Discussion.

A film provided by Dr. BRADFELD, showing various phases of the construction of the Sydney Harbour Bridge, was exhibited.

Mr. Anderson. Mr. DAVID ANDERSON felt that the structure described reflected the highest credit upon the members of The Institution. He found it very difficult to offer intelligent criticism or to bring forward points for discussion regarding a bridge which was unique. It far exceeded in magnitude the usual works of engineers in this country, and he only wished that Sir Benjamin Baker could have added some thirty years to his life and been present that evening to contribute to the discussion, as he was one of the few British engineers who had had to deal with a comparable structure.

An engineer who had been to Sydney and seen the Sydney Harbour bridge, and who immediately afterwards went on to New York and saw the Washington bridge, had remarked to him that the Sydney bridge appeared to be too big for its surroundings, while the Washington bridge, although a very much larger span, seemed to harmonize with its surroundings. There might be other engineers present who could say from their own experience whether that remark was correct or not. If it was correct, he thought that it should be borne in mind, at any rate by those who might be happy enough to have to undertake in the future a similar large structure.

Mr. Freeman, on p. 156, stated that he had not got out a suspension-bridge design because he was of the opinion that it would be far more costly than any other type, and quoted some figures from the tenders submitted in justification of that opinion. He felt, however, that Mr. Freeman had not made quite a fair comparison, as he had compared the tender of one firm for a suspension bridge with the lowest tender submitted for an arch bridge, which was by another firm. It would seem better to compare the tenders of Messrs. McClintic Marshall Products Company for a suspension bridge and for an arch bridge. Their price for the suspension bridge was £6,047,000 and for the arch bridge slightly more, £6,053,000. Both figures, of course, were well above the accepted price, but they did show the costs of the alternative designs as estimated by the same firm. The general tendency nowadays was to adopt the suspension type for large spans, and he would like Mr. Freeman to say a little more on that point, as he felt it was hardly fair to the suspension type to have such a stigma put upon

it by such an authority as Mr. Freeman. He would also like Mr. Mr. Anderson. Freeman to say whether he would qualify that opinion if the railway tracks were to be located centrally between the trusses, instead of on each side of the trusses as at Sydney.

He would like to ask Dr. Bradfield why an international competition had been indulged in. Mr. Freeman stated in his Paper that it had cost Messrs. Dorman, Long and Company £20,000 to prepare the designs they put in. Five other firms competed, and, if they had spent anything like £20,000 each, then £100,000 worth of calculations had been put into the waste-paper basket, and he thought that was a pity. He would like to know why the usual procedure was not adopted and the work given to a consulting engineer to carry out. Mr. Freeman, the consulting engineer who eventually designed the bridge, gave in his Paper a number of figures of the economies which he would have adopted if he had had a free hand. He assumed that Mr. Freeman meant that if he had been the consulting engineer called in by the Government of New South Wales he would have adopted all the economies that were set out in his Paper. Those economies totalled £725,000. They might not, and probably could not, all be cumulative, but even if Mr. Freeman had altered his views slightly, even if some of them had been discarded, and even if the cost of the structure had only been reduced by £500,000, he thought Mr. Freeman would have been very pleased indeed to design the bridge for a fee of £500,000, and the cost to the Government would have been no greater.

Many of the details were of particular interest to him, because during the early stages of the Sydney bridge Messrs. Dorman, Long and Company had built the Tyne bridge at Newcastle,¹ which had been designed by his firm. It was, of course, a baby compared to the Sydney bridge—the late Sir Hugh Bell had considered that it was only 1/27th of the Sydney bridge, because it was about one-third of the span, one-third of the weight, and one-third of the width, but Mr. Anderson thought that was rather unfair to his bridge, which had a span of 531 feet, and was the biggest arch span at that time in Europe. Messrs. Dorman, Long used the Tyne bridge as an understudy to the larger bridge, and he noticed that many of the details were similar. The use of wire rope for erection, the stretch that was likely to occur, and the tensioning of the wire ropes to equal loadings were tried out on the Tyne bridge. The jacking arrangements at the centre of the arch for the closing operation, together with other details, were also similar. There was, however,

¹ David Anderson, "Tyne Bridge, Newcastle." Minutes of Proceedings Inst. C.E., vol. 230 (1930, p. 167.

Mr. Anderson. one considerable difference. In the case of the Tyne bridge, no creeper crane was used; derrick-cranes were employed in "leap-frog" fashion. A 5-ton crane was erected and was used to put up a 20-ton crane which did the actual erection of the members; the 20-ton crane then picked up the 5-ton crane and planted it in front, and so on. It seemed to him that the creeper cranes used at Sydney must have set up heavy bending stresses in the chords on which they ran, and in addition he fancied they must have been very expensive tools. He would like to ask Mr. Freeman or Mr. Ennis whether, if they had to build the Sydney bridge again, they would use creeper cranes or some other device, and if they could say what those cranes had cost.

Dr. Bradfield had stipulated the use of coke concrete in the roadway foundation, which was stated to contain 0.5 per cent. of sulphur. He thought that even that proportion of sulphur could do very considerable damage in the course of years. He had seen coke concrete cut out, and the corrosion it could set up was very haphazard but considerable; he would like to ask why some other material was not used. He could understand the temptation to use coke concrete for the sake of lightness, but even then the weight of the steel troughing, asphalt surfacing, and coke concrete amounted to 116 lbs. per square foot. The reinforced-concrete decking adopted by Mr. Ralph Modjeski, M. Inst. C.E., for the Camden Bridge, Philadelphia, weighed only 108 lbs. per square foot, and he thought that it provided a much more durable structure and was, as far as he could ascertain from the published figures, designed to carry the same load. Going still further, from the point of view of lightness, a timber roadway capable of carrying the same load could have been adopted. It would weigh only about 50 lbs. per square foot, and he would like Dr. Bradfield to say whether he had ever considered a timber decking or whether he had excluded it entirely because of the fire risk. He would like to ask why footpaths were provided—nowadays everyone rode either in motor cars or in public buses. The footpaths were 10 feet wide, and were designed to take 100 lbs. per square foot live load, whereas the roadway was designed to take the same distributed loading of 100 lbs. per square foot, though of course there were concentrated loads as well, and the railway track loading also amounted to about 100 lbs. live load per square foot. Why were the footpaths loaded so heavily? At Philadelphia, with a span of 1,750 feet, 50 lbs. per square foot was used; would that not have sufficed for Sydney? He had made a rough calculation of the cost involved in providing those footpaths, and it appeared to be about £170,000, allowing for the weight of

steel in the trusses, the weight of the footpaths themselves, and a Mr. Anderson. proportion of the foundations. They were a somewhat expensive luxury, and perhaps Dr. Bradfield would be able to explain why they were so wide and why such a heavy unit loading was assumed. He ought perhaps to temper those observations by drawing attention to a paragraph on p. 348 of Dr. Bradfield's Paper, where reference was made to a scaling-down of all the live loads when the main trusses were being designed. It might be, therefore, that the footpath load was a local loading, and not a loading applying to the trusses and right through to the foundations. Lastly, in connection with the load on the sandstone, that specified by Dr. Bradfield was 31 tons per square foot, while Mr. Freeman cut it down to 13 tons. He had rather wondered whether the first figure could be a printer's error; it seemed an extraordinary loading to put on a sandstone foundation only 5 feet down. Actually the foundations had been carried to 30 feet down; the sandstone had been found to be considerably layered, and in that connection it was stated in the Papers that a spread of the span took place. Dr. Bradfield, on p. 378 of his Paper, mentioned a spread of $1\frac{3}{8}$ inch, while Mr. Freeman and Mr. Ennis spoke in their Paper of a spread of $\frac{3}{4}$ inch. Perhaps one or other of the Authors would deal with that discrepancy.

Dr. OSCAR FABER congratulated the Authors of the four exceed-Dr. Faber. ingly important Papers which had been presented. It was obvious, he thought, that a structure of the magnitude described could not be the work of any one man, and could be carried through only by the collaboration of a large number of experts working loyally together, each exceedingly competent in the portion which he undertook.

