Discussion.

The PRESIDENT, in moving a vote of thanks to the Authors for the valuable information given in their Papers, remarked that Mr. Popplewell was present, but Mr. Ellis, who was on his way home from China, would not be able to attend during the discussion, nor would Mr. Burr, who was resident in the United States. He thought it was the first time that The Institution had had before it Papers dealing definitely with reinforced concrete. There was no doubt that there was very wide-spread interest at the present moment in work carried out in that material. In a certain sense it was a new material, involving new considerations—a material with regard to which the experience at present, although large in amount, was limited in many ways. There were a few engineers who had been engaged for some time in carrying out reinforced-concrete work, and who undoubtedly knew a great deal about it, but the members of The Institution did not wish to be entirely dependent on the few engineers who had had such special experience, and they desired to know, as far as possible, the results of other experience. Therefore The Institution heartily welcomed Papers which brought forward the results of experience and of experimental investigation. Reinforced-concrete construction was, of course, open to theoretical investigation, like other methods of construction, but he thought it was not doing it any discredit when he said that the calculation of the stresses in reinforced concrete involved rather more assumptions than did the calculation of the stresses in steel structures, with which engineers were more accustomed to work. The assumptions which had been made, and more or less tested, were no doubt good approximations, but it was very important that these assumptions should be re-tested from time to time. It was necessary to know the limits of approximation of the assumptions which were made in carrying out calculations relating to reinforced concrete, and experimental work such as that of Mr. Popplewell, who had tested some of the assumptions on which columns had been calculated hitherto, was very valuable. The Institution itself had foreseen the desirability of acquiring information about reinforced concrete for the use of its members, and it had appointed a Committee which was collecting information as to actual structures and as to the kind of assumptions that had been made in calculating the stresses in them. It had also carried

The President.

The President. out for itself a certain number of experiments. Unfortunately, with a discrete body, like a reinforced-concrete column or beam, it was no use making experiments on a very small scale; they had to be made on a large practical scale, and consequently they were costly. Therefore The Institution was very glad to welcome Papers giving the results of experimental research. But good and valuable as experiments were, engineers did not like to depend upon them alone, and it was equally important therefore to obtain detailed descriptions of works that had been carried out, showing the devices adopted and putting forward some account of the results, good, bad, or indifferent, and of the accidents and the successes during the construction of such works. One of the Papers that evening was a very valuable one indeed in that respect. The subject altogether was one of great interest, and he hoped there would be a thorough discussion.

Mr. Popplewell. Mr. W. C. Popplewell wished to emphasize the fact, which he had pointed out in his Paper, that the experimental work described was necessarily very limited in extent. The intention had been to find out something about the relative movement of the steel in the concrete—if there was any movement—and he thought the experiments had proved that, in the columns dealt with, at any rate, there was no perceptible movement. Another point was that the experiments on frictional grip had been taken in tension. Since carrying out the experiments he had made further experiments on frictional grip in compression, by pushing bars—ten $\frac{3}{4}$-inch bars and ten $\frac{9}{16}$-inch bars—through blocks of concrete of different thicknesses, and the results corresponded very nearly with those obtained in tension, the average frictional force exerted between the concrete and the steel being between 550 and 650 lbs. per square inch.

Mr. Meik. Mr. C. S. Meik complimented Mr. Ellis upon the boldness of his design, having regard to the fact that he had placed the wharf upon piles the bearing-power of which he could not calculate by any recognized formula. It was certainly rather alarming to read (p. 82) that in one pier the piles sank rapidly under the steady weight of the hammer and cylinder, without any driving, and had to be checked by holding up this dead weight when they were at the required depth. He thought that any engineer who had to carry out works such as those described would have considered this not merely—as Mr. Ellis said—"unsatisfactory," but "extremely unsatisfactory." On the next page Mr. Ellis stated that the piles had stood the test of having more than the estimated load stacked upon them locally. Mr. Meik did not know to what extent this loading had been carried out, but it was apparent that on loading a certain area of the wharf the piles had not shown any appreciable shrinkage. That demonstrated the advantage of reinforced-concrete decking. Such decking was of great assistance in distributing the load, and it therefore assisted the piles—especially such piles as those used in the present case—to carry a heavy weight. It would be very interesting to see how the piles behaved when the wharf came to be loaded over its entire area with the load it was supposed to carry. It was well known that a great deal of the carrying-capacity of a pile depended upon frictional resistance. Some little time ago he had to carry out a pier in Italy which was built on reinforced-concrete piles driven into sand, and he found that, after allowing a pile to rest for 2 months, the frictional resistance to driving had doubled. Farther on in his Paper Mr. Ellis referred to moulding the wharf-members on shore, carrying them into position, and joining them up. Mr. Meik had tried that plan lately for the first time, but he was bound to say it had not proved very satisfactory. There was great difficulty in getting the heavy concrete members into position and in satisfactorily joining them to the other work. In future he would certainly go back to the old system of moulding them in position, even if it were between high and low water. One of the chief advantages claimed for moulding on shore and carrying out was a saving of time, but, in his own experience, that advantage had proved illusory. Although Mr. Ellis gave the working-load of the steel as 16,000 lbs. per square inch, or about one-fourth of the ultimate stress, later on he mentioned the total load on a beam as being 93 tons, which, compared with its working-load of 14·6 tons, showed a ratio of $6\frac{1}{2} : 1$. He thought that what Mr. Ellis meant by the 16,000 lbs. was the maximum allowable working-stress, because the steel of many of the beams was not stressed to more than 12,000 lbs. per square inch. He thought the beams might have been designed rather more economically: in most of them there was 2 to 3 per cent. of steel, which was not a very economical ratio in dealing with work of this description. By making the beams a little deeper some of the steel might have been saved. He congratulated Mr. Ellis on his success in getting the work carried out at a very low cost. For instance, the reinforced-concrete foundations and ground floor of the warehouses had cost 2*s.* 3·4*d.* per cubic foot, which was very low. The cost of the piles in the wharf worked out at 3*s.* 7·4*d.* per cubic foot, which also was very low. In England it was not possible to get pile-work done under 5*s.* per cubic foot: perhaps the Chinese labour available, which was very adaptable in connection with such work, had resulted in some saving at

Mr. Meik.

Mr. Meik. Shanghai; but it did not appear to account entirely for the very low cost of the work.

With regard to Mr. Popplewell's Paper, he heartily endorsed what the President had said about the advantage of the Paper to engineers. He noticed that there was considerable variation in the crushing-stress of the cubes of concrete. He presumed the cubes were made at the same time as the beams. There was a variation from 120 tons per square foot in the case of beam C to 192 tons per square foot in the case of beam D. Column C was said later on in the Paper to have been tested to destruction and found to fail at 50·75 tons, which was lower than most of the others. Variation in the strength of the concrete was one of the most difficult things that had to be contended with in designing reinforced-concrete structures. It was necessary, therefore, at the present time, to allow a considerable margin in connection with the ultimate strength. In the test-column referred to in the Paper the ratio of the volume of steel to the volume of concrete worked out at about 5 per cent., which for a column was fairly heavy reinforcement, but not abnormal. The frictional resistance obtained also showed great variation, but that was usual, and was presumably due to the difference in the length of the bars embedded: the lowest resistance was 243 lbs. per square inch of area of bar, and the highest was 614 lbs. But it was satisfactory to know that the 100 lbs. per square inch generally adopted allowed a considerable factor of safety in regard to slipping. He did not understand what Mr. Popplewell meant by "adhesion" and "frictional grip" on p. 99, where he said: "this is often called adhesion, but is more probably in the nature of a frictional grip preventing any relative motion of the surfaces of the steel and the concrete." He had always thought that adhesion was due to pressure, and he presumed grip caused pressure, and therefore he thought the adhesion was dependent on the grip.

Although the title of Mr. Burr's Paper was Composite Columns of Concrete and Steel, he noticed that in the text Mr. Burr spoke of the columns as reinforced-concrete columns. In Mr. Meik's opinion they were not proper reinforced-concrete columns, but were more of the nature of protected steel columns, such as had been used in America for many years for large buildings, and had been adopted in England in recent years. They consisted of a steel framing and what might be called a veneer of either concrete or ashlar; and a good example of this method of construction would be in existence in Great George Street shortly. The only advantage that could be claimed for it was, he believed, rapidity of execution, and that was not always secured. In the two columns shown in *Figs. 1* (p. 117) there was no bond between the outer shell of concrete and the concrete inside the column; the outer shell was really useless, and would no doubt come off if the column were subjected to severe strains. Moreover, it was of no use as a protection against fire, and he thought Mr. Burr admitted that to be the case, because he said: "This outer concrete or mortar is simply a protecting shell, only the concrete lying within the steel being considered as load-carrying. This is necessary because in case of fire much of the outside or protecting material may be destroyed or knocked off, so that the carrying member should be considered to consist only of the steel reinforcing column and the concrete lying within it." Under those circumstances Mr. Burr put his column out of court as regarded resistance to fire. It was also out of court with regard to economical carrying-capacity. It was true that such steel columns would carry very heavy loads, but considered as reinforced concrete they were exceptionally expensive. Mr. Burr's columns contained $10\frac{1}{2}$ per cent. of steel, which was of course an extravagant allowance for a reinforced column. Even Mr. Popplewell's columns with 5 per cent. were rather heavily reinforced. Mr. Ellis had also used somewhat heavy reinforcement in the column which had to stand a load of 376 tons, namely, $6\frac{1}{2}$ per cent. If Mr. Burr had reduced his steel by one-half and doubled his concrete, he would have obtained a stronger and much cheaper column. From the few calculations he had made as to the cost in England Mr. Meik made out that there would be a saving of 30 per cent. by doing so. It was true the column would be of rather larger diameter, but unless the circumstances were exceptional that did not matter very much. After reading the Paper he had the impression that Mr. Burr was advocating the use of steel from the steelmaker's point of view, and assuming that to be the case, a reinforced column such as Mr. Burr put forward would be very advantageous; but from the user's point of view it would be very expensive, and he thought that to call it reinforced concrete and put it forward seriously as a favourable example was stretching a point too far. Mr. Meik.

Mr. R. W. Vawdrey remarked that it was extremely gratifying to those members who were particularly interested in reinforced concrete that three such Papers should have been read at The Institution. He took it that the Authors of the experimental Papers would not claim they had produced any new results, but, as the President had remarked, it was of extreme importance that older conclusions should be confirmed, and the Papers did confirm the principles on which most engineers had been working. Taking first Mr. Vawdrey.

Mr. Vawdrey. Mr. Popplewell's Paper, he was not quite clear as to what the Author had really aimed at. How otherwise could the steel and concrete act in a column than together? Towards the end of the Paper the Author described tests of a further column by point loads at the end, and that seemed really to prove something, namely, that a point load which was not applied directly to the steel was yet transmitted to it by the concrete very quickly. But, assuming that the reinforced-concrete column was carrying the load, both materials must necessarily be compressed together, and as the compression was the same in each case, he failed to see how the testing of such a column tended in any way to cause a shifting of the steel in the concrete. In tension the case was quite different. In the tension bars at the bottom of a beam, for instance, the concrete would crack and fail long before the steel had reached its yield-point, and consequently there was a tendency to movement of the concrete along the steel; but where both were in compression, he failed to see that there was any tendency to movement until the actual crushing-load of the concrete was reached. On p. 99 Mr. Popplewell practically asked for a definition of adhesion. He did not quite understand what the Author meant by comparing it with frictional grip; presumably the two things were the same, whatever they were called. In connection with that point it was rather interesting to remark that the adhesion or grip on round bars appeared for some reason or other to be better than that on square bars. Possibly it was due to the contraction of the concrete around the bar. In some experiments he had made recently, on much the same lines as Mr. Popplewell's extraction experiments, it had been brought out fairly clearly that round bars, whether mechanical-bond bars or plain bars, were much superior to square bars, although the sectional area of the round bar was larger in proportion to its perimeter than that of the square bar, and the contrary effect would be expected from first principles. He noticed that Mr. Popplewell adopted the ordinary method of constructing columns, namely, timbering three sides and making up the fourth with separate pieces as the concrete was raised. That was by far the best method. He had tried both ways, and had found that the only case in which it was at all satisfactory to pour concrete from the top of the column was when the column-centering was big enough to allow a man to get down into the column. That was practicable only in columns not less than 3 or 4 feet in diameter. In such cases it could be done quite satisfactorily by having centering of a circular form and allowing a man to get down into the middle to place the concrete, a method which was adopted extensively in the United States.

