Papers Nos 6355 and 6356

**Clifton Bridge, Nottingham: initial design studies and model test†
and****Clifton Bridge, Nottingham: design and construction‡**

by

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and

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Discussion

Mr Finch, introducing the Papers, expressed his appreciation of those who had been associated with him, both in the preparation of the Papers and in carrying out the project described. It had been a very large team and he did not intend to mention all those involved, but they included Messrs R. Travers Morgan & Partners (Mr Goldstein, a partner in the firm, would be speaking for himself); the contractor, James Miller & Partners Ltd, with whom should be linked one of the directors, Mr Buckley, and Mr Parry, who had been employed by the contractor as a consultant in connexion with the actual construction of the work. The Authors had been remiss in the Papers in not drawing more attention to Mr Parry's efforts as consultant to James Miller & Partners Ltd, but the work shown in Figs 40, 42, and 36 (part) should be associated with him. In addition, the Ministry of Transport had helped considerably through their technical staff, but not so much in the provision of money. Members of the staff of the University of Nottingham had been extremely helpful, as had also the Cement and Concrete Association. In addition, Mr Finch paid a tribute to the members of his own staff who had relieved him of a good deal of the work. The Corporation, in going ahead with the scheme, had accepted his advice and had been extremely generous; if other local authorities could be persuaded to make the same contribution, the roads problem of Britain would be quickly solved.

124. The City of Nottingham was the centre of a conurbation containing half-a-million people on the north bank of the River Trent, across which there had been only one bridge. He knew of no similar city with such a lack of facilities, and Trent Bridge was one of the most congested spots in the country. They had been thinking about the building of a new bridge for the past 40 years, but 5 years ago the position had become so bad that the Corporation decided that something had to be done—it did not matter at whose expense.

125. Bringing the work to fruition had involved many difficulties, which at one time had seemed to be almost insurmountable. The River Board had asked for a flood capacity under the bridge far in excess of that under another bridge farther down stream. He did not follow the reasoning, but it had been agreed to and had not improved the appearance of the bridge. The Navigation Authority had asked for certain facilities which they felt might be needed (although Mr Finch did not agree), and their wishes too had been observed. The Ministry of Transport had asked for trunk-road standards, but refused any financial contribution.

126. Finally, after most of the difficulties had been overcome, the Royal Fine Art

† Proc. Instn civ. Engrs, vol. 12, p. 289 (Mar. 1959).

‡ *Ibid.*, p. 317.

Commission had had to be consulted and it had been necessary to have something on paper to show them. Although an architect had been employed to give a lead to the engineers, many people did not agree with what was proposed. The Commission suggested that it did not look like an arch, to which the reply had been that it was not intended to be an arch. The suggestions of the Royal Fine Art Commission had added to the cost of the project, perhaps to the extent of £20,000 to £30,000.

127. The local authority would have been satisfied, for its domestic requirements, with a bridge costing about £100,000, but the bridge was on the line of a future trunk road and the Ministry of Transport had intervened. The local authority, which was a progressive one, had agreed to the Ministry's proposals and faced the necessary expenditure. The ultimate cost had approached £500,000.

128. At an early stage it had been known that there would be some difficulties with materials. At first, in an attempt to be a little progressive, they had envisaged a welded steel bridge, but then steel had become very scarce and its use had been banned. They had accordingly turned to concrete, but almost immediately cement had become scarce. After explaining that the prestressed design used the minimum of steel and the minimum of cement, the scheme was finally allowed to go forward.

129. The quality of the workmanship and the concrete finishes was above average. The City had demanded, and the contractors had supplied, a first-class job.

130. Minor difficulties had been encountered since finishing the bridge. There had been embankment settlement, which, so far as he could see, could not have been obviated except by spending a great deal of money. He wished that it were possible to plan further ahead and build embankments when it was seen that they would be necessary on the trunk-road system, and long before it was necessary to complete the works, to give them a chance to settle. In the present instance elementary steps had been taken which had been reasonably successful, and it was not thought that there would be much difficulty, apart from making good from time to time for the next few years.

131. He was satisfied that a prestressed job was competitive in first cost, and would be more competitive when British contractors had had more practice with this type of construction. There was a big saving in maintenance as compared with a steel bridge—almost £3,000/year.

Mr Goldstein, in further introduction, associated himself with the tributes which Mr Finch had paid to all those who had been concerned with the work, and added his own to Mr Finch and his staff, who, by their joint efforts during the construction of what was a difficult job, had greatly facilitated the task of the Consulting Engineers; the resulting collaboration was a very happy one.

133. In the Paper the Authors had tried to explain not only the "how" but more particularly the "why" of the various details of constructional work.

134. The contribution by public utility undertakings towards the cost of the bridge was of special interest. At Nottingham where the local authority had paid for the bridge the Undertakers had been co-operative in contributing a proportion of the cost for the facility of getting their mains across the river, as was shown in the Paper. As a general principle, however, the Public Utilities Street Works Act did not make clear what the legal obligations of all parties were in the case of bridgeworks where it could be established that the proportion of load—however small—taken by the mains was unavailable for traffic load requirements and reduced the latter thereby. They had, in fact, repeated the Nottingham procedure on a number of other jobs where the Undertakers had made a general contribution in addition to the cost of fixings and the like. In one case the number of mains had been so numerous that they represented a considerable proportion of the total live load. He felt, however, that the policy makers concerned with this aspect of the matter should address themselves to the problem and establish a definitive ruling.

135. Mr Goldstein emphasized the point made in the Paper about the time at which the design had been done. In the past 5 years there had been much refinement of, and many new developments in, analytical techniques, and the procedure used in the

analysis of Clifton Bridge was only the outline of the much more comprehensive techniques now available.

136. Again, if the original design had been abandoned when it became clear that the bridge would have to be skew and a new design attempted, he felt that over-all there would *not* have been a loss of time since the loss involved in the abandonment of the former design would have been more than compensated by the simplicity of a new design for a skew bridge or by re-planning the whole crossing so as to use a square bridge, assuming that adjacent development made that possible.

137. It would generally be agreed that a continuous bridge would have been preferable for many reasons, not the least of which was the awkward detail (both technically and aesthetically) at the scarfed joint of the suspended span. Further, if the "cantilevering-out" technique had been adopted *ab initio* (in this case, as the Authors had said, a by no means clearly established economy) then with a continuous bridge the technique would have proceeded for the whole of the deck right to the centre of the span.

138. The torsional and skew calculations of the cantilever had been extremely subjective and provided a typical example of the use of engineering sense aided by very general calculations.

139. In connexion with the point in § 105 about the specific surface of cement, there was no doubt that a distinct cleavage of opinion existed between a good many practising engineers on the one hand and physicists, chemists, and general research workers on cement on the other. There appeared to be no conclusive experimental evidence to support either view and he would follow the views of practising engineers until conclusive evidence *per contra* was available.