He had read Dr. Bradfield's Paper first, and had been sorry to find that in his acknowledgments on p. 401 he had omitted Mr. Freeman's name, and on p. 333 he referred to him as having prepared the detailed drawings. Personally, he would understand that as meaning that the whole work had been designed, and that it was left to the contractors to prepare detailed drawings of a bridge which had been sized with regard to its structural members. He did not think that that was quite the case, because the determination of the sizes of all the structural members was in fact left to the contractors, and there was no doubt that the contractors and their consulting engineers had had to make decisions which were vital to the safety and strength of the structure. He felt sure the omission to which he referred was inadvertent, and that Dr. Bradfield would wish to correct it. Clause 20 of the contract made it abundantly clear that it was left to the contractors to put forward their own design, to calculate the sizes of all the members and to take complete

Dr. Faber.

responsibility for the structure; and that work was delegated by them to Mr. Freeman. Dr. Bradfield, on p. 332 of his Paper, stated that it was decided to revert to riveted lateral connections as shown in the specification. Fig. 6, Plate 7, showed three designs for the transverse section of the bridge. It was true that the design submitted by the contractors had hinged joints at the ends, whereas the specification and the adopted design both had riveted connections, so that in that detail it was correct to say that there was a reversion; but in other respects, which he thought were more important, the modified design put forward by the contractors was wisely and indeed perhaps necessarily retained. The cross section originally proposed contained, he thought, some very undesirable features. The bottom chords consisted of four large webs with top and bottom flanges, and their laterals were of a less depth than the chords themselves; they were therefore connected not to the flanges but to the webs of the chords, so that any lateral stresses which were imposed on the chords would have a very detrimental buckling effect on the webs, which he thought would be quite inadmissible. In that and in many other respects he thought that the bridge as constructed followed very closely the design as submitted by Messrs. Dorman, Long, and was to that extent a considerable improvement on the original proposal.

On p. 374 reference was made to the very large number of stress-measurements which were taken during the construction of the bridge. There were 25,000 gauge points and some 300,000 applications of extensometers; some of the results were given in Table VI, and they appeared to show extraordinary discrepancies between the calculated stresses and the observed stresses. On the top line, for example, the calculated stress was -14.01 and the observed stress -23.39 kips per square inch, which, of course, was an extraordinary difference. Dr. Bradfield suggested that all that difference was due "to the non-uniformity of distribution of stress and bending moment through the make-up of the member, which in many cases is of considerable magnitude," but Dr. Faber did not think that was a sufficient explanation of the enormous difference found. He had himself used the Whittemore strain-gauge, and it was undoubtedly difficult to obtain accurate results when that gauge was applied to gauge-holes and then re-applied on subsequent occasions. He thought that before the results given could be considered as reliable it would be necessary to know the history of the growth of the various readings, whether they were consistent among themselves as the load on the bridge gradually increased, whether they varied uniformly in accordance with the stress, and how they were affected by temperature and other factors, which information

would give a good idea of their reliability ; otherwise it was difficult Dr. Faber. to gauge the value of the tests. He felt that too great reliance could not be put on tests which showed such enormous differences between the calculated and the observed stresses, especially as the observed deflection of the bridge agreed so very closely with the calculated deflection, which made it appear that the actual stresses could not be so far different from those calculated. On p. 378 reference was made to the concrete skewback foundations, and it was stated that there was a spread of $\frac{5}{8}$ inch. Dr. Bradfield wrote : " It appears that the greater part of the spread was plastic in nature, due to permanent set and shrinkage in the concrete skewbacks and the rock under the heavy thrusts applied to the bearings." He very much doubted whether the shrinkage of the concrete could have had much to do with that, because the concrete was nearly 2 years old before the bearings were fixed to the skewbacks, and he felt that practically all the shrinkage must have been completed by that time. It would also be interesting to know whether there was any record of a vertical settlement, because the thrust on the bearings was inclined at 45 degrees, and therefore it seemed reasonable to suppose that if there was a spread of $\frac{5}{8}$ inch there would be a somewhat similar vertical settlement, which would perhaps have been easier to record.

Turning to Mr. Freeman's Paper on the design of the structure and foundations, he remarked that the Author made an interesting point on p. 188, where he showed the modification which he made to the design of the approach-spans by employing two girders on two piers instead of four girders on four piers, thus making the structure statically determinate and very much reducing the weight of the steelwork, piers, and foundations, saving £150,000 by that simple modification alone. With regard to the main foundations, he thought that sometimes it was not sufficiently realized that it was difficult to place a block of 7,600 cubic yards of concrete in such a manner that it would not crack owing to stresses arising from temperature-changes and shrinkage. The heat generated by the chemical action of the cement would raise the temperature of such a block by from 50° F. to 60° F. if the concrete was placed too hurriedly, because there was so little surface from which heat would radiate ; it was therefore important to divide the block into such sections as would enable the shrinkage to be controlled and induce any cracking to occur on planes which would not be detrimental to the strength. Besides the ordinary shrinkage which took place when cement set, there was also the shrinkage due to its cooling through about 60° F. from a raised temperature caused by the chemical action of the cement in setting. His firm had made actual

Dr. Faber.

tests of the rise of temperature in thick masses of concrete in certain works, and they had found that temperature rises of from 50° F. to 60° F. sometimes occurred in a week or even a fortnight after the concrete had been placed ; in the case of big blocks such as those in question, he had no doubt that it would take from 9 to 12 months for the temperature of the concrete to revert to within 5° or 10° of normal temperature. On p. 191, referring to the construction of the pylons, Mr. Freeman truly stated that it was impossible to over-estimate the importance of engineering structures being designed to be æsthetically satisfactory. With that statement he wholeheartedly agreed. He was not so sure, however, whether it was therefore advisable to build extraneous structures which had in fact nothing to do with the duty of the bridge. The cost of the pylons referred to was £750,000. He felt that anything which could be done to make the lines of a structure right both technically and æsthetically was justifiable and indeed necessary, but whether it was proper to spend £750,000 out of a total of £4,250,000 on other structures which had nothing to do with the bridge was another question, and one on which he would be very interested to have the views of the various Authors. The pylons, in addition to their own cost, added an enormous load to the skewback foundations, which in turn had to be made larger than would otherwise have been necessary. If the pylons were necessary, then he thought they had been constructed in the best possible way, and it was of interest to consider why the concrete had had to be placed so much in advance of the granite facing. That was because the concrete was subject to very considerable shrinkage and to a shortening due to its gradually increasing stress as the 300-feet-high pylons grew. That stress and shrinkage would have thrown a very large proportion of the total load of those pylons on the non-shrinking granite facing, and would have caused the facing to be highly overstressed, unless the concrete was allowed to shrink before the granite was fixed. The alternative would have produced a result not dissimilar from that which took place in the large piers of St. Paul's Cathedral, where the ashlar skin, being stronger and more resistant than the rubble core, tended to carry an undue proportion of the load, and then split off with mild explosions from time to time. In Table II Mr. Freeman had given an exceedingly valuable analysis of costs. He thought that Authors who gave such details added greatly to the indebtedness of the members to them.

Mr. Gribble.

Mr. CONRAD GRIBBLE said that in his view one of the most valuable parts of the Papers was the description of the various designs worked out by those who tendered for the work. He wished space could have been found for more information with

regard to those designs, because he felt that the four Papers on the Mr. Gribble. Sydney Harbour bridge would for many years be a classic record of the undertaking of a great and unique structure, and that engineers in all parts of the world would refer to them and would gather from them valuable help in solving their particular problems. He had studied the descriptions of the various types of cantilever, arch, suspension, cantilever-arch, cantilever-suspension, and arch-suspension structures with the object of trying to find out why the cheapest and best design turned out to be the arch structure. There was no doubt that it was the cheapest—the figures proved that—but he would like to know the reasons. It seemed that any bridge which had to be built over a waterway in which no false-work could be placed had to be erected by the process of cantilevering, and therefore it would appear at first sight that a structure of which the cantilevering was a permanent part would be inherently economical, seeing that the necessary apparatus for constructing the bridge as a cantilever would form part of the permanent structure. A very great sum had to be spent in anchoring such a bridge as that at Sydney. On the other hand, the well-known photographs of the erection of the Forth bridge showed the enormous cantilevers balanced and growing up without any temporary work at all, and indicated that in the case of that bridge the erection costs must have been much smaller relatively than those of a structure whose whole nature during its construction had to be different from that of its final condition. The engineers responsible for the Sydney Harbour bridge were very fortunate in the foundation, and they were able to impose thereon, by means of the pylons alone, a load of about 3 tons per square foot, the weight of the pylons on each end of the bridge being of the order of 115,000 tons. Considering the difference between an arch bridge and a cantilever structure, it seemed to him that on the one hand each of the main piers of the arch bridge sustained a load equal to half the dead load and half the live load on the arch itself, in the form of a vertical reaction and a horizontal reaction equal to the thrust of the centre of the arch, the amount of which was inversely proportional to the height of the arch above the springings; on the other hand, the main pier of a cantilever structure had to take, in addition to those vertical forces, the whole live and dead load of the anchor-arm plus the anchorage-force. It appeared to him, therefore, that a cantilever bridge imposed a much greater total load on the main piers, and that might, in many cases, have an important bearing on the selection of the type of bridge. In the present instance it was not of primary importance, and one of the greatest loads imposed on the foundation was that due to the pylons, which were not an essential part of the structure. It

Mr. Gribble. was not quite fair to ask the Authors to give complete details about any alternative design that they might have proposed, but he thought that some further information on the matter, particularly from Mr. Freeman, who had prepared at least three designs for the bridge, would be helpful. In regard to rigidity, which was a vital point, he thought it was true that the deflection of the arch bridge measured in inches was equal to the deflection of other types measured in feet, and that factor might have had some bearing on the economy of the design, in view of the fact that in other types of bridge the excessive deflections would entail increased cost in floor and bracing connections. Possibly—and he thought there was some hint of it in the Papers—that was to some extent a determining factor in the selection of the design. He could only conclude by expressing his very sincere appreciation of the Papers.