Mr. Vawdrey.

He did not quite understand the meaning of the paragraph dealing with column E on p. 107, where Mr. Popplewell said: "It might be suggested that the effect of the three first loadings and releases has been to disturb the grip." Assuming that disturbance of the grip between the steel and the concrete had taken place, he did not understand how it would throw more load on to the steel. It seemed that if both steel and concrete were being compressed to the same extent, there was no tendency in the steel to move relative to the concrete, and the grip, whether it existed or not, had no effect. It was rather a pity no test-block had been made from the concrete with which column A was mixed. It appeared to have differed considerably from the others in composition. He thought Mr. Popplewell was absolutely right in suggesting that the slight divergence of his curves was due to the fact that the movement or contraction of the concrete was necessarily measured at the outside of the column, whereas the steel was some little way inside. It was equally conceivable that there might be a weak part of the concrete which would compress at the outside edge of the column, or the reverse, while the portion of concrete actually next the steel would be carried down with the steel. He wished to congratulate Mr. Popplewell on the tests he had made on the extraction of bars from concrete, because a test so rarely acted exactly as it ought to do. The effect of the steel reaching its yield-point, and so pulling away from the concrete, was exactly what might be expected to happen. The Table of frictional resistance to slipping (p. 111) was extremely interesting, but Mr. Vawdrey would like to know the reason for the very marked discrepancy between the first three items and the next three. The average of the column headed "gripping force per unit area of bar" was 516 lbs. per square inch in the first three and 337 lbs. in the next three, and yet in the last three cases the age of the concrete was 7 months, as compared with 2 and $2\frac{1}{2}$ months in the first three. Surely it would have been expected that, after 7 months, the grip of the concrete on the bar would be appreciably greater than at 2 months. It appeared as though the grip of the concrete diminished with age. The point he had already referred to arose again at the bottom of that page, where Mr. Popplewell calculated a stress of 3·86 tons per square inch in the bars and said that it amounted to a load of about $1\frac{3}{4}$ ton on each bar, "which load, when compared with the figures in the last Table, was obviously nothing like sufficient to cause a movement of the concrete in the steel." But that stress in the steel was not tending in any way to move the bar in the concrete, and Mr. Vawdrey did not understand what point Mr. Popplewell was trying

Mr. Vawdrey. to make. Both the steel and the concrete were stressed, and, of course, in direct proportion to their moduli of elasticity.

Turning to Mr. Burr's Paper, he entirely agreed with Mr. Meik that the columns were not really reinforced concrete, but rather steel encased in concrete. At the bottom of p. 114 Mr. Burr said: "In columns of this type the reinforcement is not so distributed or held as to constitute a direct load-carrying element of the column." But surely that was not so. A purely reinforced-concrete column might occasionally have, as Mr. Meik had pointed out, as much as 10 or 12 per cent. of reinforcement, and he had known many such cases. Taking the ratio of the two moduli of elasticity as 15, considerably more than half the total load on the column was carried by the separate steel bars. Surely it was incorrect in such a case to say that the steel did not form a direct load-carrying element of the column. Farther on Mr. Burr said: "It has been shown by Professor A. N. Talbot and others that with this type of reinforcement the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete." That was directly disproved by Mr. Popplewell's experiments, which showed that even from the very early stages of the test the steel did actually take its proportion of the load. He agreed with Mr. Burr that the frame type of column might be very suitable for extremely high buildings—say, forty stories—but that was not on account of its intrinsic merits as a reinforced column, but because it was capable of carrying a very considerable load before the concrete was put there, and the strength of the steel could be used from a structural point of view, which was a great advantage in many cases. The percentage of steel in the frame columns was much higher than could be used economically, and for such small columns it would be quite impossible to adopt that method with any economy. He dissented from the statement on p. 126 that the frame column was the best form for effective banding. A straight strap from one angle to the next was surely not an effective banding or binding of the concrete in between the vertical angles. It necessarily stiffened the steel, whether surrounded by concrete or not; but the effect of circumferential binding in the column—either spiral binding or circular straps—was also to confine the concrete, and that, he claimed, was not done at all well by straight straps as in the examples shown.

With regard to the works at Shanghai, the raft construction was very interesting and very suitable for that class of work. Occasionally it was advantageous to construct a slab on the top of the beams, care being taken, of course, to tie it down to the beams, because the ground-pressure tended to force the slab away from them. But, when that point had been attended to, it was often very advantageous to have the actual slab, which, of course, formed the tee of all the tee-beams, on the top of the work, so as to form the permanent floor. The question of designing the beams as tee-beams did not present much difficulty because they were acting as tee-beams at the centre of the span instead of the supports, or conversely; they might act as tee-beams at the supports and as plain rectangular beams in the centre of the span. It did not make very much difference, from the point of view of economy, which way they were taken. The cover of the steel was mentioned as having been never less than 1 inch. That appeared to be rather too much for slabs. After all, with the small bars used in slabs, ½ inch cover was quite sufficient for protection against fire, and 1 inch in beams or slabs merely had the effect of causing cracks, because the surface of the concrete was farther away from the neutral axis and consequently had to extend more under load. In columns, of course, the cover might be of any thickness. With regard to the costs, that of the piles was extremely low. He had never known any reinforced-concrete piles in England carried out as cheaply as 3*s.* 7½*d.* per cubic foot. On the other hand, the cost of the superstructure, especially in the wharf, appeared to be rather heavy, 3*s.* 6*d.* per cubic foot, verging on £5 per cubic yard. A more usual price in England for that class of work would be between £3 and £4 per cubic yard. Mr. Vawdrey.

Mr. Oscar Faber, referring to Mr. Ellis's statement that the beams had been assumed to have partial continuity, and the bending-moments had been taken as $Wl/12$ at the centre of the span, observed that it would be interesting to know what had influenced Mr. Ellis to decide upon that value, because although it had been given as the bending-moment at the centre of the beam, it did not appear to have much theoretical or other justification. The bending-moment at the centre of a continuous reinforced-concrete beam depended not only on the loaded span, but also largely on the condition of the adjacent spans—whether they were loaded or not. The worst possible condition that could occur was for alternate bays to be loaded and the intermediate bays to be unloaded; and even then there would remain on the intermediate bays the dead load due to the weight of the floor and the beams. Taking that into account, with the loads given for these particular warehouses, namely, a live load of 500 lbs., and a dead load amounting to about 100 lbs., per square foot, exact calculations showed that the maximum bending-moment that could possibly occur under the worst possible conditions was $Wl/13 \cdot 1$, or Mr. Faber.

Mr. Faber. 10 per cent. less than the value actually adopted. Any such calculation assumed that there was no settlement of the supports. In the building under consideration Mr. Ellis had no doubt been justified in making some allowance for settlement, but exactly what allowance ought to be made was obviously a moot point. The actual stress in any reinforced-concrete beam calculated as continuous was uncertain when settlement occurred. He would like to know whether the outside columns had been designed for eccentric loading or not, because it was not very clear from the drawing to what extent the outside columns differed from the inside columns. There appeared to be a considerable diversity of practice in regard to the extent to which outside columns should be designed for eccentric loading, and obviously this had a marked influence on the size of such columns.

He considered Mr. Popplewell's Paper a very valuable one. This seemed to be the first time that tests had been made to find the relative stresses on the steel and the concrete, except by finding experimentally the increase in the total load which the column would carry before breaking when longitudinal reinforcement was added. All previous determinations of the value attaching to the steel relatively to the concrete had been deduced from the ultimate loading on the column, and up to the present the value of m, the ratio between the moduli of elasticity of steel and concrete, had always been taken at a lower figure than the 19·8 which Mr. Popplewell had found. The value most frequently adopted was 15; in some recent reports it was even lower, e.g., 10 or 8. The reason for that would appear to be that in all column-tests made up to the present the ends of the columns had not been enlarged, as Mr. Popplewell had made them: the specimens had been so designed that the steel was unable to take its fair share of load near the ends, failure therefore occurring at the ends. It was impossible for the steel at the ends to be taking its fair share of the stress, since that stress was transmitted to it only by adhesion; and therefore, in order to secure equal strength of the column at this point, it was essential that the column should be gradually enlarged towards the ends in such a way as to enable the concrete alone to take the whole load at this point. Nearly all the tests made hitherto, which had given a comparatively low value for the steel in reinforced columns, had been largely vitiated by that point having been overlooked. Mr. Popplewell, by enlarging the ends of his columns, and so preventing them from failing at their weakest point, had found that the steel was much more effective than had been thought hitherto. That was particularly interesting, because a very low value was usually assumed for steel, and there were many reasons for supposing that its real value was much higher. For instance, beams reinforced heavily on the compression side, had always shown much greater strength in compression than that deduced from the ordinary formula, taking the ratio between the moduli of elasticity of the two materials as 15. Actual experiments had shown that the strength of steel in compression, to which, of course, the stress was transmitted solely by adhesion, had been always much higher than that given by the usual formula; and that conclusion was borne out by Mr. Popplewell's valuable experiments, which were likely to justify giving to the steel in columns a higher value than hitherto. Mr. Burr's columns, like most other reinforced-concrete columns, had not had their ends enlarged during testing, and therefore there had been difficulty in knowing to what extent the steel was enabled to take its load before some crushing of the concrete occurred. The results might have given even a higher value for the steel if the ends had been enlarged in such a way as to ensure the two materials acting together. The value of that particular type of column would appear to be very considerable in buildings of many stories where the size of the column had to be kept within certain limits, and where there was no practical alternative to the use of a high percentage of steel, although it was known to be much more expensive. Mr. Faber.

Mr. HAROLD D. SMITH considered that, in view of the very bad ground in which the groups of piles rested, greater precautions might have been taken by Mr. Ellis, in the case of the Shanghai work, to avoid settlement, which would lead to the breaking-up of the floor in a longitudinal direction. In the cross section the bottom walings connecting the groups of piles and the top floor-beam were connected by diagonals, which made the whole structure between the bottom waling and the floor into a stiff girder about 12 feet deep; but in the longitudinal section diagonals were only provided to the end one or two bays, and if one small group of piles forming one pier should settle slightly, the floor would have an undue stress thrown upon it and would not be stiff enough to distribute the load over the adjoining groups of piles. In that event the floor might be damaged. With a deeper girder, formed by the introduction of a continuous system of diagonals, the load might have been distributed efficiently over other groups and the structure made stronger. Each warehouse was another example of the same thing; the whole of the distribution was taken in the bottom slab and the warehouse itself was a skeleton structure, with walls consisting of brick panels, which could not augment its general stability. He thought that if diagonals had been introduced in the walls, so that Mr. Smith.