140. There were now some additional data, obtained at the time of the final inspection.

141. First, a further set of readings on the expansion joints had been taken on 16 March, 1959, at a temperature of 50°F and Table 12 shows them incorporated in

TABLE 12.—MEASUREMENT ON MAIN-BRIDGE EXPANSION JOINTS

(The March and May readings represent closing of the joint and expansion of the structure).

Date	Temp. °F	Joint No. 1 (North abutment)				Joint No. 2 (North end of suspended span)				Joint No. 4 (Junction abutment)			
		East		West		East		West		East		West	
		Read- ing	Diff.	Read- ing	Diff.	Read- ing	Diff.	Read- ing	Diff.	Read- ing	Diff.	Read- ing	Diff.
25 March 1958		4 $\frac{1}{8}$ "		4"		4 $\frac{1}{8}$ "		4 $\frac{1}{8}$ "		4 $\frac{1}{8}$ "		4 $\frac{1}{8}$ "	
14 May 1958	60	3 $\frac{7}{8}$ "	- $\frac{5}{8}$ "	3 $\frac{7}{8}$ "	- $\frac{1}{8}$ "	3 $\frac{1}{8}$ "	- $\frac{1}{8}$ "	3 $\frac{7}{8}$ "	- $\frac{5}{8}$ "	3 $\frac{5}{8}$ "	- $\frac{5}{8}$ "	3 $\frac{5}{8}$ "	- $\frac{1}{8}$ "
14 October 1958	62	3 $\frac{7}{8}$ "	—	3 $\frac{1}{8}$ "	+ $\frac{1}{8}$ "	3 $\frac{1}{8}$ "	—	3 $\frac{7}{8}$ "	—	4"	+ $\frac{3}{8}$ "	4"	+ $\frac{3}{8}$ "
16 March 1959	50	3 $\frac{5}{8}$ "	+ $\frac{1}{8}$ "	3 $\frac{5}{8}$ "	- $\frac{1}{8}$ "	3 $\frac{1}{8}$ "	—	4"	+ $\frac{1}{8}$ "	4 $\frac{1}{8}$ "	+ $\frac{1}{8}$ "	4 $\frac{1}{8}$ "	+ $\frac{1}{8}$ "

Table 11. It could be seen that the mean movement at Joint No. 1 was negligible; at Joint No. 2 there was a mean shortening of $\frac{1}{16}$ in. and at Joint No. 4 there was a similar shortening of $\frac{1}{16}$ in. (all since October 1958). These movements could well be creep movements as distinct from temperature movements and the readings seemed further to substantiate points made in the Second Paper that the temperature movement of this type of concrete bridge did not appear to be nearly so great as that currently allowed for during design.

142. Only one set of strain plugs had been found intact in the deck of the bridge; these were in the south end span near the pier on an inner web and were horizontal in direction. The change in strain during the year preceding the final inspection had been about 6×10^{-5} . This was a creep compressive strain (the permanent stress being about 1,000 lb/sq. in.) and was quite negligible.

143. The strain plugs at the bases of the south rocker columns were all available, however, and were vertical in direction. During the year preceding the final inspection, compressive strains had occurred to the north faces of these columns at their base, the average value being 14×10^{-5} (range 13 to 17). At the south face of the columns the average strain had been tensile with a value of 3×10^{-5} . Analysing these strains, allowing for direct load and bending and for creep movements, an approximate assessment yielded an increase in bending stress of about 250 lb/sq. in., which had to be compared with an $\frac{1}{8}$ -in. shortening movement of the end span (this producing bending moments in the rocker column) and the correlation seemed quite reasonable.

144. The cost analysis of the bridge was given in Table 13. This, of course, was the

TABLE 13.—COST ANALYSIS
Total contract sum = £345,000 Cost of North Viaduct: £33,173 (5.9)

Item No.	Item	Unit	South viaduct and main bridge
1	Total cost	£	311,827 (8.4)
2	Cost of substructure	£	87,652 (2.4)
3	Cost of deck structure	£	224,175 (6.0)
4	Total concrete quantity in substructure	cu. yd	3,331
5	Total concrete quantity in deck structure	cu. yd	3,191
6	"Equivalent slab thickness"—deck only	in.	28
7	"Equivalent slab thickness"—deck and substructure	in.	57
8	Total weight of reinforcement in deck structure	tons	244
9	Total cost of reinforcement in deck structure	£/ton	82
10	Total weight of prestressing wire in deck structure	tons	109
11	Total cost of all prestressing materials and labour in deck structure	£/ton of wire	486
12	"All-in" cost of deck per cu. yd of deck concrete	£	70
13	"All-in" excluding cost of Items Nos 9 and 11	£	47
14	"All-in" cost of bridge per cu. yd of deck concrete	£	98
15	"All-in" cost of bridge per cu. yd of total concrete	£	48
16	Total cost of bored piling (3,444 ft)	£	9,500

Note:—

- Figures in parenthesis denote unit cost in £/sq. ft.
- Costs include 22% preliminaries.

cost to the client, there being no data on the actual contractor's cost. It should be noted that with regard to the statistical figures given in the Table, the idea had been to provide unit "covering" rates which might be of use to engineers estimating for broadly similar work. Great care, however, would have to be exercised to ensure that the rates given were not misleading, since not only did they apply to this particular job only but they might also be "weighted" owing to the contractor's particular requirements, as was not unusual when bills of quantities for this kind of work were priced.

145. In view of the fact that a second bridge would some time be built alongside the new bridge, the rates given would not be precisely relevant, since the second bridge in all likelihood would be significantly different from the first (though probably not in appearance).

146. Commenting on the costs, the overall cost per square foot seemed quite a reasonable and competitive figure. Of interest was the low cost of the substructure compared to the deck structure, the ratio being about 1:3. This might be a feature of the particular type of design and construction adopted. Mr Goldstein felt that the all-in cost of the deck per cubic yard of deck concrete (£70) was fairly high and reflected the complexity of the deck construction. Of interest also was Item No. 11 which he also felt was rather high. It could be noted that for the south viaduct only the figure was £539 as against the similar figure for the main bridge of £467. The reason for this was probably the considerably greater number of transverse prestressing cables used in the south viaduct.

147. Finally he mentioned two corrections in the text of the Paper. In § 37, line 6: "mean" should replace "main", and the captions for Figs 13 and 15 should be interchanged.

Mr H. C. Adams (Assistant Chief Engineer (Bridges), Ministry of Transport and Civil Aviation) said that one of the most valuable features of the Papers was the manner in which the Authors, after the completion of the job, had sat back and criticized their own work. They had said in many cases what they would do next time, if they had another chance.

149. The first point which called for comment was the obvious one of the skew nature of the bridge and the implicit plea that when deciding the precise line of a crossing full weight should be given to the price which had to be paid for the skew nature of the bridge, a cost, as the Authors pointed out, in time and money, and also, he thought, in most cases in appearance. Could the Authors assess the loss of time and loss in actual cost of construction in making the bridge skew instead of square? To have made it square would have necessitated a slightly different line and possibly longer approaches. If another parallel structure were to be built on the site later the cost might be doubled.

