of this type had been let in the past or whether any authorities had considered their possibilities?

169. It might be thought that the clerical work entailed in checking all the Contractor's costs at site would be excessive. On the Forth Road Bridge, the site costs were sub-divided into approximately 40 headings, covering salaries, wages, insurances, materials, plant depreciation, small tools, temporary works, etc., and the checking of them was carried out by two clerks in the Resident Engineer's office, who had other work to do as well. Mr Smith considered that the accountants who organized and put into operation the system which enabled this work to be carried out so efficiently and economically, were deserving of great credit.

Dr B. Richmond (Battersea College of Technology) wrote that he wished to comment on the tests carried out on compression members and the basis of design for such members. He was particularly interested in relatively simple theories of local buckling which were nevertheless able to reflect the effects of geometry, material characteristics and residual stresses. Table D2 showed a comparison between experimental failure stresses and various theories of buckling. In order to make the comparison more useful, the results of a series of experiments by Dwight and Harrison^{D15} on mild steel box columns were also compared with the same theories. These tests on mild steel were carried out after the Forth Bridge tests. The columns used were sufficiently short for local buckling to be the only significant buckling mode. Tests were carried out on both 'as welded' and stress-relieved columns.

171. The headings to the theoretical values in the Table were briefly indicated below:

- (a) *Ideal buckling stress.* The theory of initially flat plates gave the buckling stress of long plates with simply supported edges as

$$\sigma_{cr} = \frac{4\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2$$

for uniform compressive stresses acting in the longitudinal direction. The ideal buckling stress was found from this formula assuming that E was constant at about 13 400 tonf/sq. in.

- (b) *DIN 4114 buckling stress.* The German specification for buckling, DIN 4114^{D16} gave a correction to compensate for the proportional limit of structural steels being lower than the yield stress. The constant E was replaced by a reduced modulus of elasticity, T ,

$$T = \frac{4E_t E}{(\sqrt{E_t} + \sqrt{E})^2}$$

where E_t was the tangent modulus. The method of calculating E_t given in DIN 4114 was assumed correct for these values of buckling stress as more relevant values were not available.

- (c) *DIN 4114 buckling stress adjusted for residual welding stresses.* The above values for mild steel were corrected for the effects of residual

stresses by multiplying the average residual compressive stress found experimentally by Dwight and Harrison by the ratio buckling stress (b)/buckling stress (a) and subtracting this amount from the DIN 4114 value in column (b), e.g. for $b/t = 30.7$ the correction was $4.0 \times (15.9/50.5) = 1.3$ tonf/sq. in. No figures were available for the residual stresses in high tensile steel and so they were assumed to be the same, for equal b/t ratios, as those in the mild steel columns.^{D17}

- (d) *Buckling stress according to Bleich.* Bleich^{D18} has shown for plates in compression that the non-linearity of the stress strain curve for steel could be allowed for by replacing E by $\sqrt{E E_t}$. The values of E_t he gave were used for these values of buckling stress.
- (e) *Buckling stress adjusted for residual welding stress using effective tangent modulus.* The Perry–Robertson formula could be used to estimate the buckling stress of a hot-rolled structural section, but the primary cause of column buckling of these members was not due to the out-of-straightness assumed in the formula but the residual compressive stresses produced by cooling. These caused local yielding at average stresses lower than the yield stress of the material, reducing the bending stiffness of the column. The buckling stress could also be represented by the Euler buckling stress if a reduced modulus was used which could be called the effective tangent modulus, $\bar{E}_t$. $\bar{E}_t$ values could be calculated from the Perry–Robertson formula. If it was assumed that the residual stresses due to welding were comparable to those in rolled sections then it could be further assumed that they both have the same values of the effective tangent modulus. Using Bleich's theory, the buckling stress could be found by using $\sqrt{E \bar{E}_t}$ in place of E .

172. Table D2 showed good agreement between the theoretical buckling stresses and the experimental failure stresses when allowance was made for residual stresses and non-linearity, except when $b/t = 77.7$. At this point there was clearly a considerable post-buckling reserve of strength.

173. The high tensile steel columns did not show such good agreement. The first two columns with low b/t ratios failed at rather lower stresses than predicted by theory but the two with medium b/t ratios were in better agreement, particularly values (e).

174. The discrepancy in the first two might be due to the participation of overall column buckling, but even so the failure stresses suggested that the properties of the steel and the effect of welding were such that the Perry–Robertson formula gave optimistic values at high stresses. It would be of interest to know on what grounds the Authors decided that the B.S. 153 stresses were safe, since even with a 30% lower l/r ratio and almost the same b/t ratio the test column failed at a lower stress (17.0) than the B.S. 153 value multiplied by a load factor of 1.7 ($11.1 \times 1.7 = 18.8$).

175. Dr Richmond thought that the above comparison showed that it was possible to produce a British specification covering column behaviour which was firmly linked to a simple structural theory. It had been suggested in the U.S.A.^{D17} that welded columns should be designed on a different basis from rolled members but he hoped that by using welds and welding techniques which

TABLE D2: LOCAL BUCKLING OF WELDED STEEL COLUMNS

Experimental results (average values)			Theoretical values					
Dwight and Harrison ¹⁵ series			(a)	(b)	(c)	(d)	(e)	b/t
b/t	Yield stress of column	Failure stress of column						
W*30.7	16.2	15.2	50.5	15.9	14.6		14.8	30.7
R†30.9	14.7	14.8	49.8	14.5		14.5		30.9
W 40.0	17.6	14.9	30.2	16.7	15.0		14.8	40.0
R 40.0	15.1	15.2	30.2	14.7		14.4		40.0
W 49.4	16.7	13.4	19.6	15.4	13.7		12.3	49.4
R 49.9	15.3	14.4	19.2	14.2		13.8		49.9
W 59.6	18.2	10.8	13.4	13.4	11.1		10.5	59.6
R 59.7	16.5	13.1	13.4	13.3		12.8		59.7
W 77.7	20.4	9.8	8.0	8.0	6.7		7.1	77.7
R 78.8	21.5	9.2	7.8	7.8		7.8		78.8
Forth Bridge series (Table 2.18)			(a)	(b)	(c)	(d)	(e)	b/t
b/t	Yield stress of column	Failure stress of column						
W 32.5	21.15	17.0	45.5	20.4	18.6		18.7	32.5
W 39.6	24.65	17.1	30.9	22.8	20.4		19.4	39.6
W 44.6	22.9	16.3	24.2	20.6	18.0		16.8	44.6
W 60.5	23.4	11.1	13.2	13.2	10.9		11.1	60.5
W 82.1	24.1	8.9	7.2	7.2	5.9		6.6	82.1

N.B. The values taken from Table 2.18 are for the tests on models 1, 2, 3, and 16, 17.
All models had an l/r ratio of about 20.

* W=as welded condition.

† R=stress relieved condition.

produced the minimum residual stresses, this could be avoided. The use of a factor of 0.8 for chords to produce an effective length of pin-ended strut could, however, be dangerous as it was not relevant to the buckling of straight members with residual stresses. The chords in the Forth Bridge had low l/r ratios and variations in the effective length were not so important.

Mr G. B. Gowing (Chief Engineer, John Howard & Co. Ltd) wrote that in carrying out the south main pier construction, certain unusual problems came up for solution.

FIG. D8: CAISSON INSTALLATION

177. One which arose was the design of the circular steel walings in the cofferdam. Theoretically, if the waling was truly circular, the loading was axial, and the waling was considered continuously supported by the piles, the full allowable stress in compression members could be used. Such assumptions would be very risky and so certain deviations from circularity were assumed, which resulted in a design in which half the maximum stress was due to direct compression and half due to local bending. Some simple strain recordings were made on site, by measurements on the neutral axis and on the flanges of the waling, of the difference in strain at low and high tide. The results obtained were rather erratic, but in general they indicated that the strain in the flanges could reach a value of twice the strain on the neutral axis. A similar basis was used for circular cofferdams on the west pier of the Severn Bridge with satisfactory results.

178. The second was the amount of increase to be allowed in the radius of the caisson cutting edge over the radius of the caisson walls, in order to provide a working clearance to allow for irregularities in the caisson walls. In sinking through a very hard material such as boulder clay, which could have a passive resistance of 20 tons/sq. ft, it was important that the caisson should not become wedged in the hole excavated, as the force necessary to remould the sides of the excavation could easily exceed the available sinking weight. To be safe, a clearance of 2 in. all round was provided, but as the Engineers wished that side support of the pier shaft should be available, the cavity was filled with bentonite during construction and the bentonite was displaced by cement grout on completion. An interesting side-effect of the bentonite filling was the greatly reduced skin friction during sinking, an effect which had proved useful on subsequent occasions.

179. The third point was the behaviour of the cofferdam sheet piles after the caisson excavation had proceeded below the toes of the piles. The 2 in. clearance mentioned in § 178 meant that the piles could move inwards by this amount unless restrained by the clay. Such movement could have caused heavy leakage in the cofferdam and also increased skin friction on the caissons.

Calculations showed that the combined effect of arching action in the ring of clay plus the shear value at the base of the piles (Fig. D8) was sufficient to prevent movement. The arching action was only possible with the shape of cofferdam employed.

Mr R. W. Bishop (Ministry of Public Building and Works) wrote that, at the time the underwater bridge was being considered, he had been stationed at Rosyth and there had been talk of using the dockyard dry-docks for the construction of the reinforced concrete tubes. The scheme was ingenious, but there was a deep channel to be spanned in the sea-bed of the Forth and it was therefore surprising to learn that the cost of this scheme had been estimated to be similar to that of a suspension bridge.

181. However, even in those days, it was the idea of a suspension bridge which had really captured the imagination, but to Ministry, then Admiralty, engineers experienced in underwater work, it seemed that the foundations would present the biggest problem. The Beamer Rock, which was exposed at low water, seemed the obvious site for the north foundation, but it was equally obvious that the construction of a pier to carry an exceptionally heavy load in the deep water off Port Edgar would be a formidable task.

182. The way in which the problem was eventually solved reflected great credit on the engineers concerned; their bold approach achieved the success it deserved. But what interested Mr Bishop in particular was the use of a drum shutter and intrusion-grouted aggregate to form the lower waling of the cofferdam. The grouting was a continuous process over nearly four days and it would be interesting to know the procedure adopted.

183. The north pier was founded on the Mackintosh Rock, a whinstone outcrop, and it was not quite clear why 15 ft of rock had to be blasted away. Indeed, as the water over the rock was so shallow, one could not help wondering if a scheme involving more extensive use of divers had been considered.

184. The surfaces of the piers had been finished with a transferred aggregate. This had been used extensively in the U.S.A. and it would be interesting to know how the aggregate was applied.