Mr. Smith. the floors and columns might form members of a girder as deep as the building, it would have been much stiffer and much more economical than a girder of the depth of the foundation-slab. That seemed to be particularly important on a foundation which allowed a building to settle 14 inches within 12 months of its construction. Diagonals might also have been introduced between the columns and floors where possible, and the load spread over the floor, and that also would have resulted, he thought, in economy in the design of the floor. The floor described in the Paper was capable of carrying the weight over three adjacent bays. If diagonals had been introduced the floor might have been made capable of distributing the load over one bay only. In the inside, between the columns, diagonals could not have been introduced everywhere, but there seemed to be no reason why, where diagonals could not be introduced in the bottom floor, they should not step up to the next floor for three bays over the one bay gap and so produce a continuous girder. In temporary wooden buildings, 20 feet by 20 feet, used as offices or stores while alterations were going on, this principle had been successfully applied. Hoop-iron diagonals were placed between the upright posts, and nailed to each post, the building could be picked up by the corner or balanced on two corners, and it would hold together remarkably well without distortion or damage. At the present time the Great Western Railway Company were reconstructing a wharf built at Plymouth about 30 years ago by the late Sir James Inglis. The greenheart and jarrah piles had been very much diminished in section by the action of marine worms, and the white-wood bracings and walings had disappeared entirely. The wharf was 820 feet long by 48 feet wide. Various methods of reconstruction were considered, and ultimately reinforced concrete was adopted for the following rather important reasons:—The cost would be about two-thirds of the cost of rebuilding in timber. Reinforced concrete was not attacked by worms. The work could be carried out in reinforced concrete with very little disturbance to the business of the wharf, which could proceed as usual, except during the reconstruction of the floor itself. The existing wharf formed a scaffold for the pile-driver, which constituted the only occupation of the wharf by temporary structures. The main beams, bracings, walings, and girders could all be constructed under the existing floor without disturbing the traffic until it was necessary to build the floor itself. The foundation was rock, or what was considered to be rock, in every case. The piles were hooped like the columns described in Mr. Ellis's Paper, and had been designed by the Considère Company. It had been possible to drive the piles about 2 or 3 feet deeper than the depth at which the boring-tackle was brought up by the rock. They had been driven without any helmet or protection, but a wooden dolly had always been used. Helmets had been introduced lately to save the dollies. An 18-inch greenheart dolly driving 15-inch piles was knocked to pieces after it had driven five piles, but now, with a cast-iron helmet and sawdust, a dolly would drive more than twenty piles. The wharf carried four lines of sidings, a crane-road carrying a hydraulic crane weighing 60 tons, and a live load of 4 cwt. per square foot. At first, in calculating bending-moments for the continuous joists and the continuous main cross girders, which rested on four piles and the wharf-wall, formulas were used such as were described in the text-books and mentioned in Mr. Ellis's Paper, but it was found that the distributed live load, the varying rolling load, and the dead load could not be taken into consideration without full calculations, which were made by Professor Claxton Fidler's graphical method for every possible combination of loading. The worst cases were taken, an envelope was drawn round the whole of them, and the girders and joists were designed to resist the corresponding bending-moment. He thought Mr. Ellis deserved congratulations on the success of his work; it was the most daring construction of a wharf that he had ever heard of. Mr. Smith.

Mr. C. F. Marsh thought all three Authors were to be congratulated on having produced such interesting matter. With regard to Mr. Ellis's Paper, it appeared to him that the raft formation adopted at Shanghai was the only solution of the difficulty. He had seen lately a design for a similar foundation using inverted arches instead of straight beams, and it had struck him as being rather a good idea. Mr. Ellis stated that he took the stress on the steel at fifteen times the stress on the concrete, with an ultimate limit of 12,000 lbs. per square inch. Mr. Marsh did not quite see why he limited it in that way. It was very probable that the maximum stresses in the actual structure were about 1,000 lbs. per square inch on the concrete, and 15,000 lbs. per square inch on the steel, which were in the ratio of the moduli of elasticity. In the Table of the tests on the wharf-beam (p. 92) the factor of safety was given as $7\frac{2}{3}$, reckoned on the calculated super-load. That was taken on the 90 tons at which the beam failed, but it should have been taken on the 45 tons at which the first crack appeared, in which case it became $3\frac{5}{6}$ the calculated load. On the next page there was a Table in the second column of which it was shown that 1 : 2 : 4 concrete gave a higher result than 1 : $1\frac{1}{2}$: 3 concrete. That was curious, because in the third column, "Final Load," the reverse was Mr. Marsh.

Mr. Marsh. the case; and the reverse was also the case in the following Table (p. 94), showing the results of tests on unreinforced columns. He found on calculating the reinforced columns by the formulas given in the second Report of the Joint Committee of the Royal Institute of British Architects, that the safe load on the columns of 1 : 1½ : 3 concrete was 44·64 tons, and on those of 1 : 2 : 4 concrete 38·17 tons. The stress in the concrete in the first case was 1,002 lbs. per square inch, and on the 1 : 2 : 4 concrete 859 lbs. per square inch.

With regard to Mr. Popplewell's Paper, there was a diagram on p. 100 which showed a gauge-length of 8 inches, but on p. 108 the length was given as 12 inches. From the calculations he had made he thought the latter must be the right lengths. With regard to the conclusions arrived at, he thought Mr. Popplewell had gone to work in the wrong way. *Fig. 5* gave the mean deformations of the concrete and the steel. The modulus of elasticity of the steel was very fairly known, and it was probable that the figure given was correct; but the modulus of elasticity of the concrete was not so certain. However, from the results of the experiments it was possible to get at the exact modulus of elasticity of the concrete. Taking the unit strain on the steel, calculating from it the stress on the steel, and multiplying that by the sectional area, the total stress on the steel was obtained; deducting it from the total pressure on the column, the stress on the concrete was found; and from that the modulus of elasticity of the concrete could be calculated, using the strain obtained from *Fig. 5*. Calculating in this manner, the results given in the Table on p. 141 had been obtained. It would be seen that the mean value for the ratio of the moduli of elasticity was 14·5, which was very near the usual value of 15.

In Mr. Burr's Paper a statement was made that longitudinal steel rods in columns might be capable of resisting by themselves some compression, but it was of such small amount as to exert little or no direct material influence upon the carrying-capacity of the member. Of course, concrete properly hooped could be stressed considerably higher than the usual allowance of 600 lbs. per square inch, and with sufficient hooping it was possible to get 1,000 lbs. per square inch, or more, on the concrete, and therefore mild steel could be stressed up to 15,000 or 16,000 lbs. per square inch, which was its working-capacity. Later on, Mr. Burr said: "It has been shown by Professor A. N. Talbot and others that with this type of reinforcement the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete." Although Professor Talbot

Mr. Marsh.

VALUES OF UNIT STRESSES IN STEEL AND CONCRETE AND OF MODULI OF ELASTICITY OF CONCRETE, CALCULATED FROM THE STRAINS RECORDED IN MR. POPPLEWELL'S PAPER.

	Steel.						Concrete.						
	1	2	3	4	5	6	7	8	9	10	11	12	13
	Unit Strain (from Fig. 5) $\frac{\text{measured strain}}{12}$	Modulus of Elasticity E_s (given).	Unit Stress 1 × 2	Sectional Area (given)	Total Stress 3 × 4	Total Pressure on Column.	Total Stress 6 − 5	Sectional Area (given).	Unit Stress 7 ÷ 8	Unit Strain (from Fig. 5) $\frac{\text{measured strain}}{12}$	Modulus of Elasticity E_s 9 ÷ 10	$\frac{E_s}{E_c}$ or 2 ÷ 11	Mean Value of $\frac{E_s}{E_c}$
	Ins.		Lbs. per Sq. In.	Sq. Ins.	Lbs.	Lbs.	Lbs.	Sq. Ins.	Lbs. per Sq. In.				
Column A	0·00027	$30·2 \times 10^6$	8,154	1·77	14,432	30,240	15,808	34·23	462	0·00027	$1·711 \times 10^6$	17·65	14·51
Column B	0·00018	,,	5,436	,,	9,622	,,	20,618	,,	602	0·00021	$2·876 \times 10^6$	10·53	
Column D	0·0002	,,	6,040	,,	10,691	,,	19,549	,,	571	0·0002	$2·855 \times 10^6$	10·58	
Column E	0·00028	,,	8,456	,,	14,967	,,	15,273	,,	446	0·00027	$1·652 \times 10^6$	18·28	

Mr. Marsh. and others had found that the hooping did not come into play until the stress on the concrete had reached a point at which unreinforced concrete would fail, the fact that the failure of the concrete was prevented allowed a higher value to be used for the working-stress. Mr. Marsh thought that some method should be adopted whereby a higher stress on the concrete in this type of column could be allowed, as suggested by Mr. Burr. He did not quite agree with other speakers that Mr. Burr's form of column was not an example of efficient hooping, because he considered that in a column of that nature, particularly the diagonally-braced column, the vertical reinforcement acted both as hooping and as longitudinal reinforcement. It certainly did hoop the concrete efficiently, and he believed that for such a column a safe stress on the concrete of 700 lbs. or 750 lbs. per square inch might be allowed, or even more. It was rather a pity that Mr. Burr had not included tests on reinforced-concrete columns hooped in the usual manner, for comparison. It would have been extremely interesting if he had tried some of those columns, with longitudinals approaching the same sectional area as in the columns tested.

Mr. Ball. Mr. J. B. Ball stated that he had lately been engaged in carrying out reinforced-concrete structures of various kinds, from bridges to buildings, and in particular had put down recently a large raft on the banks of the Humber to form the foundation and pits for an engine-shed. It was somewhat similar to the raft adopted at Shanghai, in that on the Humber site there was about 25 to 30 feet of warp before anything solid was reached, and piling for the pits of the engine-shed would have cost a very considerable sum. After careful consideration, it was decided to put down a reinforced-concrete raft. Naturally, the design of an engine-shed floor, with pits about 3 feet deep, lent itself to the formation of a raft of a peculiarly strong character. The shed was 360 feet long, and 160 feet wide, and its superficial area was thus nearly 60,000 square feet. There were twelve pits, forming a series of corrugations, as it were, running throughout the length. The raft had been built only quite recently, and had not been fully tested, but a heavy locomotive had been running over the floor, and the results so far had been very satisfactory. The price of the reinforced concrete worked out at 1*s.* 9*d.* per cubic foot for the floors, foundations, and pits, or 3*s.* per square foot, or £2 1*s.* 7*d.* per lineal foot of the pits. Compared with ordinary construction in brick on a pile foundation, the reinforced-concrete structure—which he believed was unique in England, in that it had no wood of any kind for the attachment of chairs for carrying the rails in the shed, and the top of the reinforced concrete formed the floor of the shed as well—had cost about £9,500, while the lowest estimate (a firm tender) for a brick design was £12,900. In his opinion that showed the desirability of going more fully into the question of the use of reinforced concrete for engineering structures. He had heard all kinds of views about this new form of construction, and a good deal of criticism and ridicule had been thrown upon it, but he had come to the conclusion that it had come to stay, and would have to be seriously taken into account where cost was an important consideration. He did not believe that it was suitable for all kinds of structures: it had its limits; but it had also its uses, and, from the experience he had had, extending now over about 7 years, he was beginning to believe still more in it. In addition to the concrete raft, he was also carrying out a large granary, about 80 feet high, in reinforced concrete from top to bottom. It had been put out to competition based on general drawings supplied by his company, and various experts in reinforced concrete had submitted designs. These had been carefully analysed, and the differences between the formulas used by the several experts had been found to be remarkable. For instance, some had taken the bending-moment at the centre of a beam as $Wl/24$, others as $Wl/10$. The result was that the cubic contents of the concrete and steel in the building, which had to be stated in the tender, varied very considerably, the quantities ranging from 350 to 380 tons of steel reinforcement, with concrete in proportion. In the design actually accepted, and now, he believed, being very successfully carried out, there was about 500 tons of steel —which involved a fairly substantial increase. There were reinforced-concrete piles on the Humber which were sustaining 25 to 50 tons per pile. The cost of these varied from 5*s.* to 6*s.* 9*d.* per cubic foot. One curious incident had occurred in connection with the comparative skin-friction of timber and reinforced-concrete piles. Two piles were left standing for about a fortnight, and when driving was started again a considerable number of blows were given before there was any sign of yielding on the part of the timber pile, whereas the ferro-concrete pile sank at the first blow exactly as it had been sinking a fortnight before. Whether others had had the same experience of reinforced-concrete piles he did not know, but he proposed to make some further experiments on the subject. The ratio of steel to concrete in the granary as a whole, including silos, worked out at 1·43 per cent., there being 166,050 cubic feet of concrete and 500 tons of steel. The cost was approximately 2*s.* 9½*d.* per cubic foot of concrete, or about 3·4*d.* per cubic foot of building, as compared with the 3*s.* and 5·17*d.* given in Mr. Ellis's Paper. The cross-sectional percentage of steel in the columns varied Mr. Ball.