150. Another point which was brought out strongly in the Paper was the complication of construction in post-war as compared with pre-war bridges, and the extra thought, the anticipation of difficulties, and the care in supervision and construction which had to be devoted to these bridges in order to avoid serious trouble. From § 81 it was apparent that this had extended to the workmen, and that after they had produced the good quality necessary for prestressed concrete it was not to be expected that they would fall below that standard for the reinforced concrete, for which this high quality was not quite so necessary. That was reassuring, but with the rich grades in question there were the associated risks, as the Authors had pointed out in § 105, of additional shrinkage, and a very serious warning was given in that paragraph to those who advocated permissible tensile stresses in prestressed concrete. The subject was one which required careful thought.

151. It was difficult to find fault with the Clifton Bridge. There was perhaps one criticism of the appearance, for in this, as in all bridges of this type with a suspended span, there was a flatness of line at that span, which tended to spoil the grace of the curve. That was perhaps inevitable, because the suspended span with functional design should have a straight soffit; but next time, the Authors said, they would avoid the suspended

span and build by cantilevering all the way across, and then they would preserve the quality of that curve.

152. The other fault was one that had occurred in a number of skew bridges. The piers had their shoulders projecting beyond the superstructure, and that left a very awkward little area just below the beam, which in the present case had not been dealt with quite happily. He had no doubt that on another occasion the Authors would find a better solution.

153. The fluting in the elevation of the bridge had greatly improved the appearance. It would probably be used in a number of other cases. By stepping back the construction joints and carefully lining with a vertical line at the cusp of the curve, the engineers had taken great care to ensure that the quality of the fluting was not spoilt.

154. The Authors had referred to the progress which had been made in the past 5 years in the design of prestressed bridges. That had been within the period of the design and construction of the Clifton Bridge, and it was a matter for satisfaction that at long last engineers in Britain were making their contribution to that progress. One result of it might be the avoidance, or a movement towards the avoidance, of transverse prestressing in the decks of prestressed bridges. The Cement and Concrete Association had recently been carrying out very valuable work, which might result in a big movement in this direction if a way could be found of combining prestressed longitudinal design with ordinary reinforced concrete design in the transverse direction.

155. Engineers were always looking for subjects for experiment to provide an opportunity for research. In most prestressed bridges there was considerable vibration, and although the Clifton Bridge was remarkably free of it, Mr Adams thought that vibration in prestressed concrete bridges offered a field for further valuable research in the immediate future.

Mr S. A. C. Henry (Contractor's Agent, James Miller & Partners, Ltd) dealt with some practical points of construction. The casting of the 100-ft and 90-ft beams had involved a number of problems. The stressing operation produced an appreciable shortening of the beam soffit and transferred the whole of the vertical load to the ends of the beam. It had therefore been necessary to cast the beams on bearings of adequate carrying capacity for these conditions.

157. So far as post-tensioned work was concerned the grouting appeared to have been the weakest point in the whole design. It was very easy to assume that a duct could be filled with grout but no adequate explanation of what happened to the excess water appeared to be available. The wide divergence in grouting specifications indicated considerable differences of opinion on the subject.

158. He queried the value of some of the precise measurements made on the job and found it difficult to believe that anything more than a tendency could be deduced from them—as indeed the Authors had stated. For example, a 90-ft-long beam, prestressed on a hot day, was observed to have a sideways movement at its centre of as much as 1 in. as the sun passed from one side in the morning to the other side in the afternoon. With movements of that sort strain-gauge readings might be misleading.

159. Reference had been made to the method of construction by cantilevering from the piers, without any falsework in the river; and the economics of the method had been questioned. It was an interesting point for professional discussion, but the Directors of the Company who built the bridge had had to decide whether or not to put their money into this method, and there should, he felt, be more recognition of that fact from the profession.

160. Mr Adams had referred to the pleasing mat surface produced by the fluted shuttering; Mr Henry (pointing out that this was one of the many ingenious ideas put forward by Mr Cyril Parry, who had acted as consulting engineer to the contractors) advocated that all large areas of concrete should be broken up by some form of pattern.

161. Mr Henry endorsed Mr Adams's views on the cost of skew and the inordinate cost of transverse prestress. The shuttering complications brought about by skew were fairly obvious but in addition to this he considered that the effect of the skew of the

Clifton Bridge had necessitated the employment throughout the job of two extra engineers to deal with it. Apart from the direct cost of the stressing operations, transverse prestressing made the programming of a bridge much more difficult, and lengthened the time of construction abnormally.

162. Mr Finch had mentioned the cost of prestressed bridges and had said that they were competitive in first cost. When properly designed and properly constructed a prestressed bridge was certainly competitive so far as maintainance was concerned, but the first cost of a prestressed bridge was difficult to arrive at because so many contractors had built such bridges at an uneconomic price. It was interesting to note, however, that the figures given by Mr Goldstein in presenting the Papers, showed a cost of about £8/sq. ft of deck area. He understood that a major steel bridge had recently been let at a cost of £13/sq. ft—a very appreciable difference.

163. Mr Henry drew attention to the design of the deck of the north viaduct. This was a type of bridge comparable to many bridges now being designed for motorways, and a better design in his opinion than the hollow plate, or inverted-T-beam type which seemed to be commonly favoured.

164. Mr Henry concluded by showing two slides to demonstrate the very marked contrast between the approach to design in Britain and in America.

165. The first slide showed a section through a 90-ft span of the south viaduct (see Fig. 25). The beams were cast individually; the diaphragms were then cast; part of the transverse prestress was applied; and the deck was cast, but separated from the diaphragms. The remaining transverse prestress was then applied and the deck made integral with the diaphragms. The whole operation had taken several months and was made up of so many dependent operations that this time could not be reduced. The shapes were complicated, the shuttering expensive, and the concrete placing difficult.

166. The American approach to a similar problem was illustrated by a section through one of a series of 120-ft spans.¹⁴ In this case the beams were rectangular in section, the diaphragms were simple, and it had been possible to move the shutter for a complete span, as a unit, and to concrete a complete span in one day. He felt that British engineers should pay attention in design to speed of construction and to the cost of labour.

Mr J. C. Adamson (County Engineer and Surveyor's Department, West Riding of Yorkshire) said that he had worked for 2½ years on the job. One of the most important features and one of the most difficult day-to-day tasks had been the grouting of cable ducts. The grout used had been a nominal 1:1 sand/cement mix with a plasticizer added. There were very few cases where there had not been trouble in grouting. There had been frequent stoppages, and in nearly every case they had been caused by small pieces of gravel and pieces of sand in the nozzle and in the pump itself.

168. The most important point in grouting was to get the grout round the wires, and to do this it was better to use a neat cement grout. The Cement and Concrete Association recommended it, and Mr Adamson had recently heard a lecture by Professor Lyse of Trondheim University, who had done many experiments on grouting, and who also recommended it. The advantages gained by getting the grout in quickly more than compensated for the fact that there might be a little more shrinkage in neat grout than in grout that had sand in it.