Mr A. C. Patterson (Partner, F. R. Bullen & Partners (Consulting Engineers)) wrote that the geology on both sides of the Firth varied rapidly as one moved inland. On the north side particularly the conditions encountered gave rise to a number of problems of soil mechanics, and his firm were asked by Messrs Mott, Hay, and Anderson to investigate and report on a number of these. One of the most interesting and important was the cutting at Castland-hill, to which reference was made in the Paper.

186. The cutting had originally been intended to be some 35–40 ft deep for which a side slope of $1\frac{1}{2}$ to 1 was proposed as giving a factor of safety of 1.9 on a total stress basis. It was later decided to deepen the cutting to 90 ft and the initial intention was to introduce two 15 ft wide berms at depths of 30 ft and 60 ft, thus reducing the side slopes to an equivalent continuous slope of 1.83 to 1. When the cutting was excavated to a depth of 60 ft, cracks were noticed at the original ground level, about 100 ft from the top of the new slope. Although the cracks were not considered to be conclusive evidence of an incipient slip, they had been asked to investigate and report on the general stability of the cutting.

187. Five additional boreholes were sunk and revealed boulder clay overlying shale at depths of 95 to 105 ft below ground level, and fourteen 4 in. diameter samples from the boreholes were subjected by them to several different types of test in the Engineering Laboratories at Glasgow University. These gave very consistent values for the effective stress parameters of the clay. Slope stability co-efficients were used for effective stress analyses and some extrapolation beyond published results was necessary. The factors of safety produced by effective stress analyses were of course very much influenced by the average pore pressure assumed to be present in the potential failure zone. The probable range of pore pressure which might be expected over the long-term life of the cutting was considered and a range of -0.3 to $+0.3$ for the pore pressure ratio was thought to be appropriate.

188. As it was not possible to forecast accurately the rate at which the pore pressure would increase, piezometers were installed to allow this to be measured. These had now been established for just over two years and the latest readings showed a very slow rate of change.

189. The British Standard Code of Practice for Earthworks recommended a factor of safety of between 1.2 and 2 for the slopes of cuttings and it was here thought advisable to relate a high positive pore pressure to a minimum factor of safety provided this did not imply an excessive factor of safety under more probable conditions.

190. With the slope originally proposed equivalent to 1.83 to 1, the factor of safety at zero pore pressure would have been 1.5, but with a pore pressure ratio of $+0.3$ the factor of safety would have been reduced to 1.2. It was eventually decided to retain the $1\frac{1}{2}$ to 1 slope with the two 15 ft wide berms for the lower 60 ft of the cutting but to flatten the top 30 ft to a slope of 3 to 1. This provided a factor of safety of 1.3 for a pore pressure ratio of $+0.3$.

191. These factors of safety were based on the peak shear strengths of the clay as the investigation preceded by some two years Professor Skempton's Rankine Lecture to the British Geotechnical Society on the long-term stability of clay slopes,^{p19} but during periodic consideration of the long-term stability of Castlandhill the contents of the lecture had been kept very much in mind.

Mr T. M. Young (Resident Engineer, New Scotswood Bridge, formerly Section Engineer, Forth Road Bridge Project) wrote that it had been a great privilege to work for six years with such eminent civil engineers on such an important project. He had been employed as a member of the Resident Engineer's staff on the north side foundation work of Contract No. 1, and the north side approach road (Contracts Nos 5 and 8).

193. The Mackintosh Rock site work for the main north tower base had not proved as easy as expected because of overlying boulder clay, silt, and the wreckage of ships. The intrusion grout scheme for the base might also have left some doubts as to the quality of the concrete. However, the Contractor's proposals had allowed all concrete to be both placed and checked in the dry. The forming of a dried-out area 100 ft $\times$ 35 ft at a base level of -30 ft O.D. average, inside the 168 ft $\times$ 55 ft cofferdam reflected great credit on the Contractors.

194. Borehole investigations had sometimes had to be repeated during the contracts on revised routes, but a very full borehole coverage was available throughout the scheme.

195. The approach road costs of over £5½ million were mainly in addition to those estimated for the 1948 scheme. Practically all of this sum would therefore be met by Government loan, and with interest charges, the repayment due from the Forth Road Bridge Joint Board, by way of the bridge tolls, would come to approximately £10 million for the road approaches. This was almost 80% of the basic cost of the bridge (shore-to-shore unit) and seemed a high price to pay. The fact that some of the roads had now been designated as 'motorways' would indicate the importance attached to them. In passing, he would like to mention that Contract No. 1 for the main bridge foundations had been started in November 1958, not November 1959.

196. In relation to the north anchorage, great care was taken and an excellent procedure adopted by the Contractor for the excavation and roof-strengthening of the cavities. For twin caverns 42 ft square at the rear and with a whinstone rock pillar of only 26 ft width between them, the rock bolting and arch rib work had to be good. No major rock falls occurred, and the safety record here was high. The change from tunnel work to an open-cut scheme at St Margaret's Head, based on the Contractor's scheme, was a major improvement, and eliminated very tricky tunnel work in almost vertically fissured whinstone.

197. Mr Young thought that the time allowed for completion of the north approach roadworks had been too short, considering the volume of upfill which had to be placed over boggy and soft clay areas. However, the time was overrun by one year, and this allowed most of the embankment settlement in the poor formations to take place. Surcharge work had proved useful in certain areas in this respect, and the use of a 3 ft drainage layer at the base of the embankments had helped considerably. Vertical sand drains to accelerate settlement might have given further improvement, but the worst areas were subject to periodic floods, and such drains needed good run-off conditions to be effective.

198. The fact that the fly-over bridges were not standardized designs increased bridge costs but allowed designers scope to produce several interesting and pleasant looking structures. Probably the most prominent and pleasing of these was the Dunfermline Wynd Bridge (Fig. 5.12), a reinforced concrete arched cantilever structure of three spans with a suspended centredeck section and the piers outside the dual carriageway. As it had been constructed at the summit of a main road cutting, it appeared in silhouette against the skyline to travellers in both directions on the main north road.

Mr A. W. Shilston (Consulting Engineer) referred to the contract arrangements for the main bridge superstructure described in §§ 1.30 to 1.36, which appeared to him to be eminently realistic, having regard to the nature of the project.

200. Negotiated contracts in general tended to imply a closer working relationship between the parties than was customary in the more conventional contract entered into as the result of a tender competition. In the last sentence of § 1.36 the Authors underlined the extent of the co-operation in the Forth Bridge superstructure contract in referring to the sharing, by the Client, in any savings resulting from ingenuity displayed by either the Contractor or the Engineers.

201. Although closer co-operation in civil engineering contracts was

generally desirable in itself, this could mean that the parties to the contract tended to assume legal liabilities which could vary from those which existed in a normal I.C.E. Form of Contract. Mr Shilston felt, accordingly, that the boundaries of co-operation in such negotiated contracts should be clearly established from the onset. Could the Authors comment on that aspect of the contractual relationship, indicating whether full responsibility for construction resided in the Contractor throughout? In other words, were changes in construction methods which arose from ideas originating on the Engineers' side the result of a persuasive process, with the Contractor, in effect, final arbiter, or did the Client assume joint responsibility for agreed deviations from the initial construction theme?

202. The general contractual position of the engineer in such schemes tended to become slightly blurred when one considered his moral and, in England perhaps, Common Law responsibilities to satisfy himself on the adequacy of temporary works from the point of view of safety of the workmen. The discussion on the failure of the Second Narrows Bridge, Vancouver,^{D20} sprang to mind in that connexion. Was the position of the Engineers different in this respect from that which existed in a normal competitive tendered steel bridge contract?

203. Was the cost of financing the superstructure contract reflected in the 'fixed fee' profit which was negotiated? If so, had consideration been given by the Client to assuming direct responsibility for funding the work as distinct from remunerating the Contractor in the more usual way by interim payments at, say, monthly intervals?

Dr F. Stüssi (Professor, Eidg. Technische Hochschule, Zurich) wrote that the Forth Road Bridge was the first European bridge with a main span of more than 1000 metres. The British engineering profession was to be congratulated for this excellent and important engineering work.

205. He wished to make some remarks on the calculations for the suspended structure. In §§ 2.119, 2.120 and 2.121 the method of calculating the maximum bending moments in the main span stiffening truss was given. This seemed rather long and tedious. It could be greatly simplified by using 'linearized' influence lines.^{D21} If the total cable pull N were a constant, normal influence lines could be calculated and used. This, however, was not the case but it was possible to introduce into the calculations such a constant value of N as a hypothesis, although the resulting influence line was exactly valid only for a loading case that gave the introduced total cable pull. Fig. D9 gave such influence lines, calculated for the design of the Orinoco River Bridge near Ciudad Bolivar in Venezuela with a main span of 712 metres between towers and 704.4 metres for the suspended structure, for the cable pull (e_x), for the bending moment in the quarter point (e_{m_m}) and for the shear force in the quarter point (e_{q_m}) of the main span, the first two calculated for the minimum and the maximum of the total cable pull, $N = 6400$ tons and $N = 7800$ tons.

206. As could be seen from this example, the loading length giving the maximum bending moment at point m was not greatly affected by the value of the introduced cable pull N . There were therefore two possibilities in applying such influence lines: (a) it was possible to introduce into the calculation an average value of the cable pull and use from the influence line only the loading length; (b) influence lines with N_{min} and N_{max} could be calculated. By

FIG. D9: INFLUENCE LINES FOR BENDING MOMENTS

using the two corresponding influence lines for the cable pull at the same time a linear interpolation was possible which gave the correct value of the maximum bending moment at point m with an error of less than 1%.

207. A comparison of the influence lines for the bending moments of Fig. D9 with Fig. 2.38 of the Paper showed that the introduced length of load for the quarter point was not exact and it was to be expected that the curves of the maximum bending moments in Fig. 2.39 needed to be corrected.

Mr Blair Birdsall (Associate, Steinman, Boynton, Gronquist & London, Consulting Engineers, New York, formerly Chief Engineer of former Roebling Bridge Division, Colorado Fuel & Iron Corporation) wrote that the profession

throughout the world was indebted to the Authors for the full coverage of the history, design and construction of the Forth Road Bridge presented in the Paper—the more so because the bridge contained so many innovations in suspension bridge design. Not all engineers would agree at once with all the innovations. On the other hand, all engineers would certainly review their own practices in the light of these innovations and this was the great service provided by this generous contribution.