Mr. Ball. from 5·28 per cent. on the bottom floor to 1·22 per cent. on the top floor, the columns being reinforced with round bars tied together at intervals but not hooped spirally, the maximum load being about 300 tons, as compared with percentages ranging from 5·70 to 2·2 in the warehouse at Shanghai, the maximum load there being 376 tons, and the columns carrying this load being hooped. The granary was supported on timber piles, driven in groups in positions underneath the columns of the building. The heads of the piles were surrounded with concrete bound together with Clinton wire mesh. Referring to the Shanghai wharf, the cross-sectional ratio of steel to concrete in the main and secondary beams appeared to be about 2·8 and 1·35 per cent. respectively. He had had the percentages taken out in the case of the main beams of several bridges he had erected recently, and he found these to range from 3 per cent. in a girder 32 feet long, ratio of depth to span 1 : 15, to as much as 8·7 per cent. in a girder 80 feet long, ratio of depth to span 1 : 9; this latter girder was, however, very heavily loaded, being one of two main girders carrying a 36-foot main road. There appeared to be no doubt that, generally speaking, structures in reinforced concrete were less costly than those built in brick and steel. He had in mind bridges where carrying out the work in reinforced concrete had effected a saving of 15 to 30 per cent. on the estimated cost of a brick and steel structure. This, together with the reduction in the cost of maintenance—which was always necessary in the case of a steel structure—rendered the adoption of such structures worthy of consideration. Mr. Ellis referred to the cracking across several bays of the wharf. Mr. Ball had had several instances in which slight cracks, not at all deep, had occurred near the ends of bridge-girders, taking up a direction along the lines of shear. In one bridge, built on very bad foundations, the weight of the bank tipped behind the abutments forced up the ground between them, fracturing the cross ties which tied the heads of the foundation-piles together, sluing the bridge bodily round nearly 2 feet and cracking the girders in a number of places. The damaged work was repaired, and a concrete invert was built between the wings and abutments; since then no further trouble had arisen. Although water had got through where cracks appeared, there was no sign of rust on the faces, showing that there was no corrosion. The value of the monolithic nature of the work was well illustrated by this example.

Mr. Popplewell's Paper gave information of considerable interest and value, the question of the adhesion between steel and concrete in reinforced work being one which had given rise to considerable discussion, and had resulted in bars of various forms and sections Mr. Ball. being invented with a view to overcome what was thought to be a somewhat unsatisfactory state of things in the case of ordinary round or square bars embedded in concrete. It would, he thought, have been advantageous if Mr. Popplewell had extended his experiments and investigated what additional frictional resistance or adhesion was obtained where such special forms of bars were adopted. Valuable as such experiments were, when the results had to be applied to the design of actual works, Mr. Ball felt that they should be considerably discounted. In the first place, the conditions under which experimental work was carried out, and those obtaining in actual practice were very different. For example, Mr. Popplewell said, "The moulds were left in the vertical position and allowed to remain in a somewhat damp atmosphere for 14 days; after which, the boarding having been removed, the columns were kept in a damp position until ready for testing. During this last period they were frequently sprinkled with water." In actual practice there were continual changes of atmospheric conditions—varying from dry to damp, and from hot to cold and frost—and, although precautions might be stipulated, they were often imperfectly taken. The mixings of large quantities of concrete were also subject to variations which would not exist in the case of the small quantities required for experimental work: so that although tests made on concrete used in an experimental column might fairly indicate its strength in such a column, the result might be altogether too high or too low for portions of similar concrete used in different parts of a large structure. This was clearly shown by the diversity in the results obtained from samples taken from different mixings used on works; so that an experimental ratio of the moduli of elasticity of 19·8, such as that given in Mr. Popplewell's Paper, varying as it did from the accepted value of 15 (whilst, no doubt, correct for the particular columns experimented upon), ought not, Mr. Ball thought, to be used in calculating columns for actual structures, even though a similar concrete were used. The success of structures in reinforced concrete depended very largely upon the care exercised in carrying out the work; and very careful supervision was necessary in the mixing of the concrete, the correct disposition of the reinforcing bars, the actual placing of the concrete in position, the stability of shuttering, etc. These matters admitted of so much more perfection in experiments than in actual practice that he again ventured to emphasize the point that a considerable margin of safety should be allowed in applying the results of such experiments to practical work.

Mr. Etchells. Mr. E. F. ETCHELLS wished to draw attention to the method of failure of the pillars in each of the three Papers. Mr. Ellis's reinforced-concrete pillars were nine diameters in length and they failed by general flexure. It was an exceptional thing to have a pillar of only nine diameters fail by bending. The plain concrete pillars of the same length failed by direct compression, or, more precisely, by shearing along diagonal planes, giving the usual conical fracture. Mr. Burr's pillars were generally of fourteen diameters, and three out of the four of the mild-steel pillars failed by flexure, but none of the reinforced-concrete columns failed in that manner; these failed by bulging due to the bending of the steelwork locally. Mr. Popplewell gave the ratio of length to diameter for his columns as 16, calculated on the maximum diameter, and stated that he considered them short, and therefore failure by bending need not be considered. On examining his diagrams, in which the compressions were given at each of four points, it would be seen they were so regular that his assumption was fully justified. In the French Government Regulations permission was given to go up to twenty diameters, taking only direct compression; and it would be a serious matter if government regulations allowed up to twenty diameters while actual experiments showed that the pillars failed by bending at nine diameters. Mr. Ellis's results were so low as to suggest that there had been some eccentricity of loading to cause bending: such low figures as he recorded were not in accord with the general run of experimental results. On the other hand, Mr. Popplewell's tests had probably been carried out with very great care under laboratory conditions. Perhaps, therefore, Mr. Burr's experiments might be taken as coming fairly between the other two, and as representing the conditions which would be found in a general case. Referring for a moment to the type of Mr. Burr's pillar, it had been said that it was doubtful whether it was really reinforced concrete; it was quite as much reinforced steelwork. If engineers were free now to consider that it was reinforced concrete, it meant that another barrier had been broken down. For some time past pillars had been divided into two classes, those with straight-line hooping or laterals, and those known as hooped pillars. As the only detailed report published in England had broken down that barrier and allowed the two classes of pillars to merge one into the other, it would appear that reinforced-concrete pillars might be allowed to include the new type, although the reinforcement was excessive, and the cost, he should imagine, prohibitive for universal adoption. There was also another objection to that type of pillar, inasmuch as it was very difficult to fill in the concrete right up to the top so that the concrete inside at the joints of the beams and girders would be fully capable of taking the stress which the calculations showed. Of course, if there was a cavity at the top the concrete below did not get a chance to take the load; it only prevented the bulging of the steelwork. Mr. Burr gave the stresses on the angle pillars at failure as 16½ and 17 tons, and the ratio l/r as 34. The result was entirely anomalous, because on the channel pillar, for which the ratio l/r was less, the breaking load was less instead of being greater. Mr. Burr went on to say: "From this it will be seen that Nos. 1 and 2—the angle-bar columns with the usual lattice bracing—show ultimate resistances at failure distinctly higher than Nos. 5 and 6—the channel-bar columns without positive bracing between them, but with battens only. The advantage of holding the members of a column under compression by complete effective bracing is thus apparent." Mr. Etchells dissented from that entirely. Considering the individual elements of the pillar, it would be found that the ratio l/r was 25 in the first case, with an average breaking-stress of 16¾ tons, and in the case of the channels, taking the length of the channels between the bindings, the ratio was 50, thus accounting for the lower crippling stress of about 14½ tons. What Mr. Burr had shown, however, was the advantage of hooping the pillar. Mr. Etchells quite agreed that generally the braced pillar was stronger than the pillar with merely battens or plates. Professor Müller-Breslau, in a Government inquiry in Germany, had found the same action occurring. In that case it was not a laboratory experiment, but a test of a full-sized pillar of a gasholder; and unfortunately, it was not an intentional test, but an accident in which thirteen men were killed and forty-seven injured. That accident was attributed largely to unbraced pillars having unequal loading on the sides. On p. 103 Mr. Popplewell gave the yield-point of the steel as 18 tons, and the limit of elasticity as 12·5 tons. A difference of 5½ tons on ordinary mild structural steel certainly seemed considerable, if not exceptional. Since proposals had been put forward to base the working-stress on the elastic limit, and also proposals to take one-half the yield-point, Mr. Etchells submitted that in the present case it made a great difference whether the half of 18 was to be taken, or the half of 12. The Table on p. 111 showed a yield-point ranging from 7·5 to 7·96 tons, and Mr. Etchells would like to ask whether that yield-point, which was about 8 tons, was the same yield-point as the one mentioned on a previous page. Mr. Etchells.

Mr. Went-worth-Sheilds.

Mr. F. E. WENTWORTH-SHEILDS considered it a great advantage to The Institution to have three such Papers before it, as they gave rise to very interesting comparisons. After all, in reinforced concrete the great question to a practical engineer was whether the formulas he used were trustworthy and whether the assumptions made in deriving those formulas were correct; and, if not, how far would the fact that those assumptions were not correct affect the results, by making the structures either unsafe or too extravagant. The strength of the reinforced-concrete columns tested by Mr. Popplewell could be calculated by the ordinary rules, such as those laid down by the Royal Institute of British Architects; and assuming that the concrete would bear safely 500 lbs. per square inch, and that *m*, the ratio of the modulus of elasticity of the concrete to that of the steel, had a value of 15, the column would bear safely a load of about 13½ tons. A great deal of thought had been given to the question whether it was right to assume, as the R.I.B.A. report did, that *m* had a value of 15. It was interesting to notice that, although values for *m* had been suggested that evening ranging from 20 to 10, the difference in the safe strength of the column was not varied very much by taking those different values. For instance, if *m* were assumed to have a value of 20, the safe load on Mr. Popplewell's columns would be 15½ tons; while if it were 10, the safe load would be 11½ tons. Even with that very wide variation of 100 per cent. in the value of *m*, therefore, there was only a variation in the calculated safe strength from 11½ to 15½. In all cases there was a substantial factor of safety, and he thought there need be no great anxiety if the mean of those three values, namely 15, was taken for *m*. In such a column as the composite column described by Mr. Burr the difference that arose from assuming various values of *m* became considerably more marked. For instance, in the design *a* (*Figs. 1*, p. 117) if *m* was assumed to be 20, the safe load—again assuming the safe stress on concrete to be 500 lbs. per square inch—worked out at just over 29 tons; when *m* was taken as 15, the safe load worked out at 24 tons; and when *m* was 10, it worked out at 18·7 tons. On account of the large proportion of steel a fairly wide range of values for the safe strength of the column was found. It was satisfactory to notice, however, that even with a composite column of this type the ordinary rules gave a safe strength which was one-fourth, or less, of the ultimate load. Another point much debated by users of reinforced concrete was the bending-moment assumed for calculations in the case of a continuous reinforced-concrete beam. It was customary, and no doubt wise, to assume that the bending-moment was rather greater than theory would suggest, owing to the fear that settlement might take place, Mr. Wentworth-Sheilds.

or that the continuity might be imperfect in some way. To his mind that was really one of the points on which practical men had need to be most cautious, and with regard to which ignorance on the subject of reinforced concrete was greatest. Engineers did not really know what practical conditions were necessary to ensure perfect continuity in beams. It was supposed that if the bars overlapped the point of support to some extent, say, 80 diameters, or according to the rules laid down now by the London County Council—there would be perfect continuity, that was, that the tension on one bar would be transmitted through the concrete to the adjoining bar. It was supposed that this was so on account of certain experiments which had been made to show the intensity of the frictional grip between concrete and steel; but he thought he was right in saying that no satisfactory experiments had been made to show the frictional grip between concrete and steel under the circumstances in which they were placed in the case of continuous beams, namely with bars overlapping one another and lying side by side. He hoped that when the Institution Committee on Reinforced Concrete had finished the work they were now engaged upon, they would turn their attention to that matter and carry out some experiments which would allow engineers to speak with more certainty as to what was the proper design in order to ensure continuity in continuous beams. With regard to the wharves at Shanghai, he thought Mr. Ellis was an exceedingly bold man. He had put a reinforced-concrete platform on groups of four piles, which were calculated to stand a load of 45 tons; he had loaded those piles with 45 tons and found them to go down; none the less his sleep was undisturbed. For himself Mr. Sheilds confessed he would be a little uneasy if he had a platform on piles which were known to be liable to settle under the maximum working-load; though it was much easier to say that than to suggest something better. He was filled with admiration for the way in which Mr. Ellis had overcome the very difficult problem of placing a safe wharf on soft silt. In the case of the wharf-wall, the top of which was shown in Fig. 9, Plate 2, Mr. Ellis said that he put the backing behind it before the ties which secured the wall back to the foundations of the heavy building behind were put in, and consequently the wall, not being stable enough, moved forward slightly. He then removed the backing and the wall came back to its original position. Mr. Sheilds had noticed the same thing in the case of retaining-walls, which moved forward slightly during a very low tide, when the water-pressure was small, and moved back into place when the tide came in again. It rather looked as if the supporting earth sustained elastic deformation and was able to some extent to regain

Mr. Went-worth-Sheilds. its former shape and replace the wall. He thought all the members would envy and admire the very low unit cost of the work described. He believed very few engineers would be able to carry out reinforced-concrete work for 3*s.* 7*d.* per cubic foot. The low cost was partly explained by the fact that unskilled labour in Shanghai cost 10*d.* to 1*s.* per day, but it would be interesting to have the cost of the cement and the steel on the site, from which it would be easy to make some rough analysis of the total costs.