169. The Authors had rightly emphasized the need for a generous allocation of competent site staff for dealing with construction of a highly technical nature. That applied also to the Resident Engineer's staff, who had to deal with a lot of precision levelling, strain-gauge measurement, calculation of cable extensions, testing of materials, etc. Much of this type of work could not be done by the ordinary clerk of works or inspector, and had to be carried out by engineers. However, with a structure of

¹⁴ "Prestressed Bridge: a 120 ft span erected daily." *Engng News Rec.*, vol. 159 (July-Dec. 1957), p. 40.

medium size it was not possible to duplicate the site staff, and the engineer on the site had to do a great deal of work in the detailed supervision of concreting, grouting, tensioning of wires, etc. On the Clifton Bridge scheme there had been a clerk of works for most of the time, but no inspectors, and engineering assistants had to do most of the detailed supervision.

170. Although the structural records had been kept reasonably up to date, any attempts to do research work had failed. That was because the prime responsibility of the Resident Engineer had been the construction of the job, and not research work. If it was desired to have full-scale research work carried out on such jobs it was necessary to have someone on the site whose prime responsibility was to do it. Arrangements must be made beforehand and incorporated in the contract documents.

171. Mr Adamson agreed with the first two conclusions in § 117. There was no doubt that skew was a tremendous drawback on these bridges. He was at present engaged on the design of a 700-ft viaduct, 70 ft high (Paper No. 6355 indicated the results that might be obtained with a skew bridge of this type). Fortunately, at a very late stage in design he had managed to have a square crossing substituted for a skew with great benefit to the detailing of the project.

Mr R. E. Rowe (Research and Development Division, Cement and Concrete Association) said that in § 19 it was stated that the loss of prestress assumed in the initial design calculations had been 15%, and the Authors had mentioned that subsequently in analysis the loss had been found to be less than 15%. Could they elaborate on this and say what percentage loss they had found? Also, had any results obtained from the bridge shown that their assessments in the design stage had been correct? Much could be learned from measurements on site with bridges of the type in question, particularly in relation to the loss of prestress to be allowed for at the design stage.

173. Would the Authors explain their statement (§ 22) that for various reasons skew spans were preferable?

174. In §§ 43 and 44 the Authors had stated that they had carried out some control tests on small perspex beams, and that on these the creep was insignificant; but on testing the model the creep had been found to be significant. Mr Rowe thought that in the beam tests the creep had been there, but that the time period over which the readings had been taken had been the same as in the model tests. Generally speaking, with material of the nature of perspex it was necessary to allow between 12 and 18 min during which creep could take place, and standardize on a set time during which the readings were taken.

175. Referring to model tests in general, he emphasized the value of the information that could be obtained from structural models of complex bridges and other engineering structures. Where normal methods of analysis were not possible a model would show exactly what was going on, and, provided that it was possible to interpret the model results accurately, the behaviour of the prototype could be accurately predicted. Fig. 20 was quite a good illustration of this. There were two curves, one showing the results from the model as a series of dots, and the other the assessment of what would happen to the actual skew structure from a method of analysis. Mr Goldstein, when introducing the Paper, had mentioned that in fact the analysis of the bridge had not been rigorous, because that type of bridge did not lend itself to that type of analysis. He was to be congratulated on the accuracy of his engineering judgement in obtaining such very good agreement. In the series of model tests described perspex had been used, but there was no need to use a material of that type; it had been found that model concrete or mortars could give very accurate information about the behaviour of bridges and various other types of structure. This field had been sadly neglected in engineering science.

176. In § 68 the Authors stated that in the analysis of the anchorage stresses in the prestressed units the stresses due to the various cables employed could be readily estimated. He thought that this was a little misleading. Stresses in the ends of a prestressed beam were fairly complex. Several methods of analysis were available but none

of them had been adequately proved by tests, and recent tests had shown that those normally used were in error to some extent, in that they slightly underestimated the stresses and did not give accurately the position at which the stress occurred. Considerable research was needed so that designers could have a better idea of the state of stress for which they were reinforcing the ends of prestressed beams.

Mr Roy Galbraith (Assistant, R. Travers Morgan & Partners, Consulting Engineers), commenting on the Authors' statement that for the reinforced concrete mixes the question of minimum design strength hardly arose, since the strengths obtained were two or three times the minimum, said that the figures were:

Mix C. 5:8:1. Mean at 28 days, 6,050 lb/sq. in., compared with 2,850 lb/sq. in. minimum design strength. Standard deviation, 770 lb/sq. in. Coefficient of variation, 12.8.

Mix B. 5:1. Mean at 28 days, 8,550 lb/sq. in., compared with 3,300 lb/sq. in. minimum design strength.

178. The Authors attributed this quality to the fact that the concrete gang had been trained to produce prestressed concrete mixes. As a comparison, Mr Galbraith had taken some figures from a large building under construction in Holborn. With a 1:1½:3 mix, using ¾-in. aggregate, the figures were:

Mean at 28 days, 7,680 lb/sq. in., compared with a minimum of 3,750 lb/sq. in., c.p. 114. Standard deviation, 770 lb/sq. in. Coefficient of variation, 10.5.

The figures for the standard deviation and coefficient of variation were similar to the Authors. The Holborn building was typical of a number of jobs at present under construction. It seemed, therefore, that where good control was observed rather higher working stresses might be appropriate.

179. In § 103 the Authors discussed stressing. For the cantilever sections stressing had been allowed at the very early age of 2–3 days. That had been possible because the stress at any section caused by the cables anchoring at that section had been small. The purely local anchorage stresses had occurred where the concrete had been at least 7 days old.

180. The vertical offsets allowed in casting the cantilever had been mentioned. In addition, the rocker columns had been offset from the vertical by ⅜ in., to counteract some of the effect of creep and elastic shortening, so that when the bridge reached maturity the columns would be vertical, with the bridge in a neutral position so far as temperature effects were concerned.

181. Several difficulties had militated against the accurate assessment of bridge deflexion, one being that the effect of creep on the box sections would be different from that on the I-sections, so that in effect there would be two different Young's moduli.

Dr J. A. N. Lee (Assistant Engineer, Freeman, Fox, and Partners, Consulting Engineers), said that late in 1956 the Department of Civil Engineering of the University of Nottingham (where he had then been a Research Fellow) had been asked by Mr Finch to co-operate in the experimental investigations on the north viaduct of Clifton Bridge, in order to ascertain the effect of differential shrinkage between the in-situ slab deck and the precast beams. Considering the cross-section of the deck (Fig. 6b), the only beams readily accessible had been those under the cycle track and footpath, since the clear space between beams was 1 ft 3½ in. under the carriageway and 2 ft 2 in. elsewhere. Two beams in the central span had therefore been chosen for investigation. On each side of the upper flange of each beam 8-in. mechanical gauge pips had been fixed, and also under the soffit in three longitudinal positions—at each quarter-span point and on the centre-line. Gauges had also been set up in two positions on the lower face of each diaphragm.