209. Since it had not appeared in the Paper, a partial list of the innovations might be in order.

- (a) Main tower sections connected with a small number of relatively large vertical bolts located inside the tower cells. No splice plates whatsoever for the horizontal tower joints.
- (b) Main towers designed without large flat steel slabs at bases and at tops of main legs.
- (c) Main cables of the 'flat-sided-hexagon' type spun in the saddles for the first time.
- (d) Saddles designed with permanent fillers to hold the cable 'strands' rigidly to a predetermined rectangular cross section through the saddle.
- (e) Prestressed, embedded anchorage elements with strand shoes designed to be adjusted on threaded rods rather than adjusting links and shims.
- (f) 'Waisted' high-strength cable band bolts (i.e., cable band bolts, with bodies reduced in diameter to produce body stresses equal to the stresses at the threaded portions), worked at stresses above the yield point.
- (g) One end of each of the two loops of suspender rope at each panel terminating in a common socket; the socket connected to the structure by vertical bolts through the top chord of the stiffening truss.
- (h) The suspended structure, in the main span, designed with orthotropic plate floor construction. In the side spans, the suspended structure was designed with a concrete floor slab. A by-product of this arrangement was that the tower top position for cable spinning was the same as for dead load.

210. Perhaps the Authors might clarify an apparent inconsistency. In § 2.7, the phrase '... showed the stiffened plate type to be economical ...' was used, but in § 2.8 the phrase '... it proved economical to use a reinforced concrete slab' seemed to contradict this.

211. In § 2.28 some reasons for not increasing wire diameter above the time-honoured standard of 0.196 in. were given. Mr Birdsall suggested a further reason. It was known that the normal diameters of the spinning wheels and wire deflecting sheaves produced theoretical extreme fibre bending stresses in the 0.196 in. wire which were above the proportional limit. These stresses were confined to very small portions of the wire cross section and the practice had stood the test of time. However, new unknowns would be introduced if larger wire were to be used on the same equipment and the use of larger equipment would be clumsy and expensive.

212. The remarks in § 2.30 were very relevant. The proper level for

allowable working stress in this wire was always a matter of discussion. Manufacturers who knew the toughness and stability of this type of wire always leant in the direction of higher allowable stresses, and over the years allowable stresses had been creeping up. The stress selected for the Forth Road Bridge design was on the high side of the stress currently used in the United States. With respect to the secondary stresses discussed in § 2.3, the reader was referred to some practical comments added by the writer in discussing Mr Wyatt's Paper.^{2,4} One might also keep in mind that if some of the wires were to be over-stressed by secondary stresses in the plastic range, this would do no more than produce some additional cold work, which might well improve the wire rather than the contrary. This reasoning would not be sound if the secondary stresses under discussion were stresses which would be generated by live load and temperature during the life of the bridge.

213. The principal secondary stresses to which the Authors referred must be those which might occur while the bridge was being constructed and the cable geometry was changing from the free cable to the dead load condition. Secondary stresses thereafter during the life of the bridge should be of a very minor order.

214. The following comments dealt with §§ 2.31 and 2.32. It had occurred to the writer that the Authors and the general reader might be interested in the relationship between the degree of compaction of the Forth Road Bridge cables and other cables on which data were available, and the reasons for this.

215. Degree of compaction was normally measured by the percentage of voids in the circle which circumscribed the cable. This was not a perfect yardstick, since the percentage of voids in a perfect cable (i.e., a cable containing the maximum number of wires which could theoretically be placed within the circumscribing circle) was not a constant. It varied from 9.3% for a cable of infinite diameter, along an irregular curve to 50% for a 2-wire cable and back to 0 for a single-wire cable. By evaluating points on this curve for cable diameters within the practical range, it was possible to obtain a common datum of reference which might be called 'percent voids for 100% compacting efficiency'. From this, the compacting efficiency of any cable might be obtained.

216. Previous to and including the George Washington Bridge it had been customary to arrange all cables 'horizontally'. The need for a change did not become apparent until the George Washington cables were compacted. These cables were more than 40% larger than any cables previously attempted. In addition to the relatively poor compacting efficiency, it was impossible to make these cables truly round. Changes were made in preparation for the Golden Gate cables. The hexagon was rotated to the vertical position, and varying numbers of wires were assigned to the different strands. The latter was the result of a careful study to determine the arrangement which would reduce the vertical wire motion during compacting to a minimum. All corner strands were made smaller than the others and certain other variations were made throughout the cable. The beneficial results might be clearly seen in Table D3. The hydraulic pressure used on the George Washington compactors was 6000 lb/sq. in. With the same compactors, a pressure of 4500 lb/sq. in. was adequate to obtain the results for the Golden Gate Bridge. The same philosophy was used on the first and second Tacoma bridges.

217. It was not always convenient to use varying strand sizes. These had

never yet been included in the original design, and it always caused complication to introduce changes in the anchorage arrangement. Thus, a modification of the above philosophy was made in approaching the construction of the Forth Road Bridge cables. A study was made to select a uniform number of wires for each of the central 19 strands, suitable for optimum compacting. The remaining wires were assigned to the outer strands so that no strand varied in size by more than 5% from the overall average. Thus, the corner strands were not made noticeably smaller. The basic theory in this case was that, since the outer strands were in direct contact with the compactor, the compactor might well force the wires to go where they should. It was considered worthwhile to try out this theory in an attempt to avoid the use of the additional refinements employed at Golden Gate and Tacoma. The results seemed to show that the lack of these refinements produced some deficiencies, although some of the relative reduction of compacting efficiency must be charged to the weather difficulties at the Forth site.

218. As a result of the above experience, the writer would be inclined to revert to the Golden Gate system, including the concepts regarding size of interior strands developed on the Forth project, but to make the analysis in the design stage, so that the anchorage arrangement could reflect the variations from the beginning. A review of Table D3 left little doubt as to the benefits of the 'vertical' arrangement as compared with the 'horizontal'.

219. It should be pointed out that care must be exercised in collecting data for such a study as this. The actual diameter for any given cable could not be considered reliable unless measurements were carefully made and at numerous points. Only then could valid averages be obtained.

220. Many designers seemed to consider these refinements irrelevant. No bridge had yet failed for lack of them, and one could not even point to significant differences in the time required between cable band bolt tightenings. The writer submitted, however, that every improvement in compacting efficiency was a step in the direction of true parallelism of wires—in other words, of providing a sound basis for increasing the amount of allowable unit stress possible.

TABLE D3

Bridge	Cable arrangement	Approx. cable dia. (in.)	At cable bands		Voids (%) between cable bands	
			Voids (%)	Compacting efficiency (%)	Before wrapping	After wrapping
George Washington	Horiz.	36.0	21.2	87.4	22.7	N.A.
Golden Gate . . .	Vert.	35.8	17.4	92.2	19.4	18.7
1st Tacoma . . .	Vert.	17.1	17.0	94.0	20.2	18.3
2nd Tacoma . . .	Vert.	20.1	17.2	93.5	19.8	18.8
Forth Road . . .	Vert.	23.5	18.9	91.1	21.7	20.4
Tagus	Horiz.	23.3	20.8	89.0	22.8	N.A.

(N.A. = Not Available)

In the Table, the words 'horizontal' and 'vertical' under column heading 'cable arrangement' refer to hexagon with flat top and hexagon with flat sides, respectively.

221. In connexion with § 2.33 the writer firmly agreed with the Authors' reservations regarding the time-honoured practice of delayed wrapping. This theory had received a staggering blow when, on the George Washington Bridge, the diagonals and lower chords of the stiffening trusses and the complete lower deck were installed some 30 years after the cables were wrapped, with no apparent ill effects on the wrapping. The writer was inclined to permit wrapping at any time after construction had reached such a stage that no further heavy angular changes were likely to take place in the cables, or after dead load cable stress had risen to within 14 000 lb/sq. in. of its final value, whichever was later.

222. At this point it might be interesting to note the quantitative effect of differences in wrapping tension. On the first Tacoma Bridge, two types of wrapper were used. One applied wire at a tension of approximately 300 lb, the other, 600 lb. In the one case, the percentage of voids under the wrapping was 19.0%, in the other case 18.3%.

223. The function of the permanent steel spacers mentioned in § 2.34 was not adequately described by the phrase '... to facilitate spinning...'. Actually, these spacers, together with the rectangular steps of the saddle groove, provided a set of conditions which made it possible, for the first time, to spin cables in the saddles under completely controlled conditions. These devices provided a predetermined, rigid trough for the placement of each strand, thereby minimizing disturbance of wires and galvanizing during longitudinal motion of strand for adjustment, and removing any possibility of slump of wires after strand adjustment.

224. At a point such as the side tower saddles (more recognizable to American readers by the phrase 'cable bent saddles') (see §§ 2.35 and 2.36), the writer normally resisted dependence on friction to overcome the imbalance in tension, preferring the more positive solution of adding one or two strands in the backstay, dead-ended at the saddle. However, this was a special case. A by-product of the saddle design was the opportunity to add four more vertical friction surfaces by using two solid full-height spacers. With this, all parties agreed that it would be safe to depend on friction. The writer requested that, in their reply, the Authors might confirm or revise the statement in the text regarding '... shallower castellated plates...'. The writer did not recall such plates in these saddles, although the normal spacers certainly did increase the effective friction along the bottom of the cable.

225. Mr Birdsall had difficulty in following the reasoning of § 2.35 which explained the decision to use a rocker arm. In America, at least, it was not customary to mount such saddles on rollers. It was normal practice to mount them on rocker arms or tall flexible towers. However, the rocker arm on the Forth Road Bridge was shorter than in the customary arrangement—with one notable exception. The corresponding rocker arm at the West end of the San Francisco-Oakland Bay Bridge was of a length approximately equivalent to that of the Forth. Mr Birdsall suggested that the penultimate sentence of § 2.35 would be clearer if it were worded:

'The designers chose a rocker bearing rather than a roller bearing because they preferred a predictable, small longitudinal force component to an uncertain longitudinal friction, which would be likely to increase with time'.

226. The final sentence of the paragraph seemed to add more confusion than clarification. It implied that a net horizontal component was inherent

in the cable forces. Under dead load and normal temperature there was no net horizontal component. Under any other condition, involving motion of the saddle, a small horizontal component existed only because of the presence of the rocker arm, and its magnitude was inversely proportional to the length of the rocker arm.

227. The relative shortness of the rocker arm required some special consideration during erection, to minimize the secondary stresses discussed in § 2.30. Half of the free-cable-to-dead-load motion was provided by sliding the rocker base, instead of taking it all in rocker rotation, as was customary. The devices employed (§ 2.37) to facilitate this sliding worked very well. The splay casting (§ 2.38) was normally the place where rollers were replaced with a rocker arm. The sole criterion appeared to be economy. In spite of the slight deformation of the upper strands, the general appearance of the cable in the splay casting was excellent, fully justifying the expense of the special fillers used to produce the result.