Mr. Maurice Fitzmaurice. Mr. MAURICE FITZMAURICE desired to say how much he appreciated Mr. Ellis's Paper, because the particulars given, such as the information with regard to settlement, the methods of carrying out the work, and the costs, were exactly the information required by many members of The Institution, and certainly what he wished to have himself. He thought The Institution had been very fortunate during the last two or three years in getting a number of its American members to bring forward Papers, and it was indebted very much to Mr. Burr, who was a prominent engineer in New York, for giving the information about columns that had been tested at the great Columbia University, where he was Professor of Civil Engineering. He was sorry that Mr. Meik and others had rather criticized some minor points relating to the columns. He did not think it mattered whether they were called reinforced concrete or reinforced steel, because the members knew exactly what was being spoken about; the Paper gave illustrations of the columns and exact details of their construction, and stated the quantities of concrete and steel that were being dealt with. The question whether such columns were economical or not depended, of course, entirely on where they were built. These columns appeared to take up the minimum of space, and where land was almost unobtainable at any price that was a very important matter. Some people seemed to forget that in New York, where such columns were used, the city was growing upwards, and not sideways, and it was absolutely necessary to have columns which could be built quickly, took up little space, and were capable of carrying very heavy loads. When it was remembered that twenty stories was there considered an ordinary height for a building, he thought nobody would think of using what was generally called in England reinforced concrete for the main columns if they desired to get such a building done quickly. Therefore the members were indebted to Mr. Burr for giving The Institution the information, which he might have given to some of the many engineering institutions in America. He wished to make some remarks on the conduct of experiments and tests in connection with constructional work. For some time he had been taking note of the experimental

Mr. Maurice Fitzmaurice.

Papers presented to The Institution. Unfortunately there were not many, but there had been a few, and some of them were of great value; others, however, in his opinion, were worthless—although, of course, there might be two opinions on that point. He could not help comparing the three Papers under discussion with others which The Institution had received recently, and which, as far as he could see, did not add much to existing knowledge, and certainly did nothing to influence the current practice of engineers. The present Papers were of a totally different character. They caused everyone to think carefully about reinforced-concrete work. There were so many advocates of different ways of dealing with reinforced-concrete construction, and so many theories based on very little experience, that the information in the Papers before The Institution might make engineers reflect whether they knew as much about reinforced concrete as some people supposed. The moral of the Papers under discussion and of those he had heard recently—if a moral could be drawn from them—appeared to be that, as far as possible, experiments of the kind referred to—in connection with constructional work—ought to be carried out by engineers dealing with that class of work. If the experiments were not carried out by engineers who were actually dealing with such work, the experimenters, if they had had no practical experience, ought to consult, to a certain extent, men who had practical experience, so as to be sure that the experiments justified the expenditure on them and that the results likely to be obtained from them would be useful to engineers. He was not one who objected to spending money on experiments. On the contrary, he would always advise The Institution, and any other body he might be connected with, to spend money freely on experiments. He would go farther and say that, when The Institution had got over the financial effort of constructing its new building, he would be very pleased if it were to set aside annually a sum of money to be given to selected resident engineers on large works who were in a position to make experiments which might yield useful results, and who would be able to do work which would give valuable information to the members. If there was one thing he detested it was seeing money spent on experiments which were quite useless to members of The Institution or to anybody else.

Mr. Read.

Mr. R. J. G. Read remarked that it was very satisfactory to find that an assumption always made in designing columns and other members in reinforced concrete was verified by Mr. Popplewell's experiments. One thing that required further investigation was the amount of concrete that must be connected with the steel in

Mr. Read. order to transmit the pressures. He had often noticed a number of reinforcing rods bundled together so as to leave little room for concrete between them, and it seemed to him doubtful whether the full value of the steel could be utilized under those circumstances. Mr. Popplewell had measured his compressions to $\frac{1}{1,000}$ inch on a length of 12 inches, and it would be interesting to know if there was any difference in the nature of the result when the compression was measured over a longer length. Mr. Burr had measured his results to $\frac{1}{10,000}$ inch on a length of 70 inches. The result of Mr. Burr's tests of the relative strength of columns made of steel alone and of steel filled with concrete was about what engineers had been accustomed to accept, namely, that the composite columns would bear about 50 per cent. more load than the plain steel. Mr. Read would like to know how much of that extra strength was relied upon in the calculations for the columns in the high buildings in New York referred to by Mr. Burr. Engineers were accustomed to use in the steelwork of buildings riveted stanchions and columns of various forms, which were filled with concrete, but the extra strength due to the concrete was seldom counted upon in the design of the columns. In the columns of the new building of The Institution sections of various forms would be filled with concrete, and no doubt they would be very much stiffened; but that extra stiffening had not been taken into account in calculating the strength of the steelwork, which was sufficient in itself. Comparing the results arrived at by Messrs. Popplewell and Burr with regard to the compression of the columns, he found that Mr. Popplewell compressed a true reinforced-concrete column $\frac{14}{1,000}$ inch in 12 inches, while Mr. Burr compressed plain steel columns $\frac{64}{1,000}$ inch in 70 inches. If the proportions were the same, that would be about $\frac{11}{1,000}$ inch in 12 inches, while the composite columns were only compressed $\frac{6}{1,000}$ inch in 12 inches, so that Mr. Burr's columns apparently did not shorten as much under the load as those experimented upon by Mr. Popplewell. In calculating these strains he had taken the limit of load mentioned by Mr. Popplewell (50 tons). The columns were not broken, but the compressions under that load were given, and similar data could be obtained from the curves given by Mr. Burr. He did not think the comparison was quite fair in regard to the stiffness of reinforced-concrete—in the usual acceptation of the term—and composite columns. In the reinforced-concrete column the resistance of the steel might be taken in the usual way as fifteen times that of the concrete for equal areas. Comparing the columns in that way, the equivalent concrete section of Mr. Popplewell's column would be 60·6 square inches, while Mr. Burr's steel columns would be 60 and 71 square inches without any concrete, and with concrete 92 square inches. Therefore the latter had a more effective area, and the compression consequently did not show as much. Mr. Read.

With regard to the Paper on the Shanghai wharves, he thought that the piles which were lengthened and re-driven might well have been left as they were, considering that it took eighty blows to start them. They must have been thoroughly set; and as the working-load on each of the four piles was only about 11 tons, it would appear that they were quite capable of supporting their load. The trouble in lengthening piles was the delay, the piles having to be left about 6 weeks before they could be re-driven. It was often very difficult to tell exactly what length of pile was required for a given site. In one instance in his own experience he had a sample pile made and driven as far as it would go. The remaining piles were then made of the length thus indicated, but not one of them stopped at the same depth: the sample pile had gone down into the only hard piece of ground on the site. The other piles had to be driven 8 to 10 feet below the surface of the ground, but as they carried columns it was possible to excavate down and build up, and thus carry on the work without much delay. He noticed that the surface of the concrete slab under the warehouse at Shanghai was at the bottom and the stiffening beams formed ribs above. That arrangement had the advantage that it allowed one to see what was being done. When the concrete had set and the shuttering was cleared away the beams were visible. On the other hand, it was often more economical to turn the slab the opposite way, leaving the flat surface to form the floor of the warehouse, in which case the stiffening ribs were underneath and inaccessible. He had found it a good plan in soft ground to cover the surface with dry brick rubbish rammed well down, instead of using a layer of lime concrete as at Shanghai. The brick rubbish absorbed moisture. The channels for the beams were made by building rough trenches in brickwork. The reinforcement was then put in, the channels were filled, and the floor was put on. In a warehouse upon which he was now engaged—very similar to, but not so large as, those at Shanghai—there was a lift taking up vans and wagons to run on a balcony and discharge the load inside. The balcony was carried on cantilevers, without the pillar used at Shanghai, except in one case, in the lift-well. The columns at Shanghai were very heavily reinforced and were apparently on the Considère system, having spiral reinforcement, but it seemed to him

Mr. Read. that the ratio between the cross-sectional area of the vertical bars and that of the spirals was much higher than was usual with the Considère system. He made out that in the columns of the bottom floor there was about 9 per cent. of steel reinforcement, in the next tier of columns about $4\frac{1}{2}$ per cent., and in the upper tier of columns again about 9 per cent. He did not understand why there should be such variation. He believed it would have been better to increase the size of the spiral bars and decrease the size of the verticals. He could not understand from the illustration why the joints of the vertical bars had been made below the floor. He generally endeavoured to get the joints above the floor, so that the concrete could be filled into the columns and beams above them at the same time, right up to the floor-line, leaving the vertical bars standing up for a certain length above the floor. Then the next tier of columns could be placed on that, and so on. He thought there was some danger in not filling the columns and the beams right up to the floor-line. Fig. 13, Plate 2, showed the main beams splayed out from the columns on each side. Sometimes columns were filled up to the bottom of that splay, where the main girder began, and left there. Not long ago he noticed in a warehouse, after the shuttering had been taken down, cracks around several of the columns, just where the beams sprang out. He was rather concerned about it, thinking the cracks were due to stresses set up in the structure; but on examining them closely and cutting them out he found they were filled with a layer of fine coal-dust and sawdust, which in some of the columns was nearly $\frac{1}{4}$ inch thick. The cause of this was that the columns had been filled up to that point and left, and dust had been blown along the troughs into the depression; it had then been covered by the concrete, forming a serious separation between the two portions of the column. He had the cracks carefully chiselled out and grouted up with good cement mortar. The cost of the superstructure of the warehouse at Shanghai was given as 3*s.* per cubic foot of concrete. Lately he had had some tenders for a similar but smaller building, and in the lowest of the tenders the cost came out to about 2*s.* 7*d.* per cubic foot of concrete. Reinforced-concrete structures were often put out to tender on various systems, and it seemed to him important that the tenders should be made on the same basis if possible: all tenderers should have the same information. Therefore it was very necessary for engineers to design the work themselves as far as possible—not in every detail perhaps, because there were now many firms who had given their whole attention to that particular class of work and were much more expert in designing details than an engineer would be who had to deal with a reinforced-concrete structure among other work. But certainly the sections of the beams and of the reinforcement should be given. That, of course, was connected with the much-discussed question of the bending-moments, which should be defined for the supports and in the middle. If that procedure were adopted, he thought the tenders would come in on a much more even basis than was often the case now. If the amounts differed by 50 or 100 per cent., there could not be the same quantity of material allowed for in the tenders. Mr. Read.

Mr. F. HUDLESTON remarked that the columns of the new building of The Institution, referred to by Mr. Read, had been designed under the London Building Act of 1909, under which no consideration could be given to the strength of the encasing material. He noticed in all three Papers a good deal of similarity in the stresses taken as working-stresses. Mr. Popplewell had used a design in which the calculated stress in the concrete under compression was 437 lbs. per square inch, and Mr. Ellis had allowed in his plain columns, which were similar to those Mr. Popplewell had used, 450 lbs. per square inch. Both of those figures were about three-quarters of the maximum mentioned in the rules drawn up by a Committee of the Royal Institute of British Architects, which the London County Council proposed to incorporate in their regulations. Mr. Burr suggested that for composite columns the stress might be 500 to 750 lbs. per square inch, and mentioned that his crushing load came out at about 1,500 lbs., or three times to twice the stress he proposed to use. Again, Mr. Ellis in his hooped columns had used a limit of 1,200 lbs. per square inch for the concrete inside the helical winding. The R.I.B.A. rules fixed a limit depending upon the amount of winding put on, up to a possible maximum of 1,600 lbs., with 1 : 2 : 4 concrete and any amount of spiral bracing. That was 33 per cent. higher than Mr. Ellis's working-load; which seemed to show that at present most engineers designing such work were content with stresses a good deal lower than those which the R.I.B.A. had adopted as the maximum. He thought it was rather a mistake that the London County Council should have adopted somewhat higher figures than those, because when regulations were made everyone worked practically up to the limit at once: he was bound to do so, because he was in competition with others. Personally he thought the R.I.B.A. limit in that matter was a little high. When tensile stress was considered the matter was seen to be somewhat unsatisfactory. The London County Council gave 17,000 lbs. per square inch as the maximum tension to be allowed Mr. Hudleston.