Mr. William-
son.

Mr. JAMES WILLIAMSON had first been interested in the question of an arch bridge for Sydney when inspecting in the Glasgow Exhibition of 1901 a rather fine drawing of an arch, which was referred to by Dr. Bradfield in his Paper. That design was prepared by the late Mr. Carl Bonn, M. Inst. C.E. It was a 2,000 feet span arch even bolder than Mr. Freeman's design, and it was very doubtful whether such a huge span could have been successfully built at that time. Then, about twenty years ago, when with the Cleveland Bridge Company, an arch design was considered for the bridge, and still later, when the first invitation to tender was issued, he had investigated on his own account the possibilities of an arch design. He had seen the specification for the cantilever design, before an arch design had been called for, and from previous work it was clear to him that an arch design was probably more suitable. He had worked the matter out at some length, and the type of design which seemed to him to be the most appropriate for the purpose was shown in *Figs. 1*.

That type of arch had not often been built, but there was across the Seine in Paris an attractive arch, the Austerlitz Railway bridge, of very similar type, with the exception that it had three hinges, two at the line of the roadway and one on the crown. A feature of that type of central arch having hinges near the roadway was that the reaction-locus, having to pass through the haunch hinges, which were not at the ends but were some way up the arch, remained at all points closer to the centre line of the arch rib; the bending moments were therefore smaller and a lesser depth of arch at the centre could probably be used, while, instead of running to an enormous height at the ends, the dimensions of all the members were more manageable. It seemed to him that, with that type of main span, the subsidiary arches at the sides indicated in *Figs. 1*

gave a balanced design. With regard to the cross section, the Mr. William-
original call for tenders showed an unsymmetrical arrangement of ^{son}.

the roadways and railway tracks on the deck. It seemed to him at that time that a redistribution should be made, and he proceeded on the basis which was afterwards adopted of the roadway being

Mr. William-
son.

placed in the middle, flanked by railway tracks and then by the footways. The advantage was that, instead of working with two ribs, which required enormous members, it was quite convenient to have four ribs, of which the two outer pairs, braced together on each side, really formed a single rib; there again the subdivision of material led to far more manageable sections and involved much less excess material in the splices than would have been the case with heavy sections of exceptional thickness. The cross girders were suspended at four points, which effected economy of material, but were statically determinate with regard to bending moments. The two pairs of suspension-members supporting each cross girder were connected by triangular yokes with the single main suspension-members, which distributed the load evenly on each of the outer pairs of ribs. The result of those modifications was that the pair of rib members in the haunches leading down to the abutment continued to be of approximately equal size top and bottom, and each took a more or less equal share of the load, so that, having four main ribs instead of two and also two rib members to share the load instead of one at the abutment, the members in the haunches remained of manageable proportions. It was curious that the shape of the ornamental pylons proposed, particularly at the top, was very similar to that of the pylons actually provided. The height of the pylons shown seemed to be fairly appropriate to the general design, although the portions above the deck were only half the height of those erected at Sydney. The method of erection of such an arch (*Figs. 2*) needed very careful consideration. Obviously, the ends of the ribs being shallow, they could not be erected as cantilevers, but—and it perhaps answered the point raised by a previous speaker—in erecting an arch it was not necessary in the first place to erect the roadway with all its heavy weight in the central portion, but only to erect the ribs. The cross-girders of the design in question were very heavy members, and he had proposed to utilize them to form temporary erection-towers on top of the pylons. The anchorage would be effected by means of wire ropes, the method being somewhat similar to that employed at Sydney, and the haunch or lower portion of the arch rib was to be modified into a cantilever by tying it to the top of the erection-tower over the pylons and erecting another tower on the nose, from the top of which it would be fairly convenient to erect a number of panels of the main arch in a single portion standing on the hinges. As they did not have to be handled in the whole width of the arch, but only two ribs at a time of half the weight, it was feasible to lower those ribs down into their arch position by powerful winches and tackle, and then to hold them in place by more permanent wire-rope ties.

The winches, therefore, could be used for lowering one half-sector, Mr. William then shifted laterally to the companion tower on the same abutment^{son}.

Figs. 2.

for the other half-sector, and then changed over to the other side of the bridge for the corresponding work on that side. Having then

Mr. William-
son.

a fairly stable base of considerable length to work on, the further erection would proceed by methods more or less similar to those adopted at Sydney, the difference being that two small travellers would be used instead of one large creeper crane, and as they would have a smaller outreach and much smaller loads to carry, he thought they would have been much lighter and less expensive than the plant actually employed. The maximum weight from the hinge outwards in any member did not exceed 40 tons, and the panel length was about 42 feet, so that, taking account of the smaller loads to be handled and smaller outreach, the cranes would be relatively small. The floor was to be erected by the same method as that actually adopted at Sydney; the cranes would work back, putting in the suspension-ties, cross girders, and so on. Dealing with one or two of the points raised in the Papers, he thought the ingenuity and care shown in the design of the rope anchorages were worthy of admiration. It was notable that the load of about 128 tons taken by each of those rather small cables was transferred to the pin by a round rod, and that the theoretical bearing would only be a line contact. No doubt sufficient tests would have been made to ensure that that was quite satisfactory, but he would like to know whether after application and removal of the load there was any indentation of the main pins or flattening of the round rods. Regarding the tests of model members detailed by Dr. Bradfield in Table II of his Paper, he thought it would have been useful if one or two further explanations had been added. For example, he noted from the Paper by Messrs. Pain and Roberts that when the struts were tested under compression lateral bending was added in most cases to correspond with that occurring in the actual member. That important factor should be made clear in giving results of crushing strength, to enable them to be interpreted correctly. It seemed to him, moreover, that possibly the areas of sections given in the Table in question were rather nominal areas and not very closely related to the actual sections of the members themselves. In the case of the model member "0," representing diagonal 9-6, which was tested in tension, the area was given as 19 square inches gross, so that the stated stress of 61,300 lbs. per square inch in the actual member would be taken on the gross section of the member. A stress on a tension member was usually reckoned as a stress on the net section, and it would, he thought, have been more useful to give the stress in question on the net section. With regard to the shape of the Sydney arch, he took it that the very deep form at the ends had really been dictated by the convenience of the depth for purposes of erection and that, if some other equally feasible method of erection could have been adopted for an arch with parallel or

crescent-shaped ribs, one of those forms would probably have been preferred. Mr. William-son.

Mr. H. C. JENKINS said that at the beginning of 1927 he became Mr. Jenkins. responsible for watching, on behalf of the New South Wales Government, the supply of material for the Sydney Harbour bridge except that of the main pivots. There was one important point in that connection which he would like to put before the meeting, and more so as Mr. Freeman in his Paper had referred to the very satisfactory character as a whole of the material. He would endorse that statement. The quality of the work offered by the contractors was very high. The uniformity of all their plates was remarkable, and the notably high quality even of the ordinary steel impressed him very much. Furthermore, whenever some small matters needed attention, the way in which the entire staff were evidently keyed up to making the best job they could of the Sydney Harbour bridge was most helpful. Fortunately, he had had the assistance of an inspector who had been attached for many years to the N.S.W. Government office, Mr. H. D. Marsh, a man of very great experience and good judgment, so that, with regard to the ordinary steel, his own duties had been mainly supervisory, but the inspection of the silicon-steel promised a little more variety. The percentage of silicon in the steel was only 0.2 per cent., which very much reduced the various problems connected with it, but at that time there was not much published as to the precise behaviour of steel of the particular composition in question, when rolled into thick plates. It contained 0.38 per cent. of carbon, 0.2 per cent. of silicon, and 0.9 per cent. of manganese, the manganese being evidently a very important agent in giving the tensile qualities to the steel. The maximum limits for sulphur and phosphorus were 0.05 and 0.04 per cent. respectively (0.06 for Acid P), but the actual content of each was generally only about 0.03 per cent. or even less. It would be seen, therefore, that the quality of the steel was very high indeed. The specification made a general provision for normalization, but a temperature of only 750° C. was specified as that to which the steel was to be raised, and that only for half an hour, before it was freely exposed to cool in the air. From the outset he had felt a little anxious about that, as from his own previous research experience with steel it did not seem quite high enough, but it could be assumed that the specification had been settled with all possible care, and, of course, his first duty was to the specification and his second was to watch. That feeling of uncertainty, however, was greatly accentuated when a member of the contracting firm who had called upon him told him that it had been decided at Sydney that it would not be necessary to normalize except in cases where the plate had failed to meet the