228. The writer recognized the technical benefits of waisting the cable band bolts (§ 2.40), and of stressing the bolts into the plastic range, thereby computing the factor of safety as a function of stretch rather than stress. This certainly produced high efficiency of bolts and cable bands, and maximum margin against relaxation. However, in America, at least, it was not a foregone conclusion that these practices would produce overall economy. Much, if not all, of the economy of material would be lost in the extra cost of waisting the bolts and in the cost of erection. When stressing bolts against a yielding medium such as a cable, it must be quite difficult to maintain control when several adjacent bolts were stressed beyond the elastic range. Any economic study of these points should include maintenance costs. If the Forth practice removed the need for even one operation of re-tightening cable band bolts, this would be a large factor in its favour. In America, it would almost certainly prove economical to make the bolts a little longer. The resulting reduction of erection cost would more than pay for the additional material.

229. One important feature of the cable bands was not mentioned in the text; that there was no annular recess at the end of the band for wrapping and caulking. Wrapping wire was started and finished as neatly as possible against the edge of the band, and the availability of the modern polycast sealants made it possible to produce a protective fillet between the edge of the band and undisturbed turns of wrapping wire.

230. The suitability of the double socket for suspender ropes (§ 2.42) also depended on local economy. Its use in America might well be relatively expensive. The method of connexion to the stiffening truss was new and original. It would be interesting to see if time proved it to be better. There was also the question of the relative cost of making the connexion in the field.

231. The full significance of the latter part of § 2.53 might not be self-evident to some readers without further explanation. Exact determination of density of concrete was required only because of the special features of this project. On suspension bridges with similar suspended structures throughout all spans, the exact density of concrete was of minor concern. Small uniform deviations from computed dead weight had little effect on the finished structure. In this case, however, where a concrete slab was used only in the side spans, small differences could result in very noticeable changes in geometry and could leave the tower tops in a displaced position at dead load.

232. The use of an $8\frac{3}{4}$ in. slab when a 7 in. slab would have been structurally sufficient, was open to question if total cost of materials was thereby increased. It was done for the purpose of having the towers vertical at the free cable position. The writer did not recall the effect on verticality of using a 7 in. slab. If it was small, it would have cost very little to produce the required motion by proper design of footbridge and storm system. It was more likely that all concerned, including the writer, were intrigued with the idea of spinning cables with vertical towers, as long as it could be accomplished at small cost. In any event, one should guard against the suggestion that it was generally good practice to add dead weight to the finished structure to facilitate erection.

233. Whether by example or coincidence, the anchorages on the Tancarville suspension bridge in France were the first of a series of prestressed anchorages. Thereafter came the Forth Road and Severn Bridges in England, and two in America. The writer wished to offer a word of caution lest there be more fashion than foundation in the practice. The one valid criterion should be economy. Where prestressing could reduce the bulk of excavation and concrete which was not otherwise needed its use was probably valid. Otherwise, the economy was doubtful. One argument frequently heard was that prestressing removed uncertainties regarding the bond between concrete and normal embedded steel. This argument was eloquently answered by the number of successful bridges with conventionally embedded steel. If the writer had overlooked some inherent virtue in the prestressing of anchorages, he invited rebuttal.

234. There was something to be said in favour of relatively long adjusting links whether the principal steel was prestressed or not. The short adjusting linkage on the Forth Road Bridge made it necessary for the spinning wheel, at the far end of its journey, always to come to rest above all the anchor heads. This, in turn, made it necessary to pass the wire loops down from top to bottom for the lower strands.

235. With respect to § 2.106, readers were referred to the writer's paper on the tower cantilever problem for a direct solution.^{D22} The method of converging approximations might be more suitable for modern computers, but the paper developed certain basic concepts which were helpful in visualizing tower action and in selecting methods of solution. For instance, the paper disclosed, and explained the reason for, the phenomenon that the curve representing H per unit of deflexion of tower top plotted against V was a straight line (for practical purposes), whereon one intercept was the Euler critical value for V and the other the value of H for $V=0$.

236. The difference between American and British practice was no more evident than in the case of the tower erection derrick (§ 4.14). The 32 ton derrick hoist was geared down to make it possible to lift and manoeuvre the heavy tower sections with a single part of heavy wire rope. In American practice, the hoist would have been much simpler, with a higher travelling speed and a lower line pull, and the lifting tackle would have consisted of many parts of smaller wire rope. There was no question here of right or wrong method. The same lift was made in the same time. In the American practice, less money was put into the hoist and more into the tackle. Perhaps each one was right in its own national context of economy and custom, but it might be useful for each to study the other's practice to see if some good ideas had been overlooked.

237. Recognition was due to Cable Covers Ltd for their important contribution to the cable splicing. Prior to the Forth Road Bridge, no material improvements in the wire splice (§ 4.36) had been made since a swaged splice was developed for the George Washington Bridge. When the basic problem and criteria were presented to Cable Covers Ltd, their new approach to the problem produced a better swaged splice, which did the same job with much lighter pressing equipment. For the first time it became possible to make a 100% wire splice anywhere along the footbridge.

238. The purposes of the counterweight tower as described in § 4.47 needed some clarification. The purpose of ensuring equal tension would be satisfied, for uniform wire speed, by a floating sheave with little or no vertical travel. A tall tower, providing considerable vertical travel, was necessary to control changes in tramway speed, including starting and stopping. The tramway was operated by one drive, and the wire unreeled by another. The two operators would often be far removed from each other, and out of sight. The tramway operator responded to the needs of the spinning operation: the unreeling machine operator must respond to the movements of the tramway. When all was at rest, the counterweight sheave was normally at or near the bottom of its travel. When the unreeling operator saw the counterweight sheave beginning to rise, this was his signal that the tramway had started to move. He immediately put the unreeler into motion, allowing the counterweight to rise to a predetermined position and holding it steady. The counterweight would respond to any change in tramway speed, and the unreeling operator responded accordingly. Failure to respond in one direction would spill loose coils of wire over the anchorage. Failure to respond in the other direction would send the counterweight sheave smashing against the top of its travel.

239. These arrangements lent themselves perfectly to automation, and the counterweight tower was completely automated for the first time on the Forth Road Bridge. Thus, the purpose as stated, '... to control the speed of the unreeling machines' was not valid before the Forth Road Bridge and did not clearly express its historic purpose. The automated system worked well, but opinion was not unanimous in favour of its continued use. It was relatively expensive for a temporary construction job, and it was doubtful that its use reduced the required number of operators by even one man. Personnel still had to be on hand to take care of unforeseen conditions and emergencies.

240. The cable spinning required more time than normal. It was seldom possible to reach the designed capacity of the equipment for prolonged periods. This was due in part to the fact that all operations were new to all personnel, but it was due mostly to wind conditions. The miracle was that the work was accomplished at all. No one had ever been faced with the problem of spinning wire under such conditions. All hands from the Agent ('Manager of Erection' to American readers) to the water-boy were to be congratulated for their perseverance.

241. The writer was glad of the opportunity given by this project to discuss challenging new ideas and fresh points of view. In view of the overall excellence and advancement represented by the finished product, general criticism would seem to be out of order, no matter how constructive in its intent. However, the writer wished to offer one general comment, in the most friendly spirit of co-operation. The outside observer could not escape the impression that the progress of the work would have benefited by more

liaison between Designer and Contractor in a common search for the most efficient structure consistent with rapid and minimum-cost fabrication and erection.

Authors' replies

Mr J. Kenneth Anderson in reply thanked those who had taken part in the discussion for their contributions to the Paper and commented on a number of matters.

243. Besides the tunnel anchorage scheme which was adopted, he had pursued in some detail a scheme for mass anchorages which the late Sir David Anderson had planned and which were in some respects similar to those at the Severn Bridge. The costs appeared to be similar, but Mr Anderson considered that at the Forth Bridge site where there was good rock at each shore the right decision was made and from an aesthetic viewpoint side towers and tunnel anchorages were preferable to mass anchorages.

244. With reference to cable design he felt that in future consideration should be given to the use of prefabricated ropes such as were used in the 1100 ft span Tamar Suspension Bridge.^{D23} Here $2\frac{3}{8}$ in. diameter locked coil wire ropes were used in each cable. This offered a cable with better resistance to corrosion than ordinary wire ropes, and from investigations made it seemed that such ropes might well be capable of being prefabricated and delivered to site for such spans as the Forth Road Bridge. The saving in cost of plant for spinning and the time of erection would be very considerable.

245. In regard to the erection of the stiffening truss he was interested in Mr Hyatt's comments. At the Tamar Bridge, which was roughly a $\frac{1}{3}$ scale model of the Forth Road Bridge, large sections of the stiffening truss were floated out on pontoons and erection commenced at mid span. Erection was also put in hand from the side towers. He considered that this was a speedier and more economical method of erection. This view would appear to be confirmed at the Severn crossing where in spite of much more difficult river conditions, erection in large sections from mid span was adopted.

246. The figure of 20 knots had been quoted in § 1.22 which related to wind speed. One of the main purposes of the experimental work on the old railway bridge over 18 months had been to find out the general consistency of the wind. Mr Brown's remarks were quite correct. At wind speeds of about 35 knots or less many instances were recorded of inclined winds, but generally these appeared to be isolated gusts and did not occur on all three instruments simultaneously. On a few occasions, however, simultaneously inclined gusts were recorded on two of the instruments. The maximum inclination recorded on two instruments simultaneously was 11 degrees at a wind speed of 22 knots (for about 1 min). The maximum velocity at which two inclined gusts were recorded simultaneously was about 26 knots and in this case the average inclination was about 6 degrees.

247. He was grateful for the reference regarding the cost/sq. ft of the individual bridges. He too felt that, although an interesting and useful yardstick for other bridges, it could be misleading. There could be two bridges performing virtually the same function but appearing to have different costs.

248. The construction of wide reserves on the approach roads and bridges was, he considered, a wise provision in view of the ever increasing traffic and the possibility of eventually requiring 3-lane dual carriageways.