183. The beams had been precast in July and August 1956 and finally stressed 3 weeks later. The diaphragms had been cast in situ early in October 1956 and transversely stressed a week later. The average cube strength at 28 days had been 7,500

lb/sq. in., and the value of Young's modulus calculated from the formulae given by Walley¹⁵ had been approximately 5.8×10^6 lb/sq. in. Since the slab had not to be cast until March of the following year, it had been considered that the rate of loss of stress at that time would be negligible compared with any changes due to differential shrinkage.

184. Readings had been commenced a few days before the first casting of the slab and had been continued at 2-day intervals for a month, after which time the reduced results had seemed to be fairly constant. The slab had been cast in six bays. The gauge readings obtained had been adjusted for temperature and loading. The temperature at the time of reading had been recorded, but later it had been decided to calculate the temperature strains on the basis of the average temperatures over the previous 24 hours. The change in strain due to loading had been computed by the load-distribution technique based on the bare grillage (i.e. without the top slab). This had obviously been modified as each cast of deck hardened, but there had been no method of allowing for this effect. The comparison of the strain readings on each side of the top flanges had been extremely good and well within the bounds of accuracy of the gauges, but there appeared to be a fairly high degree of scatter between successive readings, the causes of which were determinate.

185. From a typical set of results obtained beneath one of the earliest cast panels of deck (Fig. 49), it was surprising to find that the expected differential strains had been

FIG. 49

achieved within a few days of the casting, and fell off thereafter. The maximum values of strain, however, agreed fairly closely with that expected for a differential shrinkage of 400×10^{-6} . He did not know what values the Authors had taken.

186. Dr Lee thought that fewer beams could have been used in the north viaduct, which was not restricted in headroom; a beam spacing of 2 ft 7½ in. (ctrs) seemed rather close.

Mr Gerald Cramp (Senior Assistant, R. Travers Morgan & Partners, Consulting Engineers) referred to the vibration of component parts of the structure. Problems of

¹⁵ F. Walley, "Prestressed concrete design and construction". H.M.S.O., 1953.

that kind were difficult to analyse by any accurate method, but a number of approximate methods had been put forward over the years. The Rayleigh-Litz technique² had been used in solving the problem. It had been necessary to make some assumptions; for example, they had had to assume that the bridge was square and that there was no restraint at the piers or from the flexible columns at the abutments. Although they had realized that some kind of torsional vibration must exist, they had not been prepared to go into the problem. Whatever torsional vibrations did occur must be almost infinitesimal.

188. They had divided the problem into two parts, analysing the suspended span as a completely separate entity and assuming rigid supports. That was a simple analysis. The problems had been confined largely to the cantilever and end span, and here the primary difficulty had been in deriving a function for the vibrating shape that would satisfy all the end conditions. However, having discovered, after much searching, a function that would express this shape, they had then equated energies on a semi-graphical basis.

189. Unfortunately the structure did not lend itself to any kind of test so far as vibrations were concerned. It was, however, extremely stiff, and one could hardly feel any movement at all, even during the passage of heavy vehicles.

Mr H. R. Webster (Resident Engineer (Bridge works), Oxford Southern and Western By-Pass; formerly Assistant Resident Engineer, Clifton Bridge) said that two skilled tradesmen had been employed for finishing the concrete on the bridge and the whole of the visible surface had been rubbed down or otherwise treated. Half-way through dealing with the second cantilever one of the men had left and his work had been continued by the second. Although both men had been trained to do the work, the effect produced by the second man was quite different from that left by the first. Mr Webster had seen the bridge a few weeks ago and the result had been toned down, but for a time it had been most noticeable, particularly after a good deal of rain.

191. For surfacing the bridge, a base-course of two layers of bitumen sheeting, a layer of tiles, and a 2-in.-thick asphalt wearing course had been used. It had been believed that that was waterproof, but where the expansion joints had to be joined in that was not the case. A problem which had arisen with the tiles was that they had had to be left for some months until the Barber-Greene could be used to lay the surfacing over them. When the weather was bad, water appeared to have percolated through the joints, with the result that at one or two places there might be pockets of water lying on the felt and beneath the tiles.

192. The standard construction joint that had been generally accepted in civil engineering for some years, namely, the key joint, could be difficult to make, particularly when there was steel projecting through it, as the contractors would no doubt agree. In view of a few minor cracks which had been seen, it seemed that this type of joint could induce cracks in the wrong position rather than avoid them. The construction-joint problem was difficult, and an answer had not yet been found. For walls, not required to retain water, a butt joint would be more serviceable, particularly where it was possible to let in a triangular fillet, which might make it a "crack-inducer", in which case any crack would not be visible.

193. The fluted finish was remarkably effective in masking the construction joints, which otherwise would have been very obvious. Mr Webster had noticed, however, although he had had no training in aesthetics, that a surface of that kind appeared to require a support, and he thought that the fluting would be improved by having a band at its base. He had occasionally had the feeling that the fluting would slide off into the river!

194. The drawings of the metal expansion joints showed that one side of the bridge was tied within small limits to the other by the nature of the joints. A joint that could be adjusted to the concrete rather than the concrete to the joint (which in fact had had to be done on a number of occasions) would have been better.

195. The roadway fall for drainage was only 2 in. in 22 ft. When levelling up for

the placing of the fascia units, on the south viaduct, which had to be set to a straight line, they had found that spans 1 and 3, which had been prestressed completely at 10 days, had "hogged" considerably more than span 2. They had had to draw a tangent between the highest points of spans 1 and 3, which left them with a very deep curved underfilling to the fascia on span 2. The resultant raising of the fascia units had left a slight band of unfluted finish, but it would not be seen by the ordinary user of the bridge.

Sir Allan Quartermaine referred to the appearance of the bridge. He congratulated the Authors and designers on what they had succeeded in producing.

197. He was at times associated with architects and others in judging the appearance of engineering structures, and in view of the differences of opinion which sometimes arose on certain points he would like to put two questions to the Authors, but he wished to say that they were put in order to obtain guidance for himself and others and not for the purpose of criticizing the design.

198. Fig. 45 was a general photograph from which it was clear that, as stated by the Authors, the bridge was divided into two parts, the main structure and the approach spans. That had certainly been done for good reasons, and he would like to know exactly what was behind the making of that division. There was an abutment pier at the end of the south arm of the cantilever, a cellular pier containing the flexible columns for the cantilever and also the columns for the adjoining approach span, resulting in that pier being somewhat wider than the other piers. It was perhaps wider than necessary, and intentionally so for the purpose of dividing the bridge quite clearly into two parts. In addition, the pier projected beyond the face of the spandrel and came up to the underside of the overhang at the top of the spandrel, thereby accentuating the line of demarcation between the main span and the approach span.

199. If in the case of a similar bridge in future it was desired on aesthetic grounds to see the cantilever span sweeping into the approach span with no definite change between the two portions of the bridge, would the Authors say whether there would be any difficulty in doing that, other than, presumably, the need to make the piers under the approach spans very slightly wider? He was not saying that that should be done, but had the Authors made the abutment pier as they had for functional reasons only, or for aesthetic reasons, or for a combination of the two?