Mr. Jenkins. prescribed tests, and official confirmation arrived from Sydney soon afterwards. Under a sliding scale, the minimum elongation specified for thick plates might be only 14 per cent., and the reduction of area of fracture on the tensile tests only 24 per cent. with 2½-inch plates, of which there were a fair number in the lower boom, and under the original specification no shock test was required. The specification did include, however, what appeared to be a very severe cold-bending test both ways on the plates. He felt it his duty to ask his staff to watch intently for the least abnormality. Nothing occurred at first, but he was informed by that staff of a certain amount of experimental work taking place, more of an annealing than normalizing character, which was going on at the works, and which again had simply to be watched, whilst smaller and thinner plates were being successfully rolled and were showing up very well. Ultimately a start was made at Redcar with some of the very heavy web-plates of the lower boom, with crutches cut out of their ends. Six ingots had been taken from one cast, of which one had been rejected owing to surface imperfection and one withdrawn. The four plates, one from each ingot, had been tested and passed, and three of them had been despatched. The fourth was being turned over on the bed by means of the crane; it had not been lifted off the ground, but was just being toppled over so as to fall upon its side when it fractured and the crutch-piece broke off. There was a rapid appeal to the testing machine again, but it was found that the steel had passed the specification; there could be no doubt about that. The analysis showed 0·415 per cent. carbon, 0·176 per cent. silicon, 0·81 per cent. manganese, 0·031 per cent. sulphur, and 0·031 per cent. phosphorus. The record of the rolling had been quite satisfactory; there had been no rolling between 980° C. and 880° C., the plate had been finished at 770° C., flattened at about 500° C., and then more slowly cooled. No permanent set could be seen up to 16·7 tons per square inch, when an Admiralty testing machine was being used; the permanent set at 20 tons per square inch, taken by extensometer, amounted to 0·008 inch on 8 inches. New test bars in the ordinary testing machine gave drop of beam at 21·5 tons per square inch, with ultimate tensile strength 40·5 tons per square inch, elongation 23 per cent. and reduction of area 32·9 per cent. The bend tests were quite satisfactory, and a test bar that he had had cut out from the plate by an oxy-acetylene jet, as used to cut out the crutch, gave unexpectedly good results, 41 tons per square inch ultimate strength, drop of beam at 22·5 tons per square inch, elongation as much as 9 per cent. and contraction of area 12·3 per cent., the only objectionable feature present being that it had an austenitic band

just round the outside. The remarkable fact about the plate was Mr. Jenkins. that Izod tests, to which he had immediately resorted, showed only 10·5 foot-pounds on the specimen in the direction of rolling and 12·6 foot-pounds across it, instead of 25 to 35 foot-pounds, which that class of steel might be expected to stand. In the crutch of the broken plate, something that might have been a slight crack from the oxy-acetylene cutting was visible, and it was at once decided to use coal-gas burners instead. The contractors called in his old friend Mr. F. W. Harbord to advise them on the matter, but Mr. Jenkins had naturally to investigate the whole situation for his own principals, the Government of New South Wales. Here he would gratefully acknowledge the assistance of a draft Report on high elastic limit mild steels, by Dr. Harold Moore, of the Woolwich Research Department, which that Department placed at his disposal.¹ Before that investigation, however, could be more than started, another plate 2 inches thick, of similar description to that already mentioned, slipped out of the crane sling when only 4 feet off the ground, while being turned over, and fell flat on to a line of sleepers. The crutch end itself touched nothing, being simply overhanging, but it was shaken off, and on the most careful examination of the fracture no flaw could be detected. Again, however, the Izod values were low, from 12·1 to 13·5. The plate analysis in that case was 0·38 per cent. carbon, 0·182 per cent. silicon, and 0·85 per cent. manganese, while the phosphorus was as low as 0·02 per cent. and the sulphur 0·029 per cent. The test results were 39 tons per square inch ultimate strength, with 20·5 per cent. elongation and 33·5 per cent. contraction of area. Izod test-bars were now made from a series of samples of plates which he carefully selected from the records of the plates of similar analyses but of different rollings, different thicknesses, and after differing periods of heating to 800° C. before being freely exposed in the air to cool (normalization), as being in better agreement with the evidence from the cooling curves as to what was needed than was the temperature of 750° C. of the original specification. Those important test-bars were broken in the presence of Mr. Freeman and himself; the results were most interesting, for he was able to report to his Government that the plates, when not thicker than 1 inch, did not need to be normalized (rolling finished at 770° C.), but that plates of 1½-inch or more in thickness certainly required it; 2-inch plates indeed needed a preliminary

¹ This Report was subsequently expanded and incorporated in Mr. J. A. Jones's Paper, "High Elastic Limit Structural Steels," Journal Iron and Steel Institute, vol. cxx (1929), part 2, p. 127.

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Mr. Jenkins. heating to 800° C. for at least an hour before being freely exposed to the air for cooling. Those requirements were embodied in the new Specification, in which plates of 1½-inch thickness were included in the exemption accorded to those of 1 inch, and all plates of 1½ inch and thicker received the full heating of 1 hour to 800° C.; the new specification also reduced the maximum carbon to 0·38 per cent., a figure which had rarely been exceeded. The contractors also had found it possible to satisfy all tests with 0·9 per cent. of manganese, although their margin with that element was somewhat increased. The possibility of embrittlement of plates of the composition now under discussion, low in both sulphur and phosphorus and yet showing good elongation, large reduction of area at fracture, and ability to pass severe bending tests both with and across rolling, was his justification for calling attention to the matter, which had fortunately arisen at a time when it could be properly dealt with, and which, as Mr. Freeman's remarks on p. 171 would indicate, might well be important to other workers. How far the need for normalization was due to the use *per se* of thick plates, or how far it was due to the need of a greater ratio of reduction of thickness by the rolls than the sixteen times required by the Specification "where practicable" (and which in the thickest plate was already one of fifteen times) could not be more fully investigated; the facts certainly suggested that larger ingots would permit thicker plates to be rolled without embrittlement. At the same time, however, it was clearly necessary to keep a watch on even the thinner high-tensile mild steel plates by means of occasional tests of the Izod or Charpy type if full advantage were to be taken of them, and he had to confess that he was surprised by the little appreciation he found in the "mild steel" world (as distinguished from that of research) of the value of those tests used within their proper limits. In conclusion, he would remark that the Admiralty "D" steels evidently had a wide use on land as well as on sea.

Mr. Freeman. Mr. RALPH FREEMAN exhibited some lantern slides showing the Sydney Harbour bridge during and after construction.

Sir Harley Dalrymple-Hay. Sir HARLEY DALRYMPLE-HAY felt that the members had before them the story of a really great work. It had been well conceived, well designed, well calculated, and, last but not least, courageously erected. Dr. Bradfield, the Chief Engineer, had written a very comprehensive Paper which dealt with the bridge and with the ancillary works on either side. Without wishing to discredit Dr. Bradfield in the least, he had no hesitation in saying, after reading his Paper carefully, that he had thought first of all, "Well, I will span this with a cantilever bridge"; that he had prepared an outline design for that purpose; that he had then gone on a tour of

investigation, and had eventually arrived at an American bridge-Sir Harley Dal-
yard where the Hell Gate bridge was being fabricated. That was rymple-Hay.
a very beautiful bridge, and he could quite believe that an engineer wishing to bridge Sydney Harbour would be greatly attracted by that design; but it could do no more than provide an idea. Probably Dr. Bradfield had passed on that idea to Mr. Freeman, who had drawn out, calculated, and erected the wonderful Sydney bridge. Mr. Freeman had referred briefly to Mr. David Hay's visits to Australia, when there had been some question whether there should be a bridge or a tunnel. Dr. Bradfield was silent on that matter, and he would like to hear the reasons why the bridge had been adopted. He thought that anybody who had studied the Papers would agree with the decision, because the ground was not fit for a tunnel, and, even if it had been, the presence in a passenger-carrying tunnel of motor vehicles giving off foul gases was not, to his mind, either pleasant or proper. He thought that had the most recent experience about the cost of ventilation of tunnels been available at the time, nobody would have ever considered the question of a tunnel; they would have decided immediately on a bridge; and he made bold to say that there must be many people in Liverpool at the present time who wished they had a bridge instead of a tunnel. With reference to the railways which had been laid out by Dr. Bradfield on both sides of the Harbour, he would like to ask him why they had not been located in the centre of the bridge, as according to Mr. Freeman the track on one side only might be fully loaded, and a tremendous rocking action would consequently take place between the one truss and the other. If the railways had been put in the centre of the bridge, and the roadway carried on the outer sides, he thought that Dr. Bradfield might have spent his £50,000 in sway-bracing, which would not, in that case, have been an objectionable feature of the design. In Sir Harley's judgment the sketches that Dr. Bradfield had made looked much better than the bridge as constructed, because, owing to an amendment made during the preparation of the drawings, the pylons, which had cost the colossal sum of £700,000, did not really beautify the bridge, as they left an ugly gap between themselves and the vertical members of the arch spandrels. Had Mr. Freeman considered whether it would have been possible and economical to have built a crescent-shaped two-hinged arch of the type which Messrs. Dorman, Long had built to the designs of Messrs. Mott, Hay, and Anderson and Messrs. Coode, Son and Matthews across the Tyne? ¹ To his mind it was a most beautiful design, and it

¹ David Anderson, "Tyne Bridge, Newcastle." Minutes of Proceedings Inst. C.E., vol. 230 (1930), p. 167.