Sir Gilbert Roberts, continuing the reply, said that he much appreciated Professor Pugsley's kind remarks on the spirit of co-operation and on the design of the towers. It was doubtful whether these could have been made much lighter and therefore more economical, except as a result of the reconsideration of the load factors and therefore the working stresses. The main stress in the tower was that due to the dead weight of the bridge, which could be accurately calculated and could never be exceeded. For this form of loading it might be reasonable to use a lower load factor than the 1.7 adopted for the load combination including dead load, live load and bending of the tower due to cable movement. For loading combinations including wind, the load factor was 1.36 but such was the preponderance of dead-load stress in the towers that this load factor would be reduced very little if allowance were made for increased wind stresses due to dynamic effects. The assumed out-of-plumb dimension at the tower top (taken as 1 ft) could be regarded as analogous to the eccentricity assumed in the empirical strut formulae to which Professor Pugsley referred. It was intended as a more-than-generous allowance to cover a possible difference in position between the free-standing tower and the tower in the completed bridge, with the whole structure at a uniform temperature of 68°F. The actual deviation was certainly much less than this and probably less than the 5 in. quoted by Mr Brown. The difficulty with attempted measurements at site was in the estimation of the cable temperature, which could only be measured accurately by a large number of thermocouples embedded in the cable. It was quite likely that the temperature of the steelwork was never truly uniform and the most that could be said about the tower position was that it was probably within a few inches of its theoretical position at uniform mean temperature. A part of this assumed eccentricity might be taken to cover the effects of non-uniformity of temperature throughout the finished cables.

250. Mr Pain commented in § 12 that the Papers did not give a true picture of the great amount of investigation, calculation and experiment involved in producing a scheme of erection for a structure of this magnitude. The reason, no doubt, was that two of the Authors, the Resident Engineer and the Agent, were concerned with the work on the site and not with the planning and design of erection equipment previously done in the office. This might also account for the emphasis given to minor troubles which caused 'difficulty and delay' such as the short grip bolts referred to in § 4.87. The first batch of these were machined down by Arrols in a few days while the erectors proceeded with longer bolts delivered for another part of the job. Mr Pain was responsible for details of the deck erection scheme and he followed the excellent procedure of putting it down in the form of a comprehensive erection memorandum, amply illustrated by drawings of the temporary plant and diagrams of the erection sequence, the loads, and the deflexions, etc. to be expected. This memorandum was prepared in consultation with the Engineers by whom, in accordance with the Contract, it had to be approved. Once the scheme described in the erection memorandum had been approved it was clearly understood that it was not to be departed from in any way by the people at the site. If they could see improvements or wished to take a short cut they must obtain approval and revised instructions before putting their ideas into effect. In Sir Gilbert's view this method of control was the only way to avoid the possibility of disasters such as had occurred in the past from the Agent making

improvisations in the erection scheme without full knowledge of all the considerations involved.

251. The incident described in § 4.108 was an example of what might happen when a change was made in the agreed erection procedure, even when the people concerned believed all the relevant factors had been taken into account. The original intention was to leave off the side footway and cycle track panels as well as two of the roadway deck panels until the stiffening trusses and lateral bracing were complete and the deck structure had its full torsional rigidity. However, the people at the site had the idea of erecting some of the footway deck panels, working backwards from the centre of the span, with the main erection derricks. The Contractors consulted Mr Scruton who had made a report on the aerodynamic behaviour during erection and he appeared to have given them satisfactory assurances, apparently under the impression that the stiffening trusses would be fully bolted at this stage. In fact the trusses were bolted only about as far as the quarter point working out from the tower, and when the footway panels had been erected to about the same point working from the centre, aerodynamic oscillation occurred with an amplitude of 4–6 in. This would not have been serious if the truss had been complete but at the junction between the bolted and unbolted parts, where vertical shear could be taken only by the unbraced top chord member, the vertical movements of the centre section evidently produced high alternating bending stresses in the 30 ft length of top chord projecting from the last rigid point. No structural member could withstand such treatment and the chord broke in the manner that one would expect. Fortunately the top flange plate remained intact and it was possible to make a repair with bolted cover plates, without removing the chord from the truss. It was subsequently disclosed that Mr Scruton would not have given his clearance if he had been aware of the true properties of the structure as it existed at that stage. This incident would not have occurred and many of the Contractor's troubles would have been avoided if the truss joints had been bolted up as soon as the curvature of the deck permitted this to be done. The erection memorandum warned them to watch the joints closely and immediately bolt up those that came fair. It also pointed out that it should be possible to drift up alternate joints at an earlier stage. If the Contractors had stopped erection at a certain stage and concentrated on bolting-up, they would not subsequently have had to force the ends of the chords apart to bring the holes into line. It was a common failing of erectors to continue putting up steelwork without making good the connexions, and young engineers had been made aware of this tendency and taught that spectacular erection progress might be nullified by the extra time required to make good the joints when under stress. In cantilever erection particularly (the first Quebec bridge was an example) the tendency to leave joints unfinished might have disastrous results.

252. The porosity (or more correctly blowholes) in the butt welds in the deck plate were caused not so much by the zinc as by the dirt and moisture lodged between the backing strip and the underside of the plate which it was impossible to remove on the site. The type of joint was really unsuitable for use except in a welding shop where the parts could be kept perfectly clean and dry and Sir Gilbert blamed himself for suggesting it. A temporary backing strip was preferable in every way except that access to the underside was

necessary for fixing and removing this. A single-run weld would then have been quite feasible.

253. Mr Hyatt referred to the welded connexions for the crash barriers and grillages and implied that these would have been easier to erect if bolted on. However, this would be true only if the holes for the bolts matched correctly and in this sort of fabrication, with weld contractions and other errors to be allowed for, it was very doubtful whether this could have been achieved with holes drilled in the shop. More time could be wasted in trying to make bolted connexions fit than in making welded ones.

254. Mr Wood added to the impressive list of 'contractor's difficulties' given by Messrs Pain, Hyatt and Knox that of providing temporary staging for access to the work. The old Forth Railway Bridge required even more staging but Sir Gilbert had never seen this referred to as one of Arrol's difficulties. Possibly the Contractors did not use the best form of staging or did not have a sufficient amount of it available. Giving the Contractors the use of the permanent maintenance gantries was no solution. That had been done at the Severn Bridge but the contractors there preferred to use their own temporary devices because the permanent gantries were too slow in movement and inadequate in coverage.

255. Mr Knox in his complaints about the double sockets on the suspender rope dwelt on the added troubles of the rope maker. Since the rope maker raised no objection to the double socketting procedure and did not claim any extra for doing it, this could largely be discounted. Once the hanger had been rolled on the drum and delivered to the site there could not really have been much difference in erection cost between a two-part hanger and a single bond of larger rope which would have a greater total weight. The delivered cost of the two-part rope and double socket was definitely less than for an equivalent single-part hanger of larger diameter rope.

256. Mr Upstone seemed to doubt whether the use of a concrete deck on the side spans produced any real reduction in overall cost. There was no doubt that this was so, however, in this particular case, because the length of the side spans was such that the cable could support additional dead load (with slightly increased sag) without any addition to its sectional area. The comparative costs of the concrete and steel decks were taken from the records of actual cost. It was difficult to believe that the cost of steelwork erection could have been increased so much as to absorb the estimated saving of £120 000. A better scheme of erection (apart from floating out large sections of the bridge) might have been to complete the truss sections before erecting any of the deck. It should be noted that the computer results did not in fact influence the erection procedure but gave in detail the position of the structure at any stage. The tower top movements themselves were easily calculable without the use of a computer.

257. The tendency of a free-standing tower to oscillate in the wind like the other examples mentioned by Dr Dutt was always known as a possibility. The natural frequency was calculable but it was not possible to estimate the amplitude that might be attained before the energy input would be dissipated by structural damping. Nothing of the sort had been experienced on the American towers. Sir Gilbert was confident, however, that if oscillation occurred and built up sufficiently to overstrain the horizontal joints, the bolts would stretch, and by allowing the joints to open provide a slight degree of

damping. There was no danger involved here because the waisted bolts had to be stretched several inches before failure. This was in fact what occurred, but not until the amplitude had built up to an amount that caused serious discomfort to men working at the top. If the amplitude had been only a foot or two no one would have complained, but as it was some further damping had to be provided.

258. The answer to Dr Dutt's last query was that the lowest point of the exposed steelwork of the tower was 18 ft above high water level, which was considered high enough to keep it out of the range of salt spray.

259. Mr Kinsella commented on the re-tightening of the cable band bolts on several occasions. This was unnecessary, since a much reduced load in the bolts would have been sufficient to keep the bands in place. The high tensile bolts used had made it unnecessary to follow American practice in this respect. The final tightening of the bolts beyond the elastic limit provided an ample margin against future relaxation. In § 142 he referred to the trouble in making the centre joints in the bottom laterals. This was due to the Contractor's oversight. If one end of the raking members from the cross girder to the truss joint had been made with a few black bolts instead of permanently bolted when first erected, the bottom laterals could easily have been completed much sooner than they were.

260. Mr Kerensky and Dr Richmond referred to the model tests on the box members which failed at somewhat lower stresses than were expected. Unfortunately no measurements were made on these models to evaluate the residual welding stress but from the results it appeared to have been 4 or 5 tons/sq. in. instead of 2 or 3 tons/sq. in. as might be expected in the full-size members. The reasons for this were not altogether clear. The sectional area of the models was about 25% less for the same welding and there might have been some gas cutting and preheating involved, which would increase the residual stress. An unsatisfactory feature of welded models generally was that it seemed impossible to keep the secondary welding effects even approximately to scale. In the model columns tested at Cambridge and mentioned by Dr Richmond residual stresses were found exceeding 6 tons/sq. in., and tests of 10 in. square box columns at the Lehigh University showed compressive residual stresses of 12.5 tons (30 kips)/sq. in. The shortening of a plated member due to welding was a measure of the compressive residual stress in the plates, except in the immediate vicinity of the welds, where the residual stress was tensile. Routine measurements taken for this purpose during fabrication showed that the contraction rarely exceeded $\frac{1}{16}$ in. in 20 ft, corresponding to about 3 tons/sq. in. for the type of welding considered here. On the heavier items of steelwork it might be considerably less, but in general an allowance of $2\frac{1}{2}$ tons/sq. in. would cover most items of bridgework.

261. In box columns the effect of residual stresses due to welding was to reduce the average stress at which local buckling occurred by the amount of the residual stress, or very nearly so. The weakening effect was confined, however, to those areas liable to fail by elastic buckling before general yielding of the section took place, and the b/t ratio could be adjusted to ensure that local buckling would not occur in advance of failure of the column as a whole. Sir Gilbert suggested that for high tensile steel a b/t ratio of 32 was satisfactory for short compression members such as the chords of the stiffening truss;

proportionately higher values would be appropriate for higher values of l/r and therefore lower working stress.

262. Apart from their tendency to precipitate local buckling of unsupported areas of the plates, residual stresses in welded square box sections probably had no effect on the strength of a strut forming part of a bridge truss. Arguments based on the behaviour of pin-ended struts could be misleading, since the struts in a truss were fixed-ended (or partially so) and would tend to fail by buckling on their full length with plastic moments at their ends resisting the buckling. Perhaps it would be more realistic to test box columns in the fixed-ended rather than the pin-ended condition. Residual stresses were always present to some extent in any form of construction and provided these were not excessive he would regard them as covered by the so-called factor of safety. Lastly, he could not accept Dr Richmond's assumption that the stress/strain curve for high tensile steel plate was not linear. Compression tests made on specimens cut from high tensile steel plate of various thicknesses showed the stress/strain curve to be perfectly straight well above the lower yield point. Tensile tests, if made with sufficient care in keeping the load truly axial, showed the same result.