Mr. Hudleston. in the steel of tie-rods. When an actual stress exceeding 20,000 lbs. per square inch was put upon such rods, the concrete around the bar would probably crack. Taking the test-loads in general use, it was very usual to put on 50 per cent. more than the average live load calculated for, and a very common proportion was one-third dead load, and two-thirds live load. With 3 cwt. per square foot the bars might be strained in tension up to 17,000 lbs. per square inch, but 16,000 lbs. was the limit of the R.I.B.A. If 50 per cent. were added to the live load it would mean a total load of 4, so that the stress on the bars was being increased by one-third, bringing it up to 22,667 lbs. per square inch. He thought that was dangerous, and that if there were limits of stress they should be limits which were not to be exceeded in any test-load put directly on a floor. With regard to the factor of safety, it was distinctly higher in all the cases under discussion. With the plain columns, Mr. Popplewell had a factor of safety of at least 4, his columns being calculated for $13\frac{1}{2}$ tons and showing no sign of breaking at 50 tons. Mr. Ellis's hooped columns were calculated for 1,200 lbs. per square inch, which gave a total of about 78 tons per square foot. In the column Mr. Ellis tested, the working-load by his rules would have amounted to 44 tons and the breaking-load was 139 tons. Mr. Marsh had already referred to the question whether the factor of safety should be referred to the time when the column went to pieces or when it first showed signs of cracking; and Mr. Hudleston agreed that when a column showed signs of cracking it had about reached its limit, and the factor of safety ought to be taken then. That would give in Mr. Ellis's case a factor of safety of about 3. There was another feature of the L.C.C. rules which he thought was a mistake. In them were incorporated a number of rather complex formulas taken almost bodily from the R.I.B.A. rules. On the other hand, Clause 32 said that where the conditions of loading were not specified the bending-moment of the stresses on the slabs or beams must be calculated by the accepted formulas of modern engineering practice. He submitted that it would have been far better if a rule of that description had been inserted in place of all the formulas given. These formulas were not quite the same as those issued 3 years ago by the R.I.B.A. Committee, and it was quite possible that a few years hence they would be altered again. He believed it was a mistake to introduce in such regulations very complex formulas which practically tied the hands of everybody. Again, the preamble spoke of the regulations as referring to buildings in which the stresses were transmitted through each story by a skeleton framework of reinforced concrete or partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls. It would be a good thing now to have such rules drawn up in a way that would enable steel columns to be combined with reinforced-concrete walls, because it would give a distinct gain in floor-area, which in London was very valuable. Even the most bigoted enthusiast for reinforced columns would admit that a steel column to carry the same load could be made a great deal smaller, that reinforced columns for heavy stresses were rather clumsy, and that they took some time to attain their full strength. If it were possible to combine the two in the new Building Regulations, or to bring in some regulations which would enable the designer to use them without asking for special leave from the London County Council, it would be a great advantage to the profession generally and to all who were concerned in designing buildings. He quite agreed that limits of stress should be given, but he wished to see them rather lower. He suggested that all the formulas which were incorporated in these L.C.C. rules would be better left out, and that a general reference to the R.I.B.A. rules would be much more satisfactory to the profession at large. Mr. Hudleston.

Mr. H. E. Steinberg observed that Figs. 10, Plate 2, of Mr. Ellis's Paper, showed the stirrups to be formed of thin bars twisted together: it seemed to him that this arrangement must have given a great deal of trouble, and he could not conceive any advantage in it. The stirrups in a reinforced beam apparently acted in tension, although they were calculated in shear, and two bars twisted together did not form a very good arrangement for tension, the extension being very great in proportion to the stress. The Paper gave the details of one of the main first-floor beams in the warehouse and showed how a great deal of reinforcement was put over the supports. He thought that if Mr. Ellis had added the eight $2\frac{1}{4}$-inch bars in the columns as well, it would have shown how very complex the armouring at that point was: it must have been very difficult to get the steel into position and to deposit the concrete. With reference to the spiral columns, Mr. Ellis stated that he had adopted the ordinary ratio of 15 between the moduli of elasticity, but he also stated that the spiralled concrete was limited to 1,200 lbs. per square inch inside the core and the steel in compression to 12,000 lbs. per square inch. Surely that was a ratio of 10, not 15. Adopting the new London County Council rules for working out the columns, and taking first of all as the dominant factor the limit of compressive stress in the steel, 12,000 lbs. per square inch, then in the spiralled concrete there would be 800 lbs. per square inch, and the quantity of Mr. Steinberg.

Mr. Steinberg armouring provided ($\frac{1}{2}$-inch bars at $2\frac{3}{4}$ inches pitch) would be sufficient to raise the plain concrete from a stress of about 560 lbs. up to 800 lbs. per square inch. It might thus be taken that the basis stress on the plain concrete was 560 lbs., which by the addition of the lateral armouring was enhanced to 800 lbs.; and the longitudinal steel reinforcement, at fifteen times that stress, namely, 12,000 lbs. per square inch, added thereto, would give the total load. But then the total load would be only 303 tons, instead of 376 tons. Looking at it in another way, and adopting the other figure given by Mr. Ellis, namely, 1,200 lbs. per square inch, as the stress on the spiralled concrete, fifteen times that would be 18,000 lbs. per square inch, and the quantity of armouring in the column would indicate that the basis stress on the plain concrete was about 840 lbs. per square inch, which was high. In any case he would like to emphasize what Mr. Read had said, that the columns were not of economical design. It would have been much better, from the point of view of economy and other practical considerations, if the amount of the spiral armouring had been increased and the diameter of the longitudinal armouring had been decreased. By adopting the proportions recommended by Mr. Considère for that type of column, and assuming a reasonable limit for the steel in compression (16,000 lbs. per square inch), and a reasonable basic limit for the plain concrete in compression (600 lbs. per square inch), it would be found that $\frac{3}{4}$-inch spiral bars at about $3\frac{1}{3}$ inches pitch would give about the right amount of spiral reinforcement. The longitudinal armouring might then be reduced from 31·8 square inches, as given by Mr. Ellis, to 21·8 square inches, and the total effect would be quite a considerable reduction in the cost of the column. There was another point he wished to raise with reference to the L.C.C. regulations. As framed they ignored the cover of the concrete outside the spiral. Mr. Ellis stated in his Paper that in the square columns the cover had been counted; but in the spiral columns, for some reason which was not very clear, the cover had been ignored. In a column of such proportions as those between the ground floor and the first floor, by ignoring the cover about 32 per cent. of the gross area of the column was neglected. Subtracting the area of the core from the area of the octagon gave 182 square inches, and taking 450 lbs. per square inch, as adopted by Mr. Ellis, the cover itself was capable of bearing about 37 tons. It would appear to be more reasonable to say, in the L.C.C. regulations, that the area of the concrete inside the spiral might be stressed to a certain figure dependent upon the amount of the spiral armouring, and that the area of the concrete outside the spiral might be taken into account up to some arbitrary stress, e.g., one-quarter of the crushing strength of the plain concrete. Otherwise, with a small column, say, 10 inches square, it might be better to ignore the lateral armouring altogether, and simply take the 10-inch square and disregard the permissible increased stress on the concrete inside the spiral. With regard to the making of the spiral columns, it seemed to him a very unnecessary procedure to lay the bars laterally round a mandrel and then wind the spiral on them, because directly the spiral was wound it would spring open and the wiring of the longitudinals would have to be done over again. The proper thing to do was to adjust the mandrel to such a size that when the winding was finished the spiral would spring open to the size required. With regard to the wharf, he would like to ask whether it would not have been more reasonable, in view of the fact that the factor of safety of the piles which supported the structure approached unity, to adopt a rather low factor of safety for the superstructure. Why put a structure with a factor of safety of 4 or 5 on the top of piles which were in a foundation that had practically no factor of safety at all? Mr. Steinberg.

Mr. M. E. Yeatman wished to utter a protest against allowing too great a reduction in the bending-moment for continuity in beams. Mr. Ellis allowed $Wl/12$, where $Wl/8$ would be the moment of an independent beam. One speaker had made out the theoretical figure to be $Wl/13$, but it was by no means safe to take such a large reduction as that. The practice of American and French engineers was to take $Wl/10$. The point was discussed theoretically in Messrs. Turneaure and Maurer's "Principles of Reinforced Concrete Construction"[1] (p. 238), where were shown the moments on two spans and three spans continuous, for various distributions of the load. It had to be remembered that it was necessary to take into account not only a uniform load over all the spans, but the possibility of every alternate span being loaded, or all but one span being loaded, or one span only being loaded; it was necessary also to consider the greatest possible moment, both at the centres of the spans and over the supports, and the reverse moments over the supports were in general greater than the maximum moments in the centre of a span. Therefore, all things considered, the maximum possible theoretical moments were not very much less than $Wl/10$, and that gave a theoretical moment which might be largely upset by any slight unequal settlement of the supports. So that altogether $1/10$ was as far as it was safe to go. That was Mr. Yeatman

[1] New York, 1908.

Mr. Yeatman. accepted as general practice in America and also, he understood, in France. Mr. Considère, at the International Engineering Congress at St. Louis in 1904, stated that that was the usual French practice. As to Mr. Popplewell's experiments, with all due respect for the care with which they had been made and the accuracy with which the minute compressions had been measured, it seemed to him that they were rather tests of the apparatus than of the behaviour of the steel and the concrete. How could steel and concrete do anything else towards the centre of the span than strain uniformly? If it had been desired to test unequal compression the measurements should have been made at the ends of the column, and then some slip might have been detected. He thought Mr. Popplewell's tests of the grip in concrete were very valuable and interesting. He had been accustomed to require an embedment of fifty times the diameter of a rod before its full strength could be taken up, basing that on an allowance of 75 lbs. per square inch of shearing stress between the steel and the concrete, and 15,000 lbs. per square inch tension on the rod. The experiment pointed to the conclusion that it was useless to allow more than about thirty times the diameter, because the rodding would begin to pull out by contraction of its area, and would pass its yield-point before a greater length than thirty diameters could take effect. The point was that, for a span of a given length, unless anchorage was provided at the ends of the rods, there was a maximum diameter of rod that would give the necessary area. If fifty diameters were required, for a uniform load the diameter of the rods must not be more than $\frac{1}{200}$ of the span, and for a concentrated centre load not more than $\frac{1}{100}$ of the span; but if thirty diameters were allowed it would bring it up to $\frac{1}{120}$ and $\frac{1}{60}$ of the span, which would be a great advantage.