Sir Harley Dalrymple-Hay. was one of the most recent bridges. He expected Mr. Freeman would say "Oh, yes, that is all very well for a span of 530 feet, but here we are dealing with 1,650 feet span; a crescent bridge of the type you have mentioned would obviously have the bulk of its weight concentrated in the centre of the span (and that is not the case with a spandrel bridge such as that at Sydney, where the bulk of the weight is nearer the shore), so that the cost of erection would be quite out of the question." That was one point about which he would be glad to have some information.

When he had read p. 189 of Mr. Freeman's Paper he had seen the alarming fact that "the specification defined the maximum pressure on the sandstone rock as about 31 tons per square foot, and showed the foundations for the main bearings sunk about 5 feet into the rock." Further, it was stated that "The rock is a soft yellow sandstone with horizontal seams of clay and shale." He had been really shocked, and had said to himself, "31 tons? Never!" And he had looked further and had seen that the figure actually adopted was 13 tons per square foot. With all respect to the remarkable method of breaking up the foundation of the concrete into a series of hexagonal or rectangular blocks, due to Dr. Faber, he thought that when a monolith was to be made the concrete ought to be poured as one mass. He did not think that the idea of making artificial cracks in the concrete in definite lines was of much value; any concrete would crack, and he would just let it look after itself.

Mr. Frank.

Mr. T. PEIRSON FRANK remarked that unfortunately he had not been able to read all the Papers, but that by Mr. Freeman was full of the most useful information for those who might have to undertake the design of bridges in the near future. Under the heading "Quality of Steel" (p. 169) Mr. Freeman indicated that the third type of steel which required consideration was "A nickel or other alloy steel with appreciably higher tensile strength." He presumed that such a steel, like the silicon-steel to which reference was also made, was rather expensive for general use as the demand for that particular type of material was as yet rather small; a little later, alluding to the silicon-steel, Mr. Freeman said: "A sufficiently extensive demand for steel of higher tensile strength than standard mild steel would enable this steel to be supplied at a price enabling the cost of ordinary bridges and structural steelwork to be much reduced." In the light of that observation, coming from such a source, he thought it was incumbent on those engaged in the design and construction of bridges and structural steelwork to see if they could not make use of a commodity which would give them a more artistic, lighter, and probably more enduring structure. In that connection, he believed that under the Road and Rail Traffic Act

of 1933, section 30, bridge construction and reconstruction in Mr. Frank. England during the next 10 or 15 years would far exceed that which had taken place during the past 30 or even 50 years. If for that work a more general demand for a better steel of higher tenacity could be engendered, it should be possible, according to the Author's statement, to reduce the cost of construction. He thought that the Sydney Harbour bridge might have looked better if a more delicate design employing steels of higher quality had been adopted, but the Author had explained that it had been impracticable in that particular case. He did not know if æsthetic grounds were taken into consideration. On p. 172, the Author said that if the permitted compressive stress on the silicon-steel employed had been 85 per cent. of the tensile strength under the British Standard Specification for ordinary mild steel, that stress would have been 8·9 instead of 7·8 tons per square inch, and a saving of no less than £200,000 on the cost of the bridge could have been made. The next considerable saving which Mr. Freeman said could have been made was mentioned on p. 183, where, referring to the riveting of butt-joints, it was stated that "The workmanship was such that when two faced ends were placed in contact it was impossible to insert a 1/200-inch feeler gauge at any point." It would be generally agreed that the workmanship was of remarkable excellence. On the same page it was stated: "If the butt-joint covers of compression members had been reduced in size the weight of the main trusses would have been reduced by about 20 per cent., and the cost of the bridge by at least £250,000." Those two apparently small but most important factors could therefore have effected a reduction in the cost of the bridge of some £450,000. Then came the large possible saving to which Sir Harley Dalrymple-Hay had referred, the omission of the pylons, which were stated in the Paper to be structurally unnecessary, and which cost £700,000. Those three figures taken together showed that a saving could have been effected of approximately 25 per cent. of the cost of the bridge. That raised the question of how far artistic development was to be permitted. He felt sure there would be general agreement that the best architect for an engineering structure was an engineer. He suggested that it might have been cheaper to have used Indian granite for the pylons, instead of the Australian granite specified, as certain qualities of Indian granite had been offered in England which were excellently dressed, and he believed they might have been obtained in Australia at a reasonable price. The expansion joint shown in *Figs. 20*, p. 187, was ingenious; he would like to know what was the maximum expansion that had taken place. Was the bridge exposed to great extremes of climate? He would also like to know whether there had been any

Mr. Frank.

contraction of the concrete foundations. In common with the previous speaker, he was not quite certain of the utility of the manner in which the concrete had been placed; he would not put it more strongly than that. In conclusion, he noted that the cost of the bridge and approach-spans was approximately £9 per superficial foot. The Forth Bridge had cost about £7, and the recent Tyne Bridge at Newcastle about £8 per superficial foot. In view of the magnitude and difficulties of the work he thought that the engineers, contractors, and all concerned in the construction of the bridge were to be congratulated on that achievement, as well as on the manner in which the bridge had been erected.

Mr. Barling.

Mr. I. C. BARLING congratulated the Authors on their work; the bridge, he said, was a most wonderful structure. There was one point in regard to it, however, which was not referred to in the Papers and had not been mentioned in the discussion, namely, the financial aspect. That was the more remarkable in view of the fact that four of the Authors were employed by the contractors, whose not inconsiderable function it was to make a profit. It had been stated that Dorman, Long and Company had lost £270,000 on the bridge, and that loss was said to have been due to variations in the rate of exchange. He had found from the *Australian Year Book* the rates of exchange which obtained between the year 1926 and the year 1932. Dividing the construction of the bridge into two periods each of 2 years, in the first period the average drop on the exchange was 1 per cent., but in the second period it was 15 per cent. It could be said that, taking half of the job as costing roughly £2,000,000, on the first half £20,000 would be lost and on the second half £300,000, the total loss due to exchange being thus £320,000. When a contractor was making an estimate for work such as that in question and was handling everything, including the making of the steel and the designing of the bridge, he did not think it would be unreasonable for him to expect a profit of 20 per cent., which on the Sydney Harbour bridge would have been roughly £800,000. Taking the loss, subject to correction, as £270,000, it was therefore necessary to account for £1,070,000; the loss due to the rate of exchange, according to his figures, amounted to only £320,000, leaving £750,000 unaccounted for. In one of the Papers a figure of £50,000 was given as the extra cost of labour in Australia. It was perhaps hardly fair to deduct all of that, but he had done so, and that left £700,000 to be accounted for, or between 16 and 17 per cent. of the estimate. There must be something there, he thought, which had not been made clear, and he felt sure that the Authors would explain it. It had been said that Dorman, Long and Company could not get the money over to England, and that

they invested it in wool. He would like to know whether that Mr. Barling figure of £700,000 was due to wrong estimating or to extraneous trading. The Sydney Harbour bridge was a splendid structure, but it had been done entirely by the contractors, without, apparently, an engineer uncontrolled by them. It would be very interesting to know how it was that they had not profited to the extent that would naturally have been expected.

Professor S. M. Dixon remarked that very seldom was so much important work presented at one meeting of The Institution. Referring firstly to the Paper by Mr. Freeman, he would like to thank him for the very clear reasons he had given for a great deal of his work; it was not often that a Paper provided so much real help. He would like him, however, to explain the reasons for the reversed curvature at the ends of the top chord. It was to be noticed in some other bridges, but he had never seen any reason given for its adoption. He could not understand why the excellent concrete of the pylons was covered up with cut granite. The pylons were supposed to be an architectural effect, but engineers had a right to judge of the artistic effect of a bridge if it was designed as an engineering structure. The following dictum of Albrecht Dürer was often quoted as a test of true artistic value: "This, for what it has to do and the way it does it, cannot be excelled." The cut granite had cost £50,000 for "the way it did it," and he could not see that granite hung on in that way by iron clamps made a better job. In the case of the very large bridges in America and of some on the continent of Europe the concrete was left as it came from the form, and the pylons looked extremely well.

He would like to ask Mr. Pain and Mr. Roberts why the calculations were made to such refinement when the data for the design were necessarily rather rough.