263. Sir Gilbert was grateful to Mr Blair Birdsall for his lucid comments on the cable-spinning operation and its influence on certain aspects of the design. In reply to his query in §§ 210 and 232 on the concrete deck, the stiffened steel deck was economical only where its lighter weight produced a consequential saving in the cable stress, and this was the case in the main span; in the sidespans there was no consequential saving and therefore the concrete deck was cheaper. The thickening of the concrete slab on the sidespans, referred to in § 232, was economical because it allowed some secondary steel cross beams to be eliminated.

264. For the anchorage steelwork Mr Birdsall must surely admit that high tensile wire cables were more economical for taking tension over a considerable length than the standard American arrangement of eye-bars and pins, which went out of fashion in this country in the last century. If wire cables were used it was a simple matter to give them an initial tension. The Forth Bridge arrangement enabled the anchorage concrete to be completed before cable spinning started.

265. As regards the waisted cable band bolts (§ 228), Sir Gilbert assured Mr Birdsall that the reduction in the number of bolts and length of cable band more than offset the slightly increased cost per bolt. There was no reason to suppose that the cost of erecting them was increased and their final tightening to the ultimate strength of the bolt was expected to avoid any necessity for retightening in future. There would be no purpose in making the bolts longer than required to give the nuts their initial grip.

266. In § 224 Mr Birdsall questioned the 'shallower castellated plates' mentioned in § 2.36. These were in fact welded into the side tower saddle and continued upwards with the spacer plates as shown in Fig. 2.11. They were considered useful for increasing the friction on the cable.

267. Referring to § 236 Sir Gilbert did not see how the design of an erection derrick with small diameter rope could possibly be justified if it was at all practicable to use a single fall. The extra pulley blocks used (according to Mr Birdsall) in American practice provided a means of increasing the purchase which was very inefficient when compared with an extra reduction in the gear

train. The loss of efficiency meant that the total weight of rope was increased. When the crane was lifting from the ground most of this extra weight, and the weight of the block, were hanging from the jib-head and therefore reduced the working load. For an 8:1 reduction the efficiency of the rope blocks would be no more than 50% compared with, say, 90% for the other arrangement, so that the extra weight of rope required was quite considerable. It was clearly desirable to have the reduction done by gearing on the crane itself where it could be enclosed and properly lubricated, instead of by ropes and pulleys exposed to the elements and difficult of access. He believed that British derrick crane makers had abandoned the pulley block thirty years previously and since then the standard derrick, up to 20 tons at least, had had only a single fall.

268. Mr Birdsall rightly stressed the importance of studying each other's practice. For Sir Gilbert's part he had done his best to absorb all the background information on cable spinning and associated problems so generously provided at his many discussions with Mr Birdsall, to whom he was extremely grateful.

Dr J. A. K. Hamilton also thanked Sir Alfred Pugsley for drawing attention to the co-operative spirit shown by the many teams engaged upon the bridge. He wished to endorse that remark and say that with many years of bridge building experience in the field he could not recall a more united and pleasant collection of people, all imbued with the single aim of creating this large bridge in the best possible way.

270. Sir Alfred referred to the design of the towers and queried if metal could have been saved by reducing the diaphragms in number or weight. His remarks had already borne fruit, as the towers of the Severn Bridge had considerably fewer and lighter diaphragms than those described in the Paper.

271. Mr Pain, Mr Knox, Mr Kerensky, Mr Blair Birdsall and others had discussed cable design in their contributions. The most desirable end result was probably a tightly packed parallel wire cable, but in localities comparable to the Forth Estuary, and subject to almost continuous wind, the method of construction merited great consideration.

272. Other approaches to this problem were:

- (a) hauling prefabricated coiled strands across the gap;
- (b) the use of thicker wire;
- (c) prefabricated parallel wire strands.

273. Prefabricated coiled strands had been used at Tancarville and Tamar, but this type was less efficient and gave a less compact cable. Flat ribbon type strands, as used on the Otto Beit Bridge, would probably have been difficult to handle in strong winds. The use of thicker wire, always provided that it could be produced with the required tensile strength, would provide a cheaper form of construction by reducing the number of passes on the spinning wheel. Mr Birdsall stated that the wire diameter should not be increased because of bending stresses in the sheaves during spinning and also stated that stressing wires during erection beyond their elastic limit probably had a beneficial effect due to cold working. It would seem, however, that the limiting factor was the handling of the wire at the strand shoes.

274. The use of preformed strands of parallel wire construction, the manufacture of which was being investigated, could have many advantages for future bridges.

275. Mr Goldstein referred to the target contract arrangements. The agreed target price less £100 000 (minimum target) was beaten and the Contractor received 50% of this saving in addition to the fixed profit fee. The Contract was so arranged that the rate of reward for saving was greater than the penalty for excess cost. It would certainly have been possible to limit 'profit' in the same way as 'loss' was limited; but a substantial inducement to save was considered preferable in this case. Plant, other than that supplied by the Client, was paid for at rates of depreciation stipulated in the Contract.

276. Messrs Hyatt and Wood discussed the erection of the suspended structure, and while it might appear now that a floating-out scheme would have been economical in time and money, this view, arrived at after experience gained on the Severn, could only be surmise. Certainly it was not appreciated in the early discussions that wind could interfere so much with steelwork erection, and how desirable it was that lifts should be kept to a minimum by arranging for erection in large pieces, requiring few lifts, rather than piecemeal. Access for floating plant below the side spans was far from ideal and the removal of the large rock (Long Craig Island) in the north span would have been very costly. Some time could have been saved and some delays averted had it been possible to treat the fabrication of 99 almost identical units as a production line job rather than a piecemeal job to be done when the weather permitted.

277. Much experimental work was done at site before it was agreed that the battledeck should be welded with double runs. The backing strip appeared essential for sound workmanship, but welding would have proceeded faster if more care had been taken to avoid damage to these strips during transit and erection.

278. Mr Wood referred to the difficulty of providing sufficient staging over the whole length of the stiffening trusses to enable work to be carried out efficiently, and to the lack of any permanent travelling carriages and rails carrying them. Three permanent travelling carriages were now to be provided, but these would be supported by bogies running on the tops of the lower main chords. It was agreed that it would have been very helpful if carriages and rails had been included in the first instance. They would have saved time, especially during the later stages of truss erection.

279. It would appear that the selection of Drem Airport had considerable advantages as a workshop for protective treatment and storage of steelwork. It was doubtful if it would have been possible to recruit and train labour for this work in a city or town with any greater facility than at Drem, where there appeared to be no shortage of willing labour. The site at Drem involved no great detour for the steelwork fabricated at Middlesbrough and Darlington, but did involve extra haulage for the items made in Glasgow. These latter consisted mainly of tower sections erected immediately they arrived on site and needing no storage space there. The storage and transport arrangements were excellent and reflected great credit on those responsible for them.

280. Mr Knox discussed the difficulties in manufacture and erection of the pairs of suspender ropes in double sockets and gave cogent reasons for his remarks. There was little doubt that his recommendation in § 51 was correct,

and single sockets should be adopted in future work of this type. A further advantage was that bases of single sockets could be machined to suit the slope of the top chord at the point of attachment and their connexion made to the chord without the use of separate taper packings.

281. Mr Berridge observed that the web members of the stiffening trusses had to be fitted between gusset plates already welded to and forming part of the box section chords and made the point that this design involved loss of efficiency in the joints as the grip bolts necessarily had to bend the gussets inwards to close erection clearances. This difficulty was appreciated and the end batten plates of the double channel web members were bolted. Connexions were not tightened up until after the web members had been bolted to the chord gussets between which they were easily inserted.

282. Mr Kirk would have liked access shafts left in the main body of the concrete in the anchorages for inspection, ventilation and grouting purposes. This question was discussed fully during the construction of the anchorages and abandoned as it appeared to increase the difficulties of placing the concrete, without any obvious advantage. He made the suggestion that one central laboratory for controlling and testing the concrete would have been of value and Dr Hamilton agreed with him, but facilities for aggregate testing and concrete cube making would still have been necessary at each mixing plant.

283. Mr Scruton's contribution gave only a slight idea of the work that he and his colleagues had contributed towards solving the varied and difficult aerodynamic problems of the Forth and Severn Bridges. He stated that there was no way of calculating the damping factor of a structure beforehand, and that the oscillation of the tower in its free standing position could only be assessed from the performance of the structure itself. Dr Hamilton agreed absolutely with his suggestion that provision should have been made for adding damping to the towers during their construction, if only for everyone's peace of mind, as the performance of the nearly completed tower with a ring girder and crane weighing about 230 tons at the top left everything to be desired, until a damping system was installed.

284. Mr Stephenson suggested that actual live load could be easily determined by various measurements of stresses or strains, on recording instruments. The difficulty was that measurements would have to be related to the actual weights of traffic on the bridge at the same instant of time. The actual live loads normally on the bridge would be small compared with the design live loads, so that very small strains would be measured. It was doubtful if any additional information would be gained in this way.

285. Mr Kinsella referred to the 'labour efficiency' of cable spinning. The figures in his table clearly demonstrated the effect of experience and weather conditions on this work. Monthly records of cable spinning were kept and showed that while in December 1961 this figure was 24%, in June and July 1962 it had risen to 86%. On several occasions over 550 miles of wire were spun in one day.

286. The difficulty of bolting up the bottom laterals was probably due to the weight of deck on the cross girders tending to push the lower chords apart, and had the bolts in the lower extremities of the raking members been slacked off, the chords could have been easily pulled together. The Contractors preferred to use jacks and drift the joints as this saved staging. Referring to

the positions of the main and side tower saddles, these were for standard temperatures of 68°, not 60°.

287. Mr Little's remarks on cost/sq. ft of main span and viaduct were gratifying and there was no doubt that the design procedures materially contributed to this result.

288. Dr Hamilton's recollections concerning the suggested use of naval divers in the north pier cofferdam were somewhat different from those expressed by Mr Little. Divers were not required to inspect the whole area inside the cofferdam, but only for a strip 10 ft wide adjacent to the steel piling. The dam was dewatered after the lowest waling, of tremie concrete, had been poured. The original scheme for a pier base of intrusion-grouted concrete had been abandoned before any divers were required. Due to the excellent work of the civilian divers, the tremie concrete was found to be perfectly sound and of high quality. Had Mr Little seen it while the dam was empty, it was possible that some of his opinions might have been modified.