200. The junction between the suspended span and the cantilever arm was masked by a projection on the spandrel. Did the Authors see a reasonable chance of avoiding that projection on the spandrel without leaving something more untidy in appearance at the junction? He did not say that that would be better, but he would like to know if it was practicable.

201. Sir Allan said that he had to listen to criticisms of engineers' designs, and in some cases the designs that came before him and his colleagues were aesthetically unworthy of the engineering profession. That happened to an undesirable extent in the case of structures which, although small, were in important positions, much in the public eye. He would urge the younger generation of engineers to remember that the reputation of the profession depended not only on efficient design from the structural point of view, but also on the fine proportions and graceful appearance of the finished structure. If engineers themselves achieved noble and worthy designs architects would not then take from them the credit for the appearance of their structures. Remembering Brunel, Telford, and others of their predecessors who designed with inspiration, engineers today should use modern skill to produce reasonably light structures of good appearance and pleasing lines worthy of their heritage.

The following contributions were submitted in writing.

Mr E. S. Lyle (Senior Design Engineer, John Mowlem & Co. Ltd) referred to § 32 where, when investigating the vibration characteristics of the deck, the Authors had assumed $E = 6 \times 10^6$ lb/sq. in. In § 53 the value was assumed to be 4×10^6 lb/sq. in.,

while in § 108, when comparing the calculated with the measured deflexions, the value was assumed to be 5×10^6 lb/sq. in. Would the Authors explain the reasons for the different values and say what value had been used for the main prestressing calculations and on what basis it had been assessed.

203. Mr Lyle's company were at present concerned with some long prestressed concrete piles. In the design state a cube strength of 7,500 lb/sq. in. was adopted. It had been important to make a realistic assessment of the corresponding value of E so that the load-carrying capacity of the pile, as a long column, could be established. The British and German Codes suggested that for this cube strength E was about 5.5×10^6 lb/sq. in. This, he felt, gave an optimistic view of the long-term column strength and $E = 4 \times 10^6$ had in fact been used for design. Recently a test had been carried out on one pile, 119 ft long, by measuring its deflexion under its own weight. The actual cube strength had been 8,900 lb/sq. in. and E had been found to be 3.9×10^6 lb/sq. in. This isolated result might be misleading and further tests were being carried out.

204. Would the Authors agree that in prestressed work the value of E was an important factor and that in any major work it should be checked by tests? Had any such tests been carried out for the Clifton bridge?

Mr Finch, in reply, said that the Authors very much appreciated the reception given to their Papers. He accepted Mr Adams's criticisms and his compliments. His criticisms were very fair and Mr Finch was in agreement with most of them. He had expected criticism of the skew plan and must accept a great deal of responsibility for it, but there had been some reasons for it. The first was that the alignment of the bridge had been fixed about 25 years ago, when it had been hoped that it would be possible to construct it within 3 or 4 years. Provision had had to be made for other development, and they had been gradually forced into accepting the line laid down at that time. That had not been the only reason for keeping to the skew. He had underestimated the effect on construction, but other considerations had had to be taken into account. At the south end of the bridge there was now, and would be in the future, a very difficult road junction. The straight-through road would go to Leicester and the road to the right to Birmingham and the south-west. It would provide a major link with motorways. If the bridge were square it would make the angle of junction very acute, and it had appeared at the time that this would create more difficulty than the construction of a skew bridge. There could be no doubt that that had affected the cost of the bridge, but Mr Finch could only guess at how much. The quantity of material would be about the same, but in considering labour, where most of the loss of money went, he would guess the loss as a period of 3 to 4 months. Translated into money, that would mean between £15,000 and £20,000. He would like Mr Goldstein to refer to that, because they had not discussed it.

205a. Mr Adams had referred to the concrete quality, and it was true that the form of construction adopted did demand a high-quality concrete. The contractors had felt the effect of that; it meant that they had to be on their toes the whole time. Mr Finch agreed with the comment about the projecting heads of the piers. They had intended to cap them all, but, since the bridge was likely to be duplicated in the next 3 or 4 years, it would only be necessary to cap the piers on the south-west side.

206. The Authors appreciated the comments of Mr Henry and Mr Adamson from opposite sides of the fence. They had worked amicably together. On one occasion Mr Henry's ganger had ordered Mr Finch off the job, but subsequently Mr Henry had fallen into the river, so that they had finished up all square!

207. Sir Allan Quartermaine had referred to the wide pier. That had been introduced, with the fascia unit, by the Royal Fine Art Commission. There was room for argument there for another 12 months. The City of Nottingham had been prepared to accept anything to get the bridge built, and it showed what was likely to happen when too many opinions were introduced, unless there was some individual with over-riding authority. The cantilever had been swept into the beams in the original design submitted to the Royal Fine Art Commission. Mr Finch was in favour of consulting

anyone who could contribute anything useful, and it had been the practice of the City of Nottingham to employ architects when necessary.

208. With regard to Dr Lee's suggestion (§ 186) that the number of beams in the north viaduct should be reduced, it had been thought at the time of the design, 5 years ago, that the beams might be made in a local factory and delivered to the site. The beam size chosen corresponded to the factory lines available at that time. Mr Henry had commented on how simple the construction was to carry out.

Mr Goldstein, in reply, mentioned that several speakers had queried the necessity for a skew bridge. Mr Finch had already replied as to why the line had to be skew and it remained to state why the bridge itself should be skew. Once the line was decided as a skew one, then, with a narrow bridge, it would have been possible to "swing" the piers, resulting in a square bridge of somewhat larger span (300 ft). The appearance at the bank might be somewhat awkward and, clearly, the obstruction to flood-waters would be greater.

210. When it was considered, however, that a second bridge would be built alongside the first, resulting in a total width of pier of about 100 ft, it could readily be seen why the bridge had to be constructed on a skew basis.

211. He would also mention a further fundamental point on the general question of skew *versus* square. Engineers should always vigorously press the case for a square bridge. Nevertheless, in the last analysis they must remember that the purpose and function of a bridge is to carry a road across the river. Ultimately it is the road that is important. The road user, i.e. the general public, who is the engineer's client, would not thank him if, through an effort to simplify the bridge by making it square, the resulting road was not as satisfactory from the alignment point of view (e.g. sharp curves, etc.) as would have been the case had the bridge been skew.

212. He agreed with Mr Finch's estimate of the extra cost involved by making the bridge skew.

213. In § 150, Mr Adams had referred to a fundamental point on the question of post-war construction. Mr Goldstein would have thought that in the anticipation of difficulties, care in supervision, and construction, there was really little reason why practice should have changed. He agreed that construction had perhaps become more complicated and thought that this question ultimately must be decided by harsh economics. He would refer to this point in detail in his reply to Mr Henry; but complications *per se* should be avoided and it was the engineer's job to see that they were, unless of course considerable economies resulted.