He noticed that Dr. Bradfield in his Paper gave a close estimate, and therefore presumably had a design, for an arch bridge incorporating 36·7 per cent. of nickel-steel. It was a very great pity that the outline of that design was not given in the Paper. The most interesting, and certainly the most important, part of Dr. Bradfield's Paper was that concerned with strain measurements (pp. 370 to 376). The discrepancies between the observed and calculated stresses were very remarkable, and, on account of the work involved in making over 300,000 observations, he thought that they required some more discussion, and should not be dismissed merely with the remark that those discrepancies were unavoidable. If the measurements were worthy of being taken, he thought the discussion should show that they were useless and why they were useless, because the stresses in some cases were five times those calculated. The great

Professor
Dixon.

Professor
Dixon.

confidence that Dr. Bradfield must have had in the designer's figures was evident. On that point, he would like to ask Mr. Freeman how the extensometer results on models compared with the calculated results. It might be that the results in Australia had been taken too hastily, as they had been taken much more quickly than those at Hell Gate, and it was recorded that at Hell Gate the observations agreed fairly well with the calculated results. It would be of value to know how the points were chosen, because evidently a good deal depended on that. He noticed on p. 374 that the stresses were calculated on the assumption that Young's Modulus was 30,500,000 lbs. per square inch as determined by tests, whereas it was stated on p. 334 that Young's Modulus as determined by tests was 28,700,000 lbs. per square inch. He did not know why those two values were given, and it might be interesting to have the reason. It had been found generally in experiments on built-up members that the modulus of the member was very much less, certainly on the first tests, and it might be that the tests carried out on the big testing machine would account for that difference. In other words, Dr. Bradfield's tests showed the importance of large testing machines. The engineer could not do without them; he had to make tests on models and, when he found a discrepancy, those tests could be repeated over and over again, not making twenty-five thousand different tests, but making the same test twenty-five thousand times if necessary in order to find out why there was a difference, as if the reason were not known it might be that the stresses which were calculated as being quite safe were really very much greater. However, he thought that the designer's calculations might be considered to be perfectly safe. Dr. Bradfield realized the importance of the extensometer experiments, because he said, on p. 371, that "The extensometer also provided a practical check on various erection details, notably the silicon-steel bolts connecting the anchorage cables to the tops of the end posts." It would be of interest to know how the results of those tests compared with the calculated stresses.

Mr. Gelson.

Mr. W. E. GELSON remarked that the Paper by Messrs. Freeman and Ennis formed a particularly valuable record for engineers who might be faced with similar problems in the future. Discrepancies had always been observed in India between actual and calculated deflections of bridge trusses during erection by the cantilever method, even when the workmanship had been of the highest order; actual deflections had always somewhat exceeded those estimated. One instance of that was recorded on p. 252 of Mr. Everall's Paper on the Attock Bridge.¹ Dr. Bradfield, on pp. 378 and 379 of his

¹ Minutes of Proceedings, Inst. C.E., vol. 230 (1930).

Paper, suggested that the shop lengths of members should be adjusted to provide a camber about 50 per cent. in excess of the theoretically-desired camber to provide a closer approximation of the actual dead-load conformation of the structure. It was a remarkable fact that, had that been done in the case of the Attock girders, a very close approximation to the desired camber would have resulted. *Fig. 2* on p. 276 of Mr. Everall's Paper might be consulted in that connection. An unaccounted-for loss of 2 inches of camber at the crown of the Sydney arch could be entirely neglected, and was evidence that the fabrication and workmanship at Sydney was of a sufficiently high order even for such a structure. Further information would be welcome with regard to the amount of clearances allowed in fabrication, for example when a member was to be fitted between gussets at the site. That was a matter which even to-day was not given sufficient attention by experienced bridge builders in England, and which to his knowledge had recently caused considerable delay and inconvenience in the case of a bridge renewal on one of the home railways.

Mr. HARRIS BOOTH said that on p. 167 of his Paper Mr. Freeman referred to the condition of loading when there was a 3,000-ton test load on one of the trusses, and stated: "The vertical reaction below the loaded truss is 2,700 tons, and that below the unloaded truss is 300 tons." He would like to ask Mr. Freeman to amplify that. Out of the total load of 3,000 tons, 300 tons was regarded as transferred to one side, and constituted a couple which would be applied to the earth and would have to be compensated by an equal resisting couple. Whether that came along the decking, down the pylons, or in some other way he could not quite satisfy himself. He submitted, therefore, that the passage to which he had referred should be explained, eventually tracing that 3,000 tons down to the earth in its original line of action. In the Paper by Mr. Freeman and Mr. Ennis, the Table on p. 227 gave the various stresses allowed in the pins that were used in erection, and the reasons for those rather high stresses were mentioned. The last item in that Table was "Bending stress on pins, 16·0 tons per square inch." That figure was, he took it, the ordinary bending-moment divided by the modulus of the section. If that was the case, the long-beam formula had been used, which did not apply to short pins. The full mathematical elastic theory of bending could deal with beams whose span was small compared with their depth, but it was complicated, and he thought that the soundest rules for an engineer to employ when dealing with pins were that if they were strong enough in shear the bending did not generally matter, and that it was desirable to test them or, in a case such as that described, to test models.

Mr. Snow.

Mr. F. S. SNOW remarked that, besides the technical questions involved, there were other lessons to be derived from the Papers which were of extreme importance to engineers at the present time. Like Mr. Anderson, he had been astounded at the tremendous cost of tendering as disclosed in Mr. Freeman's Paper. Dorman, Long and Company spent £20,000, and, as Mr. Anderson said, it might be assumed that altogether something like £100,000 was spent by other tenderers in doing no useful work. It was a fact that most inquiries coming into a contractor's office, and especially where there were attendant risks, had a clause or clauses which might be interpreted by the contractor as: "Here is a scheme on which to tender. This scheme may be modified. Your suggested modifications may or may not be approved by us, but in any case we are not responsible, whether you carry out your scheme or our scheme; if anything goes wrong, you bear the cost." It was a serious loss to the country that so much time and money was wasted in the preparation of tenders of that nature, which resulted in price-cutting and enabled the client to get something more for his money than he was entitled to, though in the majority of cases he was perfectly willing to pay for it. It would be of interest if Mr. Freeman would say whether in his experience the cost of tendering was as high in proportion on other jobs.

Referring to the film which had been shown, he noticed a singular lack of modern plant in the excavation and quarrying. From the various drawings and photographs it seemed that the maintenance cost of the bridge, for painting and so on, would be very high. He thought that if the approach spans had been constructed in reinforced concrete all the present facilities could have been given, and the increased cost of the piers would have been amply repaid by the reduction in maintenance cost. He would like to ask if reinforced-concrete construction had been considered in that connection. The prices given in Table II of Mr. Freeman's Paper were approximately double those ruling in England at the time the work was carried out. He would like to know if that was the general proportion which costs in Australia bore to those in England. In connection with the design of the pylons, described in the same Paper, he noticed that a firm of architects had been consulted; Mr. Freeman had not merely employed an architect's assistant in his office to touch up his designs, a practice which was becoming very prevalent in consulting engineers' offices. Considering various Thames bridges which he had in mind, it struck him that he had not heard of the Sydney type of bridge being proposed for any of them, and he wondered what the reason was. Could it be that the æsthetics were wrong or the cost too high? To satisfy himself with regard to the

appearance of such a bridge, he had had a rough perspective made Mr. Snow. showing Sydney Harbour bridge to a smaller scale superimposed on the site of Waterloo bridge. He thought that it looked very satisfactory, and that it was time there was an engineering bridge in London, instead of yet another architectural bridge. It would be of interest if Mr. Freeman would say why that type of bridge could not be used in London, and whether in his view such a bridge would cost more than the scheme at present favoured for a five-arch stone-faced bridge.

Mr. RALPH FREEMAN, in reply, said that the appreciation of Mr. Freeman. The Institution was the greatest reward that could be conferred on any engineer for the work he had done, and therefore he was especially grateful to the members of The Institution for the interest they had taken in the Papers submitted. Except in America, bridges of large span were so rare that very few British engineers had had to deal with any long-span bridge, but it was a matter for pride that the great Forth bridge was the work of a British engineer. He had always thought it remarkable that the Proceedings of The Institution contained no account of that bridge. Had they done so, it would perhaps have been a little easier for him to deal with the Sydney Harbour bridge. He had tried to record not merely what had been done but why it had been done, and to refer to those points which he thought would be of interest to others who might read the Papers in the future.

Every one who had been connected with the bridge regarded his association as a great privilege, and realized that his contribution was only that of one member of a great group of people, each of whom was entitled to a share of the credit; he thought, therefore, that he ought not to miss the opportunity of referring briefly to some who should be mentioned in connection with the evolution and history of the bridge. He felt that he should refer in the first place to his late chief, Mr. G. A. Hobson, who had presented to The Institution an account of the Victoria Falls arch bridge. It was to Mr. Hobson that he owed his own association with the steel arch bridge, and to Mr. Hobson was due any share of the credit which might be attributed to himself. He would also like to refer to the late Mr. C. F. Dixon, formerly Chairman of the Cleveland Bridge Company. It was due to the initiative of Mr. Dixon that that Company, for whom Mr. Freeman originally acted in connection with the work, decided to tender for the bridge, their tender being afterwards taken over by Messrs. Dorman, Long and Company. The Cleveland Bridge Company had interested themselves in the Sydney bridge project for nearly 30 years, having submitted a proposal in the earlier competition, and it was really a great disaster for them

Mr. Freeman. that Mr. Dixon's death deprived them of the opportunity of tendering.