289. Mr Wright referred to a difference of 0.1 ft in the two measurements across the Forth. This was rather less than 1 in 60 000 and had no consequences at all, except that it deserved an expression of thanks to the surveyors concerned! No further check was carried out on the completed bridge as the constantly changing deck profile made this almost impossible. Aerial photographs were used only to record progress and not for any other purposes.

290. Mr Lindsey discussed the effects of blasting. Apart from superficial damage to nearby buildings from flying debris in the early stages there were no significant claims for damage due to blasting. The Engineer's Office on the north shore, a private house erected in 1901 and about 150 ft west of the northern anchorage tunnels, suffered no damage, but a dwelling house a little further away on the east side, and much older, had slight damage to interior lath and plaster walls and some of the outside harling was loosened and required replacing.

291. The demolition of the masonry bridge at Cramond, with half the width of the new bridge built alongside it, was an excellent example of a carefully designed scheme. The new work, as well as a row of bungalows at one end and a public house at the other, sustained no damage at all from the explosion.

292. Mr Gowing referred to the design of the south pier cofferdam and caissons. He had given some very useful additional information about them. The form of the cofferdam and the waling structure was evolved as a result of the unusual construction techniques required and represented a most successful solution. The strain readings that were taken indicated that the original design assumptions were quite justified. This design approach had produced very satisfactory structures in two river estuaries, each of which was subject to extremes of wind and tide.

293. The 2 in. clearance to the caisson walls, originally adopted to allow for inaccuracies in construction, had the additional merit that it allowed for any expansile movements of the boulder clay. The proposal for filling the cavity with bentonite originated with Mr Bartlett, who wished to keep the cavity clear of debris so that the subsequent filling with cement grout would be more effective. The additional benefits of watertightness and reduced skin friction became apparent during the sinking process.

294. Mr Bishop requested further information regarding the intrusion-

grouted concrete waling of the south pier cofferdam. Whinstone aggregate, $1\frac{1}{2}$ in.–3 in., was placed in the 8 ft 6 in. wide annular space between the drum shutters and the steel piling for a height of 18 ft. Grout pipes $1\frac{1}{2}$ in. in diameter were inserted at 10 ft centres round the centre lines of both the circular spaces. Perforated tubes, 4 in. in diameter, were placed alternately with the grout pipes and were used for sounding the grout, which was pumped at an average pressure of 55 lb/sq. in. This operation was carried out progressively round the pipes and repeated until the whole waling was grouted. Some of the grout pipes became blocked and had to be withdrawn, cleaned and replaced. Observations in the sounding tubes indicated that the grout settled at an angle of about 35 degrees causing depressions, which were filled with grout later.

295. The object of blasting the Mackintosh Rock beneath the north pier was to remove its central pinnacle and form a reasonably level base of clean rock. Eight divers worked continuously for over three months preparing the river bed. Any scheme involving more diver work would have prolonged the construction time even further.

296. Mr Blair Birdsall, in his very useful contribution, referred to the pre-stressed anchor blocks. In the case of the Forth Bridge this type of construction ensured that the concrete anchor block was always in compression, a very desirable feature in an embedded anchorage where cracks might allow ingress of ground-water and start corrosion of the anchor strands. It also prevented a relative movement between the anchor crossheads and the concrete face.

Mr H. Shirley-Smith, continuing the reply, said that his first association with the Forth/Severn Road Bridge was in 1947 when, as explained by Mr Kerensky and Mr Crosthwaite, it was thought that the Severn Bridge would be built first. He well remembered having tests carried out on behalf of the Cleveland Bridge Company for the late Sir David Anderson, who at that time favoured site welds at the butt joints in the towers. At the instigation of the late Sir Ralph Freeman he worked on schemes for pulling across strands of parallel wire instead of spinning cables by the usual American method. Up to that time this had only been done on the Otto Beit Bridge of about $\frac{1}{3}$ the span of the Forth, but he came to the conclusion then, since borne out by researches of the Bethlehem Steel Company^{D24} and others, that it was certainly feasible.

298. In reply to the most interesting comments and questions by Mr Pain and Mr Morley, he thought that under the extraordinary weather conditions obtaining on the Forth it would unquestionably have been more economical if the cables had been erected by hauling over parallel wires instead of spinning in situ. All the vast amount of complex purpose-designed plant and machinery required for the cable spinning would have been saved. Nearly all the training period for the cable spinners would have been unnecessary. Moreover, on account of the enormous reduction in the number of adjustments needed for erecting strands compared with individual wires, there would have been at least four or five months' overall saving in time. Mr Pain's suggestion that the section of cables might be proportioned throughout their length to the actual stresses corresponding to their inclination was well worth studying. It might be possible to achieve this if the cables were erected as parallel wire strands, but Mr Shirley-Smith could see no way of doing it if

they were spun. He agreed with Mr Pain that there was considerable scope for further development in cable design and construction, and with Mr Kerensky that twisted wire ropes should be ruled out as theoretically unsound and uneconomical on long spans.

299. He did not agree with Mr Hyatt's assertion that six or seven months of construction time might have been saved if a floating-out scheme had been adopted for erection of the deck; this seemed to be very much in the realm of speculation. Detailed alternative schemes had been worked out and priced during the tender days, and it was agreed by all concerned that piecemeal erection by travelling derricks was the better scheme. Nothing that had happened at site gave Mr Shirley-Smith any cause to alter this opinion—in fact, quite the reverse. Owing to the aerodynamic instability of the deck, if panels were erected complete, very serious difficulties would have arisen. Also if the panels had been only partially assembled before floating out so as to keep them aerodynamically stable, the Contractors would have had to include the extra costs of plant and equipment needed to complete their erection in situ, in addition to the costs of the floating-out scheme.

300. In reply to Mr Brown, an area of at least $\frac{1}{4}$ sq. mile would have been needed for carrying out protection of steelwork, storage, and pre-assembly of panels prior to floating out. Even if the cost of reclaiming this area at Rosyth had not been prohibitive, he considered that the method adopted for erection of the superstructure was much the wisest when all the circumstances, including aerodynamics and the weather on the Forth, were taken into account. It was misleading to make comparison with the Severn and Tamar Bridges. These were entirely different designs in which no part of the deck panels had at any time to be omitted on account of aerodynamic considerations.

301. Mr Wood and others had referred to the great cost and loss of time incurred in fixing and moving all the temporary staging that was needed throughout the $1\frac{1}{2}$ miles of the underside of the deck. Mr Shirley-Smith agreed that the provision of three or four carriages running on a system of permanent rails fixed below the stiffening trusses might have saved an enormous amount of time and money. The rails would, of course, have had to be incorporated in the design of the superstructure, and the platforms would ultimately have served as the final painting gantries. During erection the Contractor would have had the use of the rails as suitable and safe supports for additional gantries, stagings and systems of safety nets. He sincerely hoped that no other great bridge would be built without this provision.

302. Mr Shirley-Smith much appreciated the President's kind reference to the safety precautions taken on the bridge, and he hoped and believed that the comprehensive series of tests carried out on site on the design of safety nets had been of service to the British Standards Institution and the Factory Inspectorate—there was now a British Standard Specification dealing with this subject. He entirely agreed with Mr Kinsella that safety nets, in addition to providing the last defence against accidents, gave confidence so that men worked more freely and quickly over them. It might well be true, as Mr Kinsella suggested, that the supporting frames of the two systems of nets used in the main span suffered from the almost ceaseless movement to which they were subjected, compared with those in the side spans which suffered less disturbance. This was an example of what Mr Shirley-Smith meant when he spoke of the *imponderable* forces to which structures were liable to be subjected

under working conditions and for which adequate allowance must be made in design.

303. He was entirely in agreement with Mr Hyatt that crash barriers, grillages and parapets in the deck should be designed with bolted and not welded site connexions. Months of tedious and costly work would have been saved on the Forth Road Bridge if this had been done. Many more temporary bolts and connexions had had to be welded on to the deck (and then cut off again) to erect these items than would have been needed in permanently bolted connexions. Any parapets or crash barriers which were damaged by traffic during the life of the bridge could also be much more easily replaced if these were bolted than if they were welded.

304. Mr Shirley-Smith agreed with Mr Knox that the design of the suspenders with two bights of rope and double sockets had led to considerable difficulties and called for the closest supervision to avoid risks during erection. It all seemed very easy on a drawing board, but the practical difficulties wasted hours of time on site. Single sockets at the ends of a single bight of rope were unquestionably preferable, and had a further advantage in that they needed no spacers to prevent ropes from clashing together.

305. Mr Scruton and Mr Crosthwaite had made interesting contributions on the aerodynamic aspects and he would like to congratulate the Designers on the success of the structural damping incorporated in the design of the deck. They seemed to have definitely overcome any tendency for oscillations to build up in a rhythmic manner and movements always petered out before amounting to more than a few inches. He was amazed, however, by Mr Scruton's statement that in 1957 he had warned the Consulting Engineers about the likelihood of oscillation of the free-standing towers, because the Consultants had given no warning whatever of this to the Contractors (was this perhaps an example of the kind of thing Mr Blair Birdsall had in mind when he advocated 'more liaison between designers and contractors in a common search for the most efficient structure consistent with rapid and minimum-cost fabrication and erection'?) No doubt the Consultants thought they had overcome this risk by the system of bolting adopted between sections of the tower legs. This damping, however, proved inadequate and, as Mr Morley pointed out, if the Contractors had been warned in advance, a period of considerable anxiety and risk could have been avoided. The Designers were to be complimented, however, on the speed with which they acted when the oscillations reached their climax.

306. Mr Shirley-Smith was grateful for the authoritative contributions from Dr Stüssi and Mr Blair Birdsall, both of whom had visited the site more than once during the building of the bridge. The points raised mostly concerned design, but he would like to express his agreement with Mr Birdsall in regard to the permissible periods for carrying out cable wrapping, the reduction of erection costs if the cable band bolts had been made longer, the disadvantage of the double sockets on the suspenders, and his clarification of the purpose of the counterweight tower and doubt whether the cost of its automated system was justified. Mr Birdsall had been on site whilst one of the extreme storms was raging in February 1962. He had seen conditions at their worst, and the havoc wrought on the catwalks, and his verdict on the cable spinning was one for which Mr Shirley-Smith had the highest regard.