Mr. Lyster. Mr. ANTHONY G. LYSTER thought it might be interesting to the members to hear what had been done with reinforced concrete in the port of Liverpool, though as he had not expected to take part in the discussion, and had not refreshed his memory in regard to details, he must speak in a general way. His attention was first directed to this form of construction about 13 years ago, when the problem he had to consider was the construction of a cattle-wharf in the Mersey, adjoining the pier-heads. Such a wharf, overhanging the river and needing to be washed down periodically and kept thoroughly clean and sweet, was not easy to provide for with ordinary materials, and therefore he naturally considered the question of the suitability of reinforced concrete for the purpose. He limited the use of reinforced concrete to the platform, however, as in those days the piles were less trustworthy and much more expensive than they were now. The materials used in the concrete were somewhat rough and coarse, and perhaps at that time the methods of construction left something to be desired, but the work was fairly well done. The steelwork had been exposed to the action of the salt air, and there had been some rusting in patches; but when the defects came to be measured in money the expenditure on repairs was very small indeed, and the wharf had proved entirely satisfactory. The piles were of greenheart. Following that he constructed two other quays which were covered with sheds. Those quays were designed to take a working-load of 3 tons per square yard, and each consisted of a reinforced-concrete platform on greenheart piles. They were tested with a 50-per cent. overload and found to be thoroughly satisfactory, the deflection being within what was prescribed. Another wharf was constructed in which reinforced-concrete piles—the first used in Liverpool—were employed. The bottom was rock, and long spiked shoes were put on the piles. That structure also had been entirely satisfactory. In the later structures, with the experience already gained, greater care had been exercised in the selection of the aggregate, and in the careful disposition of the steel rods and stirrups and bars, and the work, he thought, was of a better character. The next structure was a three-story shed, 628 feet in length and 78 feet in width, on the quay of a dock. The first floor was designed to take a load of 25 cwt., and the second floor one of 16 cwt., per square yard. The roof was flat, the front part of it being reinforced to carry cranes, and the general area strong enough to carry a snow load. One remarkable feature of the structure was the use of cantilevers stretching out over the roadway, which was extremely narrow. An alteration of the dock-quay on the other side of the roadway was contemplated, which would enable the construction of a wider shed in the future, and in order to take full advantage of that he designed the two upper floors to overhang the lower floor by about 16 feet. The cantilevers had to carry all the weight of the upper structure of the two floors as well as their loads. They were tested to 50 per cent. overload, with entirely satisfactory results. After that he constructed another wharf to carry a working-load of 3 tons per square yard, very much on the lines of the others. What he would like to lay stress upon more than anything else, from the point of view of the practical treatment of reinforced concrete, was the extreme accuracy and care with which the work had to be done. The materials, particularly the sand and the broken stone, had to be selected and proportioned with great Mr. Lyster.

Mr. Lyster. care. The disposition of the bars in the girders and columns also required to be very carefully attended to, and steps had to be taken to prevent the bars from being disturbed during the process of concreting. In that respect he thought finality had not yet been reached, and he believed very considerable improvements were likely to be made. His attention had already been drawn to forms of reinforced concrete in which arrangements were made for keeping the bars at their proper distances apart and preventing disturbance of the various constituent parts during the process of construction. He was sorry his memory did not serve him well enough to allow him to give details of costs, but he could say that in the case of the three-story shed he obtained tenders for a steel and brick structure of the ordinary form, as well as for reinforced concrete, and there was a difference of about 20 per cent. in favour of the latter. Alternative tenders recently received for a double-story shed showed a saving of about 30 per cent. in favour of reinforced concrete. A point of great importance had been mentioned by Mr. Yeatman, namely, the reinforcement of continuous beams. That was a matter to which Mr. Lyster's attention had been directed quite recently in considering the designs of a structure on which he was now engaged. No allowance seemed to be made for want of continuity in the system of bars. Obviously where credit was taken for reduction of the bending-moment in a span owing to its being continuous, there must be certainty that the continuity of the system of reinforcement was maintained over the points of support; and that was a very material point, to which those who had to deal with reinforced-concrete structures should devote their attention.

Mr. Thorpe. Mr. W. H. THORPE observed that Mr. Burr's experiments showed an increment of strength of about 45 per cent. in the steel column when concreted. The utility of that method as a means of making the best use of steelwork had already been remarked upon. In a Paper read about 4 years ago[1] on "The New York Rapid-Transit Subway," by Mr. Wm. Barclay Parsons, M. Inst. C.E., experiments were cited which Mr. Parsons had made with rolled steel joists embedded in concrete, and from which he found as a result of the embedment—precisely from what cause he did not attempt to say—an increase of strength, as compared with the normal transverse strength of a steel joist, of about 70 per cent. at 1 month. Quite recently, in the new building-regulations of New York for reinforced-concrete structures—which came into force on the 1st January—it was enacted that where a structural steel column was encased in concrete, though the concrete

[1] Minutes of Proceedings Inst. C.E., vol. clxxiii, p. 83.

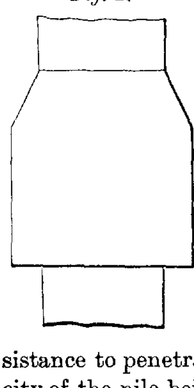
itself was not assigned any compressive resistance, the steelwork in that concrete might be stressed to 16,000 lbs. per square inch, up to a ratio of length to radius of gyration for the steel of 120 : 1. The safe load of such a steel column without concrete would be about 4 tons per square inch, as compared with 7 tons permitted under the new regulations in force in New York. Therefore it was evident that under these regulations the steel was credited with enhanced strength to the extent of about 75 per cent. If that was good practice—and he did not question it—and if the few experimental results which had been obtained were supplemented by others, and it was established that there might be, in the case of steelwork encased in concrete, an enhancement of the strength values, to be reckoned upon in design and permitted by the rules of the various authorities controlling such matters, it would give engineers a means of designing structures placing steelwork on a more even footing with reinforced concrete properly so called. Reinforced concrete was credited, as the result of a large number of experiments, with its full strength, and he saw no reason why steelwork should not also be credited in the same way with what increment of strength there might be when it was encased, and so placed on a more equitable footing with reference to reinforced concrete. Mr. Thorpe.

Mr. Popplewell's results were apparently satisfactory as far as they went, but, as had been pointed out, those displaying the behaviour of the steel with reference to the surrounding concrete at the middle of the column by no means settled the question. Mr. Thorpe had always had serious doubts whether the steel rods in a reinforced-concrete column really took up the stress in the way that was assumed. It might be so in the design tested, where the head and base of the column were enlarged, so that there was a considerable body of concrete around the rods, and where there were spiral hoops to increase the resistance of the concrete and its grip on the steel; but he questioned strongly whether it was so in columns as commonly designed, in which large bars were used and those bars were near the surface of the concrete. Between structural steel and reinforced-concrete work generally there was one marked difference. In ordinary steelwork there were just the same difficulties in the accurate determination of stress as there were with reinforced concrete, assumptions being made which, though convenient, were as little justified in one case as in the other. In a riveted framed structure the joints were treated as if they were hinged joints when they were nothing of the kind. Cross girders were treated as though they had hinged attachments when they were really very different. But no inconvenience commonly

Mr. Thorpe. resulted. In most steel-framed structures—he believed Sir Benjamin Baker once said in all steel structures—there was never an entire absence of overstress; some part of the structure was stressed beyond the elastic limit, though as a rule no trouble ensued. There was overstress in particular parts and there was yielding; but the deformation was not progressive; it could not be seen without the use of special instruments, and nothing happened. The structure looked none the worse, and commonly was none the worse. But reinforced concrete when first made was monolithic. There might be movements, reverse moments, stress where stress was supposed not to be, or possibly had been overlooked or considered too trifling in amount to be taken seriously, cracks occurring as a result. There was, he thought, no material, or combination of materials, in which any want of care would so remorselessly "give away" the designer as reinforced concrete.

Sir William Matthews. Sir WILLIAM MATTHEWS, K.C.M.G., Past-President, remarked that there were one or two practical points in connection with Mr. Ellis's Paper to which he might refer. First of all he could sympathize with Mr. Ellis in the great difficulties he had had to face in carrying out the work. Those who had had to encounter the difficulties of great depths of silt could appreciate the boldness and the skill with which the work had been designed and carried out.

Fig. 1.



In reading the Paper he had been reminded of a work with which he had something to do at Port Melbourne not long ago, and of what the Victorian engineers had done to increase the bearing-areas of the piles under the conditions found at that place, where a silt bottom only was available, which resembled in character to a remarkable degree the Shanghai bottom. The piles that had been generally used up to then were jarrah piles from Western Australia; they were driven with the small end downwards and chocks were bolted on so that as the pile was driven its resistance to penetration was considerably increased, the bearing-capacity of the pile being, of course, added to materially. It appeared to him that the system of reinforced-concrete construction lent itself to an increase of bearing-area of the piles in a very convenient way, and that by some such method as was shown in *Fig. 1* a considerable increase in the carrying-capacity of the piles might have been provided without any great increase of cost. The other point to which he wished to refer concerned the platform of the quay-wall. He had occasion about 2 years ago to examine a reinforced-concrete wharf of rather an extensive character, in which there was a platform similar to that at Shanghai. At dead low water he found he could penetrate in a boat right underneath the platform. The filling at the back of the work had sunk away from the deck, and in consequence of the platform the material could not follow it down from the top, with the result that there was a cavity underneath which admitted of a boat going right under the platform. He had risen, however, more particularly to say, as a Member of the Institution Committee on Reinforced Concrete, how very gratified the members of the Committee were to have Papers of this kind brought forward for discussion. He believed it was really the first time The Institution had had Papers on this important subject. There was no doubt that the use of reinforced concrete would spread; in fact, its use was only just beginning. His own firm intended to use it very extensively in the East, because they thought they would get advantages from its use. In carrying out the work they would be making some tests, and it would be their duty, as members of The Institution, to bring the results of those tests before their fellow-members. He quite agreed with Mr. Fitzmaurice as to the value of practical tests made by engineers on works, and he therefore hoped that when any tests were made relating to reinforced concrete—a form of construction as to which some uncertainty existed—they would be brought before The Institution for discussion and inclusion in the Proceedings. The members of the Committee were now engaged in making tests on slabs, and when those were finished they would proceed with tests on continuous beams, which were well worthy of investigation. All those tests required considerable time and entailed heavy expenditure, and he hoped the members would remember that, although the Committee had only issued so far a preliminary report, they were not inactive, but were busily engaged in making tests and had further tests in contemplation. As indicating the extent to which the information they had got together had been appreciated, he might mention that already about 4,300 copies of the report had been issued, and he understood that applications for it were still received pretty regularly. Sir William Matthews.

Mr. GEORGE H. T. BEAMISH remarked that, from what Sir William Matthews had said, it appeared that before long the members would have a general account of the origin and use of reinforced concrete, and he would like to call attention to an experiment made by Sir Marc Isambard Brunel, at the time of the construction of the Thames Tunnel, with reinforced brickwork, Mr. Beamish.

Mr. Beamish. Brunel succeeded in building a cantilever in brickwork about 60 feet long. It was recorded in the life of that eminent engineer,[1] and as a combination of iron and masonry Mr. Beamish thought it was well worthy of being brought to notice.

The President. The PRESIDENT remarked that he was very glad Mr. Hudleston had made some protest against incorporating very detailed rules in what was practically an Act of Parliament. One instance had been given already, in the course of the discussion, of the way in which such rules, even when they were not very minute, hindered the proper use of the materials by the engineer—namely, the regulations that had been made for steel-framed structures. By those rules engineers were precluded from taking into account the strength of the concrete in which the steel was encased. He did not wish to lay down the law, but it did seem an improper thing that when an engineer was encasing steel in a large quantity of concrete he should be told he must not make any allowance for the strength which the concrete added to the steel. He had been obliged to look a little into the proposed regulations of the London County Council for reinforced concrete, and he thought the practice of incorporating a large number of exceedingly detailed and minute rules reached almost a limit in those regulations. Reinforced concrete was a tolerably new material; although a certain amount of work and experiment had been done with it, and certain conclusions had been reached, it was impossible not to believe that, with further experience, very considerable modifications would be made in the methods of construction which had been adopted so far. When Mr. Considère wound a column with a spiral reinforcement he made almost a revolution in the construction of columns; he showed that the result was a large increase in the stresses which the concrete would carry. It was impossible not to believe that other modifications of that kind would be made. For instance, he had seen lately a modification in the usual method of constructing columns. He had no idea whether it was right or wrong, but it had been used in Scotland on a large scale in one very large building, and it seemed likely to present considerable advantages if it proved to be as good as its inventor supposed. But it would be absolutely shut out from use by the regulations proposed by the London County Council. He wished therefore to reinforce the protest Mr. Hudleston had made against engineers being bound down by too minute regulations. It was easy to see what the driving force had been. Concrete

[1] R. Beamish, "Memoir of the Life of Sir Marc Isambard Brunel," p. 285. London, 1862.

structures, especially buildings, had to be referred to the District Surveyors, men who were used to dealing, and no doubt very competent to deal, with buildings of the ordinary type, but who were less accustomed to deal with the new material; and therefore for their sake very detailed regulations were being laid down, in order that the responsibility might be taken off their shoulders. But it would be a great hindrance to the development of the new material. The President.