214. He agreed with Mr Adams's point in § 151 that double cantilevers of this kind tended to have a flatness of a line at the centre. In this instance the soffit of the suspended span was in fact curved (the depth varied from 6 ft to 6 ft 9 in.) but the radius of curvature was considerably greater than in the case of the cantilevers. It should be remembered that notwithstanding the functional solution of a straight soffit in a suspended span, the presence of the scarfed joint at its ends would often result in a larger depth at the end of the beam than the centre, if too much local weakening was to be avoided.

215. The awkward area just below the deck, mentioned in § 152, was a vexing problem. It was virtually impossible to find any solution which did not present an awkward appearance when viewed from *some* angle. The optimum solution would probably be to make that part of the pier a round shoulder, which would be unsymmetrical and accept the added complication of formwork and construction.

216. He thought that the reason why Clifton Bridge was "stiff" with regard to vibration lay in the cellular nature of the end-spans and cantilevers.

217. He entirely agreed with Mr Henry (§ 157) regarding the question of grouting. Recent research work tended to indicate that neat cement grout might be preferable. At the time (and, indeed, even now) opinions were very divided. If sand were to be used then an improvement would be to ensure its fineness, if necessary by further sieving. While he agreed with Mr Henry that only tendencies would be observed by strain

measurements (§ 158) he doubted if the sideways movement mentioned by him was relevant in this connexion. Clearly, temperature strains would tend to mask strains caused by stress (as would creep and other strains); but by the proper positioning of strain-gauge positions a number of these variables could be analysed and eliminated.

218. Referring to § 160 Mr Goldstein thought that Mr Henry was probably not aware of the actual history behind the fluted face; he was certainly not quite correct in his statements. Originally the spandrel faces of the bridge were specified as plain faces to be cast against lined formwork. During early discussions following the letting of the job, the contractor expressed some concern over the possibility of obtaining really good plain faces in this way. This difficulty was accepted and stringent workmanship control, with regard to caulking of lining at joints, etc., would have been required but for the following.

219. At the time, the contractor, Mr Parry, and Mr Goldstein had visited Germany briefly to examine the concrete faces which had been obtained on similar bridges there. The bridges examined had been cast using approximately 4-in.-wide rough-sawn vertical boards for all forms. This formwork was extremely rough and intentionally so, and it resulted in 4-in.-spaced boardmarks being quite obvious, because of the fins left on the concrete, the non-planar nature of adjacent boards, etc. No rubbing down had been attempted. Although, therefore, the resulting face had poor texture, its appearance—certainly from any distance—was not unattractive as a result of the regular nature of these boardmarks. The shuttering used was, of course, extremely economical. The contractor had pressed him to accept this kind of formwork in lieu of the plain face-lined formwork, but he had not been prepared to do so. He approved the principle of using vertical boardmarks only and suggested that these should be featured by a fluted type of board. This would result in a good texture of concrete face, i.e. avoidance of air holes, fins, etc., and still feature a regular pattern resulting in a mat surface. At that stage Mr Parry had suggested that the formwork be built by using spindle-shaped cutters; this resulted in a most acceptable type of form, and, through Mr Parry's detailed formwork design, a quite economical price.

220. Referring to §§ 162, 164, *et seq.*, Mr Goldstein thought there was much in what Mr Henry had said on the question of design simplification. For himself he would much prefer to design a simple structure, even, if necessary, using more materials to do so, provided always that the resulting labour savings would ensure a more economical job to the client. Designers, however, faced a considerable difficulty in this respect. With the present system of tendering it seemed to him that contractors did not always study the drawings as carefully as they did the Bills of Quantities. Accordingly, simplicity of design was often not reflected in the contractor's prices. The classical case was the normal column foundation where the designer had a choice of providing the usual trapezoidally-shaped design involving shuttering on the sloping faces or, alternatively, providing a rectangular thicker base involving no shuttering but more concrete. With a trapezoidal base the designer frequently faced criticism on site where the contractor's agent would state that it would have been much cheaper to have designed the rectangular base (although not prepared to provide the extra concrete and materials without further cost). In fact, Mr Goldstein agreed with that criticism, but if the designer were to incorporate a rectangular base and bill it accordingly with its greater concrete quantities, the result as a rule was a more expensive base! How could contractors expect a simplification of design when time after time their pricing of the resulting increased materials made such a design more expensive than a complicated one? The solution lay probably both with the contractors and the designers. The former should use the detailed drawings to a great extent in pricing and reflect accurately both complications and simplicity: the latter might perhaps draw these factors to the contractor's notice in a more express way in preparing contract documents.

221. He agreed with Mr Adamson's point in § 169 about the necessity for an ample resident engineering staff and the fact that the balance in that staff, i.e. numbers of

engineers as against numbers of inspectors, would differ from that for work of a more traditional nature.

222. Similarly, he tended to agree that it was unfair to expect the Resident Engineer's staff to carry out research on the structure. He would continue trying to do as much research work as possible using resident engineering staff, but obviously in the event of conflict between the interests of the job and the interests of research the former would take precedence. A happy solution would be the incorporation of a precisely defined research programme, to be carried out by the contractor, in the contract documents. He doubted, however, if this was practicable since, apart from the difficulty of defining a precise research programme *a priori* without interfering too much with the job, the client might have different views about the expenditure in this way.

223. In reply to Mr Rowe, no measurements were taken which could lead to an evaluation of actual prestress losses and it would be difficult on a job of this nature to do so. The actual calculated losses in the case of the cantilever and end span varied, depending on which cables were considered. For some cables they were less than 10% due to the fact that stresses in the concrete leading to creep affecting those particular cables were very small.

224. Mr Rowe was quite right (§ 174) that there had been creep in the control tests on small Perspex beams. The Authors' statement referred to was, in fact, a misstatement in the sense that it had been intended to say that *differences* between creep in the control tests and creep in the actual tests on the model were insignificant.

225. Similarly, referring to Mr Rowe's point in § 176, the Authors had intended to say that methods to calculate end block stresses were readily available. Mr Goldstein agreed that recent tests showed that almost all of them had errors in various degrees. His practice on end-block design had for a considerable time been to use these methods but actually to position the reinforcement both to provide a greater intensity of steel and to extend the area over which this steel was provided.

226. Replying to Mr Galbraith's comment in § 178, he doubted if in fact higher concrete working stresses would be appropriate. It should be remembered that for "under-reinforced" beams (and such beams were obviously desirable in bridgeworks) the post-design load behaviour of the beam was important and an increase in concrete design stress did not necessarily lead to increased load factors.

227. The waterproofing weakness at expansion joints referred to by Mr Webster in § 191 was a general weakness in most bridges. The best that could be done would be to provide a fairly foolproof method of sealing the waterproofing against the joint. Mr Goldstein agreed that the traditional key in construction joints might be undesirable. The pattern of the minor cracks at construction joints in the reinforced concrete section of the work (vertical walls of abutments) tended to confirm this and a butt joint would be preferable.