Although he was acting as an engineer on behalf of the contractors, it was due to them to say that his position was quite as unfettered as if he had been an engineer acting independently for a client responsible for payment—perhaps more so. He could not speak too highly of the anxiety shown by Messrs. Dorman, Long and Company throughout the whole period of construction, to do nothing but the best work, altogether regardless of what it cost. That attitude never varied, even during the period of the fearful depression in Australia which struck them a blow so severe as almost to remove the possibility of a profit. Their association with the work was due to the initiative of Sir Arthur Dorman and Sir Hugh Bell. Throughout the work, almost to its end, when unfortunately both of them died, their constant co-operation, guidance and encouragement were invaluable to Mr. Ennis and himself. The credit for the success of the work was due primarily to them.

In mentioning Mr. Ennis, he would only say that after 10 years' work he and Mr. Ennis had finished better friends than when they began; there was never a problem which they had to tackle together where they did not finally agree that they had arrived at the best solution, acceptable to them both. Mr. G. C. Imbault, who was Chief Engineer of the Cleveland Bridge Company, was associated with him as adviser to Dorman, Long and Company when the tender was submitted and in the earlier stages of the work; he took a very active part in all the discussions and was responsible for the essential processes of erection. He would not attempt to refer individually to the members of his staff. They all showed the highest skill, and their interest in the work never flagged. With regard to the erectors and other workmen engaged on the bridge in Australia, he would say that the more hazardous the risk, the more arduous the work, the greater was their enthusiasm to face it, and better manual work had never been done than was done on the Sydney Harbour bridge.

He wished to make one other acknowledgment, namely, to the engineers of the Hell Gate bridge, which was undoubtedly the precedent for Sydney. Although it was on a scale of comparable magnitude, he could fairly say that, while he had used the example where he thought right, he had retained nothing where he thought it could be improved. He hoped his Paper would produce the explanation of the extraordinary centre panel in that bridge, which he considered to be a blot on the work.

Mr. Anderson had spoken of the scale of the Sydney Harbour bridge in relation to its surroundings, and it was for that reason that

he had shown some slides which bore on that point. He did not think Mr. Freeman. Mr. Anderson's friend could share the view commonly held by Australians with regard to Sydney Harbour, which was that no bridge could be too big for it ; but, apart from their personal view, he had never noticed that the bridge looked at all out-of-scale. He had not heard that criticism from any one else, and did not consider it was justified, but even if it were, it would scarcely have been possible to adopt any form of bridge less massive in appearance except by omitting the pylons of the arch. The architectural value of the pylons was, of course, a matter of personal taste, but the possible alternative types of bridge—the cantilever and suspension—would both have required shore spans adding appreciably to the mass of the superstructure at a high level, for which there was no counterpart in the arch. The pylons did undoubtedly make the bridge look heavier than it would otherwise have done, but he thought it was agreed by all who had seen them that they were an architectural triumph. If the pylons were to be used at all, he believed the best had been done with regard to them, and that was a debt which Sydney owed to Sir John Burnet and his partner Mr. Thomas Tait. Sir Harley Dalrymple-Hay had referred to the pylons being set back from the arch, but that was exactly what Sir John Burnet had aimed to do ; he was insistent that the pylons should be so far detached from the arch itself that the form of the arch would be conspicuous and would not be disguised or interfered with in any way by the pylons being placed too close. In the case of the Hell Gate bridge the pylons were placed very close to the bridge and the top chord was run back into the pylons, implying that the bridge, which was really an arch resting on bearings, was anchored to or built out from the pylons. He thought that that feature of the Hell Gate bridge was a great misfortune. His personal view was that the Sydney Harbour bridge would have looked far better without any pylons ; they had no structural value, and there was no doubt that they added very considerably to the difficulties of erection. He felt that so strongly that when the tender was submitted he persuaded Dorman, Long and Company to put in a fully-illustrated proposal for a bridge without pylons, but it was disregarded. The New South Wales Government had decided to have pylons, whatever they cost, and had also determined to have Australian granite facing for them ; no cheaper granite or alternative to granite would be entertained. Pylons of simple concrete were also proposed, but the Government would not admit any alternative.

The erection cranes had been discussed by Mr. Anderson, who suggested that the "leapfrog" type of crane, as used for the Tyne bridge at Newcastle, might have been employed at Sydney. That,

Mr. Freeman. however, would have been out of the question ; the suitability of that type depended somewhat on the relation of the width of the bridge to the panel length. The advance forward of each panel in the case of the Sydney bridge was nearly 60 feet, but the width between trusses was about 100 feet, and a crane suitable for erecting the arch members would not have been suitable for erecting another crane on the other side of the bridge 100 feet away. The creeper cranes were very expensive ; they had cost about £45,000 in England and about £50,000 by the time they were erected on the bridge. They were, however, perfect appliances. They did every duty that was imposed on them without the slightest hitch from beginning to end, and if the same problem had to be faced again he did not know a better way of dealing with it. The chords as designed were strong enough to support the cranes. Mr. Anderson was hardly right in saying that the Tyne bridge was an understudy of the Sydney Harbour bridge, because, although the Tyne bridge was completed first, its design was made after that of Sydney Harbour bridge. Indeed, the circumstances were almost the reverse, as Mr. Freeman made use of much of the investigation made for the design of the Sydney Harbour bridge when designing the structural details and the erection scheme of the Tyne bridge.

Mr. Anderson asked if it would not have been possible to do without the footways on the bridge. That was a question for Dr. Bradfield to answer, as he was responsible for the scheme of the bridge. The deck had exactly the same dimensions as the Philadelphia-Camden bridge, which carried very heavy road traffic. His personal belief was that the Sydney bridge could have been built for a narrower road and for two lines of railway instead of four, and, had that been done, with the other possible economies a sum exceeding £2,000,000 could have been saved. That could have been set aside and, bearing ordinary interest, would have sufficed to build another bridge at another site by the period when further accommodation for road traffic would be found necessary. Congestion at the bridge heads could then be avoided and the new bridge arranged to suit the traffic which it would then have to carry. No one could foretell how road and rail traffic would distribute themselves many years hence, and he thought it was unfortunate that so much money should have been spent to provide for traffic requirements which might not develop.

Dr. Faber had referred to the design of the bridge, and commented on the illustration of alternative types of lateral connections. The figure taken from the specification to which he referred was not, however, in any sense a design. The specification contained no reference to it and no contractor was asked to take any notice of it.

It was merely an illustration reproducing a feature of the Hell Gate Mr. Freeman bridge.

The form of arch suggested by Mr. Williamson was an interesting alternative. In the case of another bridge the Author elaborated a design of that type for a large and heavy span and it appeared quite practicable. Its appearance was not attractive, and in the conditions of the Sydney Harbour bridge the Author did not believe that it would have proved to be less costly than that adopted. It would have required erection processes less straightforward and apparently more difficult. The suggestion of four main trusses was interesting and was a possibility that had not been overlooked. The Author's design, nominally with two trusses, had really four trusses with the two on each side combined into one. This produced members of considerable breadth and had the maximum lateral rigidity both generally and locally where the plane of the deck intersected that of the trusses. If subdivided into four separate trusses, the areas exposed to wind would be doubled and the resistance to wind stresses afforded by the same material would be reduced. There would be no change in the erection lifts, splices and other details, because, as stated above, each of the trusses used was virtually two trusses joined together and all the heavier members were erected in two parts. Four trusses would have led to more costly bearings and foundations. The form adopted for the arch was discussed in the Paper.

The pressure on the rock had been mentioned by Mr. Anderson and Sir Harley Dalrymple-Hay; that given in the Specification was 70,000 lbs. per square foot, and there was no printer's error in the Paper, for it worked out to 31 tons per square foot. Not only was that stated in association with the drawing which said the depth of the rock was to be 5 to 10 feet, but clause 425 of the Specification read: "It is not anticipated that the foundations for the main piers of the arch bridge will be at a greater depth than that shown on the plans." Those proposals had caused him the gravest apprehension. He had insisted that, regardless of the specification, Dorman, Long and Company's tender should provide for a depth of 25 feet and a pressure of only about 13 tons per square foot, and he was so anxious that the rock should be properly examined and that the foundations should be reliable that he went to Sydney himself to see them before any concrete was deposited. The facts which he had mentioned with regard to the tender were the origin of the level which Dr. Bradfield had described as the designed level of the bridge.