Mr W. Henderson, completing the reply, mentioned that several contributors had raised points on the target price contract. Referring to Mr Goldstein's queries, the target sum was £4 254 000; setting aside variations in wages and materials, there was no material adjustment in the final assessment. In the outcome, the target price was beaten by a reasonable amount which was shared equally by the Contractor and the Client. Overheads were intended to provide for the cost of management and related real expenditure of this nature. Depreciation payments were based on the actual cost of new plant or on an agreed valuation of plant already owned by the Contractor; the rates were 2%/month for 12 months, and thereafter 1%/month. Referring to the suggestion that the modification of the profit seemed unbalanced, the following points should be noted.

308. In a normal fixed-price contract economies in construction techniques which were developed after the contract was made benefited the contractor alone. In this Contract, the Client shared to the extent of 50% of the saving. If the 'carrot' was made too small there would be little or no incentive to a contractor to strive to beat the target—he would simply be content to equal it. In Mr Henderson's opinion the division of savings on costs below the target must lie somewhere between a 60:40 and a 40:60 split between contractor and client if it was to remain an effective inducement, and, of course, the greater the proportion the contractor gained, the greater his inducement to achieve a reward in which the client automatically benefited. Looking back on this Contract, the 50:50 arrangement still seemed a most satisfactory division, and an equitable one.

309. Had the costs exceeded the target, the penalty was not solely a drop in profits; this would also have been associated with an increased turnover, and, since a large element of this turnover was in wages, with probably an increase in the working period, i.e. the Contractor would not only have been earning less but also taking longer to do so. In consequence his annual reward would have been diminished to a markedly greater extent. Given that the profit element was a reasonable one in relation to the target price in the first place, the loss of earnings to the Contractor resulting from exceeding the target price would have been, in Mr Henderson's opinion, a quite severe penalty.

310. In balancing the rewards and penalties in a contract of this nature, it was not simply a matter of aiming at arithmetical equality. One must set a reward which was likely to give the client the greatest benefit by endeavouring to adjust the contractor's reward and the client's share to the best advantage, while, on the other hand the penalty must be a powerful disincentive to inefficiency, but not so heavy that the contractor would refuse to participate.

311. Quite the most important aspect of a target price contract was that the target should be soundly based both as to rates and measurements, and that the finished job should not differ materially from the original conception. Mr Henderson agreed with Mr Smith's suggestion that the preparatory work must be at least as thorough as for an orthodox contract. This type of contract had not, to his knowledge ever been used in the past for bridgeworks, although he understood that this was the case in other fields where, he believed, the content of the work changed considerably in the course of its execution. One would imagine that in these circumstances contractual problems no less difficult than in an orthodox contract would arise. Indeed, they could be considerably more difficult.

312. Mr Shilston referred to possible contractual difficulties in this form of contract; this undertaking was carried out under the I.C.E. conditions and no serious problems had arisen. It was possible that this might have been more to the credit of the Engineers and the Contractors than to skill in the drafting of the Contract, but generally it was felt that contractual relationships were no more complex in this Contract than in the more normal type. This, of course, was a negotiated contract with a single tenderer, which might have made some difference. Mr Smith considered there was scope for this type of contract in the competitive field and one would hope this might be so. Considerable ingenuity would be necessary, however, in devising the terms to give satisfactory comparisons between tenderers, since a low 'offered' target price might be associated with a rather high profit element, the contractor being prepared to accept some abatement of his profit in order to win the contract on an unrealistic target tender. On the other hand a second firm could offer a larger, but realistic, target with a keen profit element, having in mind ideas yet to be developed and tested which could offer them a considerable reward in terms of bonus, and this could be most advantageous to the client. It could be that the solution would be to seek offers from two or three firms and after thorough examination and discussion with each contractor to assess which was likely to be the most advantageous, although not necessarily the initially lowest, offer.

313. Reverting to Mr Shilston's query about financing the Contractor, this had not been done beyond the point that the Client had paid for and retained ownership of the major items of special plant. In a contract of this sort it should not present insuperable difficulties for the client to set aside funds from which the contractor could draw directly to meet the cost of wages and purchases. A continuous audit would be necessary, but this was so in any case. In these days of expensive borrowing this procedure should be advantageous to public authority clients, but Mr Henderson wondered how much of the benefit would find its way back to the authority.

314. Mr Hyatt had suggested that it might have been advantageous to bolt rather than weld the crash parapets and grillages to the structure. It was probable that the costs of the two expedients would differ little, and after some experience of the bridge in use, there was no doubt that bolting would have been much more satisfactory; it would have allowed much more rapid and less expensive replacement in the event of damage and would have greatly facilitated access to the area between carriageways and below deck level. Undoubtedly all equipment of this type, liable to damage or possibly having a short life, e.g., road expansion joints, should be designed so as to be replaceable quickly and readily. Elsewhere²²⁵ Mr Henderson had suggested that battledecks and other elements highly susceptible to fatigue might economically be designed for a shorter life than the rest of the structure and be renewed as required. Whatever the cost aspect might be, Mr Price could be assured that this was a feasible proposition on the Forth deck, although it would be a considerable undertaking. Fatigue effects would not be significant elsewhere in the structure.

315. He was in entire agreement with Mr Wood that permanent travelling carriages ought to have been designed into the structure and provided at an early stage. They would certainly have been beneficial in completing construction after the bridge was opened to traffic. Economically they would

have shown a return at this stage in simplifying the work of completion. Early provision would also have eliminated a great deal of frustration and inconvenience for the Client following the take-over. Similarly, had painting gantries been provided on the towers, measures could by now have been taken to prevent the ingress of water between joints in driving rain. It was worth noting that in these conditions the vast catchment area presented by the vertical face of a tower was covered by a thick layer of water, much of which was thrust through minor openings by wind pressure. It was, however, only after completion of a project of this magnitude and complexity that the considerable importance of providing maintenance apparatus at an early stage and designing it into the bridge became apparent. In the United Kingdom at least, when this scheme was begun, far less thought had generally been given to maintenance problems than was now the case.

316. Mr Kerensky had referred to the live loads used in the design; at the time these were first put before the profession for consideration,^{D26} there was controversy as to whether they were too light or too heavy and opinion was about equally divided. Experience was now indicating that on fairly short loaded lengths they were adequate but none too heavy; in some fairly small elements fatigue became a factor of significance under HA loading with the high stresses now used. Providing for 45 units of HB loading generally eliminated this since it was infrequent and caused much higher stresses than HA on these elements. This seemed appropriate for the non-dynamic aspects of live load. On the other hand in the days following the opening of the bridge, when it was congested with slow-moving sight-seeing traffic, an estimate of its intensity led to the conclusion that 400 lb/lin. ft of lane for the longest loaded length was quite adequate, even with only two lanes taken as fully loaded and the others as $\frac{1}{3}$ loaded.

REFERENCES

- D1. SCRUTON C. An experimental investigation of the aerodynamic stability of suspension bridges with special reference to the proposed Severn Bridge. *Proc. Instn civ. Engrs*, 1952, **1** (March) 189–222.
- D2. FARQUARSON F. B. and VINCENT G. B. Aerodynamic stability of suspension bridges. *University of Washington Bulletin*, Parts 1–5, 1950–54.
- D3. STEINMAN D. B. A generalized deflexion theory for suspension bridges. *Trans. Amer. Soc. civ. Engrs*, 1935, **100**, 1133–1170.
- D4. TIMOSHENKO S. The stiffness of suspension bridges. *Trans. Amer. Soc. civ. Engrs*, 1930, **94**, 377–405.
- D5. ATKINSON R. and SOUTHWELL R. V. On the problem of stiffened suspension bridges and its treatment by relaxation methods. *J. Instn civ. Engrs*, 1939, **11** (March) 289–326.
- D6. CROSTHWAITE C. D. The corrected theory of the stiffened suspension bridge. *J. Instn civ. Engrs*, 1947, **27** (Feb.) 470–496.
- D7. CROSTHWAITE C. D. Shear deflexions in latticed structures. *Int. Assoc. Bridge Struct. Engrs Publications*, 1952.
- D8. SMITH H. S. and FREEMAN R. (jun.). The design and erection of the Birchenough and Otto Beit Bridges, Rhodesia. *J. Instn civ. Engrs*, 1945, **24** (May) 171–208.
- D9. EDITORIAL. Can the old spinning wheel retire? *Engng News Recd*, 1965 (25 Nov.) 15.

- D10. BROTTON D. M. *et al.* The solution of suspension bridge problems by digital computers (Part 1). *Struct. Engr*, 1963, **41** (April) 121-126.
- D11. BROTTON D. M. and ARNOLD G. The solution of suspension bridge problems (Part 2), *Struct. Engr*, 1963, **41** (July) 213-222.
- D12. BLEICH F. Dynamic instability of truss-stiffened suspension bridges under wind action. *Trans Amer. Soc. civ. Engrs*, 1949, **114**, 1177-1222.
- D13. VINCENT G. S. A summary of laboratory and field studies in the United States on wind effect on suspension bridges. *Proc. Int. Conf. Wind Effects on Bldgs and Structures*. National Physical Laboratory, Teddington, June 1963.
- D14. DUTT A. J. *An investigation of certain problems relating to wind loads on buildings and structures*. Ph.D. Thesis, University of Liverpool, 1963.
- D15. DWIGHT J. B. and HARRISON J. D. *Local buckling problems in steel columns*. Brit. Wldg Res. Assoc., November 1962.
- D16. DIN 4114, *Steel structures stability*. Deutsches Normen, July 1952.
- D17. TALL L. (ed.). *Structural steel design*. New York, Ronald Press, 1964. Chap. 9, pp. 293-295.
- D18. BLEICH F. *Buckling strength of metal structures*. New York, McGraw-Hill, 1952. Chap. 9.
- D19. SKEMPTON A. W. Long-term stability of clay slopes (4th Rankine Lecture). *Géotechnique*, 1964, **14** (2), 75-102.
- D20. FREEMAN R. and OTTER J. R. H. The collapse of the Second Narrows Bridge. *Proc. Instn civ. Engrs*, 1959, **12** (April) N36.
- D21. STÜSSI F. *Baustatik*, vol. II. Basel, Birkhäuser Verlag, 1954.
- D22. BIRDSALL B. The suspension bridge tower cantilever problem. *Trans. Amer. Soc. civ. Engrs*, 1942, **107**, 847-862.
- D23. ANDERSON J. K. Tamar Bridge. *Proc. Instn civ. Engrs*, 1965, **31** (Aug.) 337-360.
- D24. *A new concept in suspension bridge cable construction*. Bethlehem Steel Co., Bethlehem, Pa, Nov., 1965.
- D25. HENDERSON W. Fatigue life of structural members. *Final Report, 7th Congress, Int. Ass. Bridge & Struct. Engng*, 1965, Rio de Janeiro 1964.
- D26. HENDERSON W. British highway bridge loading. *J. Instn civ. Engrs*, 1954, **3**, Pt. II, 2, June.