228. Sir Allan Quartermaine had raised two interesting points on the question of appearance. With regard to the junction abutment Mr Goldstein agreed that this was a little wider than was necessary from purely functional reasons. However, the reduction in width that was possible—once the basic design of flexible columns and rigid viaduct supports was accepted—would be quite small and the resulting width would certainly be considerably greater than the viaduct piers. The reason why the edge wall of the junction abutment had to oversail the spandrel was because the edge flexible column was near the spandrel and for inspection, etc., additional room was desirable to give some clearance between the edge of the column and the inside of the wall. This clearance was not very much anyway (see Fig. 21). The history of this abutment was interesting. The original design envisaged the splitting of the two different kinds of bridges, i.e. approach spans and main bridge, into two separate structures reflecting their different nature. The Royal Fine Art Commission doubted the validity of this aesthetic treatment and he was interested to see that Sir Allan (who, he believed, was not on the Commission at the particular meeting when this was discussed) seemed of similar opinion. The junction abutment was, therefore, "repressed" by reducing it in size to the present size and by oversailing the fascia over the edge wall. Through the continuous

fascia and a relatively feature-free junction abutment, it was thought to "integrate" the two structures and, in fact, not to separate them as was thought by Sir Allan. Similarly the spandrels of both main bridge and viaduct were made plane for the same purpose. This had received the approval of the Royal Fine Art Commission; but Mr Goldstein agreed with Sir Allan that there was probably still scope for further improvement. It must, however, be remembered that to avoid a conglomeration of cable anchorages at the end-span end block some projection of the end block from the flexible column was desirable. This, together with the need to support the viaduct, would generally result in a fair increase of width in the abutment compared to the piers. If it were desired to "sweep" the end span into the approach viaducts, a different kind of pier altogether would be required. Mr Goldstein was adopting just that approach on another current job where that pier was hinged above the base and could accommodate movement due to temperature. The pier would still have to be a little wider than the others but this could be accepted. On Clifton Bridge, however, owing to the skew nature of the bridge, this kind of articulation presented severe difficulties.

229. He thought that the junction between suspended span and cantilever (§ 200) was not really "masked" as such. Owing to the presence of prestressing cable anchorages an additional width of end block was required at these positions, and it is this additional width which might have given the appearance of masking. This was one of the vexing points referred to in the Paper in connexion with the type of scarfed joint obligatory with a suspended span.

230. Referring to Mr Lyle's contribution, he thought that there was some misunderstanding between him and Mr Lyle as to what E actually represented in the case of concrete. The difficulty here was profound. Pure elastic Young's modulus hardly ever existed in the case of concrete, particularly when the stresses used were at all significant. Creep commenced virtually as soon as loading was applied and, accordingly, the measured value of deformations leading to an evaluation of E depended very much on both the rate of loading and the time intervals between loading and measurement. That was the reason why different values of Young's modulus had been used. The Authors' effort had been an attempt at intelligent anticipation of time of measurement. First, in the case of vibration characteristics, it was likely that for those effects the values of Young's modulus would be the nearest approach to the purely elastic one. For this reason Mr Goldstein thought that a higher value should be taken when considering vibration and, accordingly, the figure of 6×10^6 was used. In § 53 a lower value was used on the basis of "an equivalent Young's modulus" in lieu of using a pure Young's modulus and allowing for creep effects separately. In § 108 there was no intention to make a precise comparison. If Mr Lyle preferred to use 4×10^6 again he would have no objection to making the comparison on that basis. In this instance the time between loading and measurement was reasonably small and whether the value of 4×10^6 or 5×10^6 was used was of relatively little importance. It should be remembered that the whole argument would break down if the point were put that full-scale tests simply determined the value of effective Young's modulus! It was, unfortunately, not possible to make rigorous stress comparisons because of this difficulty of obtaining purely elastic strain in concrete.

231. The above point was, in fact, brought out by Mr Lyle's own results. Mr Goldstein suggested that the difference between the codes and Mr Lyle's assumed figure and, in fact, the measured figure, had been largely due to creep.

232. He would not agree with Mr Lyle that the value of E was any more important for prestressed work than for other work. In fact, there was considerable argument to suggest that E was less important in prestressed work than, say, reinforced-concrete work, when stress evaluation only was considered. As Mr Lyle would know, the question of modular ratio in reinforced concrete had bedevilled designers for a very long time. Mr Goldstein doubted if Young's modulus was related in any particularly finite way to the cube strength of concrete.

233. Similarly, he was extremely doubtful if any tests for E on laboratory specimens would be at all useful for deflexion calculations of the prototype. Not only is creep

such an important masking factor but the actual size and shape of the prototype would affect E and creep when compared to a standard laboratory specimen. In a bridge built some time ago he had attempted to relate measured values of E on a laboratory specimen to strain readings on the prototype, and the attempt had proved fairly conclusively that no particular correlation of the two values existed.

234. As a general principle he would try to avoid using design techniques which were highly sensitive to the value of E . In normal stress evaluation this problem did not arise; but in cases such as quoted by Mr Lyle, where the ultimate load was of predominant importance, it was necessary to be extremely cautious in the assessment of E , high concrete cube strengths notwithstanding.

235. The following additional data had recently been obtained.

236. Expansion joint readings and level readings had been taken on 7 July, 1959. The ambient temperature was 85° and several weeks of very hot weather had preceded the taking of readings. Since the previous reading on 16 March, 1959, expansion joint No. 1 had closed by 13/32 in. on both sides. Expansion joint No. 2 had closed $\frac{1}{2}$ in. on the west side and 5/16 in. on the east side. Expansion joint No. 4 had closed $\frac{5}{8}$ in. on the west side and 9/16 in. on the east side. The different closing between the west side and the east side was probably due to differential temperatures in the structure. In view of the hot period prior to taking readings it was fair to assume that the structure as a whole had "heated up" as distinct from daily variations of temperature. The actual movements which had taken place agreed reasonably well with calculated values for approximately 25° increase in temperature.

237. With regard to the level readings a difficulty had arisen because the bench mark, separate from the structure, had been "lost". Readings had, therefore, been taken on the bridge only and an assessment had had to be made with regard to deflexions using the "fixed" points of the bridge as level datums. This was not entirely satisfactory because differential settlements between these fixed points had occurred, amounting to a maximum of $\frac{1}{4}$ in. (there were eight fixed points altogether, i.e. two abutments and two piers, each having points on the west and east side).

238. Using an "averaging-out" process the following deflexions were obtained as having occurred since the initial calibration reading carried out on 29 April, 1958:

	<i>West side</i>	<i>East side</i>
	<i>inch</i>	<i>inch</i>
Centre south end span	+1/16	-1/16
Centre north end span	—	-3/32
End of cantilever (at suspended span) south	+9/16	+5/8
Centre of suspended span	+15/32	+3/8
End of cantilever (at suspended span) north	+5/8	+11/16

Note: + denotes downward deflexions.

239. These deflexions were probably due to losses in prestressing forces combined with creep effects. They were of the right order. Of interest was the upward creep deflexion of the suspended span, which amounted to $\frac{1}{8}$ in. on the west (footpath) side, and to 5/16 in. on the east side. The difference was due to the considerably greater permanent compressive stress on the east side edge beam.