The measured span of the bridge had been mentioned by Mr. Anderson. Messrs. Dorman, Long had made two careful measurements after the bridge was finished. They agreed very closely with

Mr. Freeman, one another and gave the span as $\frac{3}{4}$ inch more than the original estimate. Dr. Bradfield had recorded the figure as $1\frac{3}{8}$ inch, or $\frac{5}{8}$ inch more than the Dorman, Long measurement. The only conclusion Mr. Freeman could draw was that both surveyors' measurements were likely to be wrong. It was most probable that the span of the bridge had not been correctly measured in the first place, and he was in agreement with Dr. Faber in that he had no reason to credit the suggestion that there had been any contraction of the concrete since the bearings were placed. The concrete was deposited slowly under conditions which allowed its heat to be dissipated, and about two years elapsed before it was subjected to any appreciable load. He was not aware of any record of surface disturbances or other indications or measurements indicating subsequent movement. It seemed to him inconceivable that there could have been movement without such visible evidence. As Dr. Faber pointed out, contraction after the bearings were placed was highly improbable.

Mr. T. Peirson Frank referred to the quality of steel used. Reasons for the choice of silicon steel were given in the Paper. At the time of the construction of the bridge the only other qualities of steel available for such a structure were relatively costly alloy steels. Since then, steels having properties similar to mild steel, but with a strength about 50 per cent. greater, had been produced at a cost a little above that of silicon steel, and there was no doubt that, in Great Britain at all events, those steels would displace both mild steel and silicon steel for bridge structures of more than small span.

Professor S. M. Dixon discussed the stress measurements made on the bridge, and in his Paper (p. 371) Dr. Bradfield said that those stress measurements were used for adjusting the stresses in the anchor bolts and the reinforcing plates on the top chords of the bridge. He himself knew nothing of that matter; the contractors had never had those or any other extensometer measurements passed to them during the whole course of construction, and did not use them in stressing the anchorage bolts or in placing the reinforcing plates. Professor Dixon also referred to the reversed curve of the top chords. That was perhaps the criticism of the outline of the arch most difficult to answer. He could only say in defence of the outline used that it was thought to be the best, but would admit that if he had the opportunity of repeating the structure, he would be inclined to avoid the reverse curve. The decision to use granite was made by the Government of New South Wales. The contractors submitted with their tender a proposal for piers finished in white concrete, and he agreed with Professor Dixon that if pylons were desired, granite facing was not essential for good appearance.

The passage to which Mr. Harris Booth referred was incorrectly worded, and he was obliged to him for calling attention to it. The passage should read "absorbed by" instead of "below." The end reactions referred to were those acting on the arch trusses within the structure; the resultant forces on the bearings of course had to be in line with the externally applied loads. The bending stress on pins was intended to be the ordinary bending moment provided by the modulus of the section: the stress of 16 tons per square inch was a limit allowed by the specification, and applied to pins of any proportions. Some of the pins were short, for example, the 27-link plate pins, with a span of little more than their diameter; others were relatively long, such as the cable attachment pins which had a diameter of 11 inches and a span of nearly 3 feet.

In reply to Mr. Snow, he believed that there had not been another case of competitive design and tendering comparable with that of the Sydney Harbour bridge. He was satisfied that reinforced-concrete for the approach spans would have been more costly than steelwork, and he was not prepared to agree that the maintenance cost would be less than those of steel. When reinforced-concrete cracked or failed the cost of repair or reinstatement was very high. He was unable to make a comparison between the cost of work in England and Australia. Costs for work of such an exceptional character as a large bridge were not a reliable guide. There was no reason why a single-span arch bridge should not be used across the Thames. It would certainly be less costly than a multiple-arch bridge with a number of river piers.

It was unfortunately impossible to comply adequately with Mr. Gribble's request for more information regarding the relative merits and costs of some of the alternative types mentioned. Useful information regarding the cantilever and cantilever-arch alternatives would be expanding the reply into the subject-matter of another Paper. Mr. Gribble asked why the arch was the cheapest type of bridge. He could only say that from the outset he had firmly believed that that was the case, and subsequent events had confirmed that belief. The inception of the arch design had been referred to by Dr. Bradfield. In 1922, when Mr. Freeman had been advising the Cleveland Bridge Company, tenders were invited for a cantilever bridge, and he had informed Mr. Dixon that he thought they would be wiser to confine their studies to an arch bridge, which he felt sure would be a better proposition. In August of that year Dr. Bradfield visited England and met Mr. Dixon, who told him of their wish to tender for an arch bridge. The following year tenders were invited for an arch as well as a cantilever bridge, and various firms, the Cleveland Bridge Company amongst them, proceeded to prepare

Mr. Freeman. their tenders. In October Mr. Dixon died, and the Cleveland Bridge Company cabled to Dr. Bradfield that owing to that misfortune they would be unable to tender. Dr. Bradfield cabled back, "Hope you will be able to reconsider your decision, because arch bridge was specified at Dixon's special request."

With regard to the relative merits of the cantilever and arch types of bridge, the best evidence was that of the tenders; the cantilever cost 30 per cent. more, if compared on an equal footing. The principal reasons for that, in his opinion, were that the cantilever made no use of the ability of the rock to resist thrust, and necessarily required anchor arms which, in the circumstances of the Sydney Harbour bridge, would be practically useless parts of the structure. The position was quite different in the case of the Forth bridge, where the anchor arms were effective parts of the bridge spanning the waterway. The difference was indicated by comparing the pressures on the principal piers. The vertical pressure in the case of an arch bridge was approximately 55,000 tons, and in the case of the corresponding cantilever bridge 95,000 tons.

Furthermore, rigidity or freedom from large deflections was especially important owing to the peculiar disposition of the railway tracks to which reference had been made in the Paper.

When commenting upon the erection of the arch as a cantilever, Mr. Gribble overlooked that at the cantilever stage of the arch erection the anchorages had to support only the arch trusses and lateral bracing. The anchorages required were very much lighter than those of a complete cantilever bridge, which would have had to support the deck and live load in addition to the trusses and bracing.

In conclusion, he would make one observation in justice to himself, to the contractors for the bridge, and to those who worked with them. Dr. Bradfield's business was to see that Sydney was provided with a good bridge at a minimum cost, and he was justly entitled to all the credit due to him if those objects had been attained. To secure them had been his lifetime's work, and only those who had been there could realize the magnitude of the task which he undertook and his claims on the gratitude of his countrymen. One part of that work, however, was the building of the structure and foundations of the bridge which now spanned Sydney Harbour, and Dr. Bradfield's Paper hardly did full justice to those who carried out the duty imposed on them in unmistakable language in the contract, which, as he had stated on p. 160, made the contractors entirely responsible for the satisfactory design and construction of the bridge. Every drawing for the works was made by the contractors, and throughout the entire execution of the work neither Dr. Bradfield

nor any member of his staff directed or inspired any process of Mr. Freeman. design or erection.

Mr. J. F. PAIN thanked The Institution for the very kind way in Mr. Pain. which the Papers had been received, and said how gratified he was at the very interesting discussion which had taken place. On behalf of himself and those of his colleagues who were concerned with the calculations, he would like to acknowledge their indebtedness to Dr. Bradfield and his staff for their very careful checking of those calculations, which gave them considerable confidence in carrying out the work.

He would certainly like to have further information with regard to the stress-measurements carried out on the bridge, which had been described by Dr. Bradfield. The discrepancies between the measured and calculated stresses obtained from the strain-gauge readings on the actual members seemed hardly to be accounted for in the way there suggested. He could only say that in the model member tests, where many strain-gauge readings were taken, there was no evidence of the same sort of discrepancies due to differences in fabrication; in fact, in all cases where a number of models of the same section were tested the crippling loads were remarkably uniform.

The areas given for the model members in Table X were the actual areas of the models as obtained from the weighbridge weights. In many cases the weights were found to be appreciably different from the calculated values, and in such cases the theoretical cross-sectional areas were adjusted accordingly. The net area given for the tension members was that obtained by the conventional methods of allowing for the diagonal pitch of rivets.

In the case of the models of members subjected to bending stresses, sufficient additional load was applied to produce in the model the same bending-stress intensity as would exist in the actual member. The bending stresses in the members due to their own weights were calculated on the assumption that the ends were freely supported, and in order to reproduce that bending stress in the models it was necessary to apply a certain amount of additional load, since the bending stress induced in the model by its own weight was reduced in proportion to the scale of the model.

Professor Dixon had commented upon the extreme degree of accuracy to which many of the calculations were carried. Any calculations for structures of the kind in question were based on certain assumptions with regard to loading which were determined arbitrarily before the work was put in hand, and in many cases the data on which those assumptions were based were unreliable. It was nevertheless necessary to calculate the stresses

Mr. Pain.

accurately for the particular assumptions made, and, as calculating machines were employed, any required degree of accuracy could be obtained without additional trouble. The length and laborious nature of the stress-analysis arose from the large number of loading conditions investigated and the indeterminate condition of the structure rather than from the degree of accuracy to be attained.

* * Mr. Ennis's reply will be found on p. 460 and Dr. Bradfield's on p. 462.—SEC. INST. C.E.