Mr. Ellis, in reply, desired to express his thanks to the President and the members for the very kind way in which they had received his Paper. With regard to the small factor of safety in the bearing-capacity of the wharf-piles, Mr. Wentworth-Sheilds had assumed perhaps too hastily that Mr. Ellis's sleep had been undisturbed by the unsatisfactory results of some of the trial loadings of the piles. The risk of subsequent settlement had, of course, been fully recognized, and had been guarded against to a large extent in the design of the superstructure. It must be borne in mind, also, that he had had the advantage of noticing the behaviour under working loads of a number of timber wharves at Shanghai—many of them less adequately supported—and also of watching pile-driving in the early stages of the work in question; thus being able to observe many encouraging indications of the ultimate bearing-power of the river-silt, though it would be difficult to put these on paper in the form of figures. At the present time, after a year's steady use, during which the wharf over the greater part of its area had been frequently stacked with cargo to its full surface capacity, though only locally with its estimated maximum load, no sign of settlement or distress could be observed. The local heavy loading mentioned in the Paper and referred to by Mr. Meik had not been made as a test, but had occurred, owing to some misapprehension of orders, during the ordinary course of working. It consisted of cargo stacked over a deck area of 40 feet by 40 feet, so as to bring a load of about 1,000 lbs. per square foot upon it. As the maximum distributed load had been taken as 350 lbs. per square foot this might be looked upon as a sufficiently severe test; and it was satisfactory to record that no settlement could be detected in consequence. He had been more anxious as to the behaviour of the front groups of piles under the 5-ton travelling crane, when it should be working for some time in one place, and so adding continual vibration to the effect of the maximum load on two piers. Happily, however, no sign of movement was visible there. The moderate unit-costs of the work were due chiefly to the low rates for labour in Shanghai. Credit must also be given, particularly in the case of the wharf, to excellent organization on the part of the contractors, who, it might be Mr. Ellis.

Mr. Ellis. mentioned, were not in any way reinforced-concrete specialists, but a firm of high repute for dock-construction in North Germany. In designing the work, also, simplicity and economy of construction had been constantly kept in view, sometimes at a sacrifice of theoretical economy of materials; and the various structures being on a large scale and of practically uniform design throughout, it had been possible to erect them with the minimum outlay on moulds and falsework generally. Contrary to Mr. Meik's experience, Mr. Ellis was convinced that a decided gain in time, cost, and quality of work was achieved by moulding on shore all wharf-members, except the pile-caps, beams, and deck, which were necessarily monolithic. The costs, on the site, of Portland cement and of steel bars, asked for by Mr. Wentworth-Sheilds, were approximately £2 13*s.* and £9 10*s.* per ton. The somewhat high percentage of steel in the beams, alluded to by Mr. Meik, was due not so much to there being an uneconomical proportion of steel in tension, as to the provision of double reinforcement throughout. The top bars served to resist reverse bending-stresses in a light span between two which were loaded, and also afforded a firm support for the upper ends of the shear members. The heavy reinforcement of the lower warehouse columns was necessary in order to keep their dimensions within the sizes suitable for working purposes. He was inclined to agree with Mr. Vawdrey that $\frac{1}{2}$ inch of concrete cover below the reinforcement of floor-slabs of a building was sufficient. A cover of 1 inch had been fixed for the expanded metal used at Shanghai, because this material was apt to assume a curved form when bent to pass over the supports, and it would thus come dangerously near to the lower surface of the floor at the centre of each span if no margin were allowed. Probably a minimum of $\frac{1}{2}$ inch to $\frac{3}{4}$ inch of cover had been actually obtained in the warehouse floors. In the wharf-deck, on account of its damp situation, 1 inch of cover had been rigorously required. With reference to Mr. Faber's inquiry as to the reason for taking $Wl/12$ as the bending-moment at the centre of the spans of interior beams, that figure had been adopted as a compromise between the theoretical positive bending-moment for beams truly continuous over three spans and that for freely supported beams. It referred directly only to the cross beams, which were designed for a uniformly distributed load, but applied also in principle to the treatment of the girders, the moment-curves being first worked out for continuous beams under the various conditions of loading alluded to by Mr. Faber. The fact that he gave $Wl/13 \cdot 1$ as the maximum bending-moment under the worst possible conditions, while Mr. Yeatman considered it dangerous to take a lower value than $Wl/10$, appeared to show

Mr. Ellis. that the adoption of $Wl/12$ as a safe mean had some practical justification, whatever might be its correctness in theory. The outside columns were not designed for eccentric loading so far as the arrangement of the reinforcement was concerned. They were rectangular in section, with the longer side in line with the wall, and had a comparatively larger cross-sectional area than the interior columns. With regard to Mr. Smith's suggestion that the wharf might have been further stiffened by a series of longitudinal diagonal members throughout, the provision of these had been considered, but not adopted, partly on the score of expense (nearly £2,000 additional), and also because of the undesirable multiplication of reinforcing rods thereby entailed in the pile-caps and girder-supports where so many members met. In the warehouses it would have been out of the question, from the owners' point of view, to introduce diagonals between the interior columns; and on the outside the existing panel-walls, of 15-inch brickwork in cement mortar, answered the same purpose, so far as a purely local stiffening could be effective. The foundation-grill was expected to be strong enough to withstand any local inequality of settlement, and, so far, had fulfilled this end. He deferred to Mr. Marsh's opinion that the actual maximum stresses in the first-floor columns of the warehouses would be about 1,000 lbs. per square inch on the concrete, and 15,000 lbs. per square inch on the steel. At the same time it appeared not unreasonable to assume that concrete, when confined by close steel hooping, had no longer the same physical characteristics under high pressure as when free to expand laterally, and that m, the ratio of the moduli of elasticity of steel and concrete, might in this case be more nearly 10, as the Author had taken it, than its normal value of 15. In the Table of tests of the wharf-beam, the figure $7\frac{2}{3}$ was not given as the factor of safety, the value of which would approximate to 4, as Mr. Marsh observed, but merely as the relation between the calculated safe superload and the test-load which caused actual destruction of the beam. The comparatively high unit-cost of the buildings, to which Mr. Ball called attention, was due to the expensive foundations, which accounted for nearly one quarter of the total cost. Mr. Etchells was possibly correct in his suggestion that there had been some slight eccentricity of loading in the column-tests. These had been carried out at the works in an ordinary hydraulic testing-machine, and although as much care as possible had been taken over the operation, the test could not be said to have been made under laboratory conditions. It must be remembered, also, that the ends of the tested columns, unlike those in the work, were not fixed in any way. He

Mr. Ellis. agreed with Mr. Read that the piles which had been lengthened and re-driven might well have been left as they were, considering that it had taken eighty blows to re-start them. For this reason only one group had been so treated. With reference to the varying percentages of steel in the warehouse columns, those carrying the first floor had had to be kept of uneconomically small diameter in order not to block working-space: this had entailed a high percentage of reinforcement. The columns carrying the second floor had been allowed to be of almost the same dimensions with a much lighter load, hence making possible a smaller percentage of steel, which was repeated in the columns next above. The joints of the vertical rods had been made below the surface-level of the floors, instead of above them, as Mr. Read preferred, in order to have the lateral support of the concrete floor and beams. This arrangement had not been found to cause any difficulty, the skeletons for the series of columns next above being fixed in place before each floor was concreted; and it gave a stronger joint. Both Mr. Read and Mr. Steinberg considered that the spiral hooping in these lower columns should have been stronger, and the vertical reinforcement correspondingly diminished. He did not follow their argument that sensible economy would have resulted thereby, so long as the diameter of the columns remained the same. The size of the hooping had been fixed at $\frac{1}{2}$ inch for practical reasons, that being the largest diameter of rod which allowed of easy winding round a skeleton of such small sectional area. Mr. Steinberg had apparently misunderstood the description in the Paper of the method of winding this hooping, as none of the drawbacks conjured up by him existed. When the $\frac{1}{2}$-inch rod had been wound round the main bars—a simple and expeditious operation—the skeleton formed a rigid framework ready for erection in place. Similarly, the stirrups of twisted wire to which Mr. Steinberg had objected were adopted on account of their practical advantages, as being readily clinched round the top and bottom reinforcing bars, and forming, with occasional struts, a securely adjusted skeleton. The justification for adopting a factor of safety of 4 for the superstructure of the wharf, while that of the piles was between 1 and 2, was found in the loading-incident already referred to, when cargo to the weight of nearly three times the calculated maximum load was stacked over several adjoining bays. The deck had only its own strength to rely on in such periods of exceptional stress, whereas any isolated heavy loading on the piles was distributed over three or more adjacent groups. He much appreciated the sympathetic remarks of Sir William Matthews as to the difficulties

Mr. Ellis.

of the foundations. The hardwood chocks which Sir William had suggested for increasing the bearing-capacity of the piles had been used by him in the case of a steel-pile wharf at Hong-Kong, where they had been found useful for checking excessive penetration into silt and sand. The silt there, however, was of a much stiffer nature (below the surface mud) than the ground at Shanghai, as shown by the fact that screw-piles had been used with success both in Hong-Kong harbour and up the Canton River, whereas at Shanghai they were an entire failure, hardly bearing their own weight. One reason for adhering to the use of plain rectangular piles was that the pile-drivers in use—of a special heavy pattern—were unsuited for driving piles having projections moulded on to them, or chocks bolted to their sides; and in view of the fact that it was apparently skin-friction alone which supported the load, the provision of this additional bearing surface did not appear to offer advantages counterbalancing its drawbacks. The tests showed that when once the mud gripped the concrete there was no further penetration of the piles in a group under any reasonable load. Settlement was caused entirely by the subsidence or compression of the stratum of silt in which the piles were embedded. It was the bearing-capacity of this stratum which determined the maximum load; and this bearing-capacity appeared to be greatest when the silt was compressed by the driving of 14-inch by 14-inch piles in groups. Twelve-inch square piles were experimented with first, but showed comparative inferiority in stability, owing, presumably, not only to smaller area of surface, but also to less severe compression of the silt about them. With reference to the platform of the creek wall to which Sir William Matthews had referred, it was unlikely that much settlement of the ground would take place here, owing to the row of close sheet-piles, the joints of which were grouted solid in front. In any case, the deck was supported by bearing-piles, and was designed to carry the weight of the superincumbent filling, so that no disastrous results should follow the formation of cavities beneath it.

Mr. Popplewell.

Mr. Popplewell, in reply, thanked the President for his remarks, and expressed his gratification at the way in which his Paper had been received and discussed. Referring to Mr. Meik's remarks, he had used the word "adhesion" in his Paper as suggesting some property peculiar to the concrete, enabling it to attach itself to the steel in a manner similar to that in which two pieces of wood were held together by glue. He had intended to indicate that in his opinion there was no such property, and that the resistance to the slipping of the bars was due to purely mechanical friction caused by

Mr. Popplewell. the fluid material setting against the rough surface of the steel and afterwards being forcibly held against the bar by shrinkage. The variation in the values obtained for the resistance to slip per unit of area might have been due partly to a difficulty in determining the length of the sliding portion of the bar beyond that which had been thinned by yielding. In this connection the figures in the following Table might be interesting as showing the frictional resistance of bars which, instead of being pulled out of the concrete, were pushed through it. The concrete in this case had the same composition as that described in the Paper.

RESULTS OF GRIPPING TESTS, IN WHICH SPECIMENS OF ROUND STEEL WERE PUSHED THROUGH PRISMS OF CONCRETE 6 INCHES SQUARE IN SECTION AND OF VARIOUS LENGTHS.

Size of Bar.	Length of Prism.	Age of Concrete.	Frictional Resistance in Lbs. per Square Inch of Surface.	Average.
Inch.	Inches.	Months.		
$\frac{9}{16}$	6	4	422	
,,	7	,,	471	
,,	8	,,	675	
,,	9	,,	481	587
,,	10	,,	590	
,,	11	,,	790	
,,	12	,,	660	
$\frac{3}{4}$	6	9	660	
,,	7	,,	642	
,,	8	,,	680	
,,	9	,,	520	635
,,	10	,,	505	
,,	11	,,	695	
,,	12	,,	743	

He failed to appreciate Mr. Meik's remark with regard to column C; it was stated in the Paper that column A failed at 44·5 tons, while columns B, D, and E, were loaded up to 50 tons, at which load they were probably not far from failure. It was not possible to account definitely for the difference in the result obtained in the crushing-tests of the cubes. It was well known that these differences did occur when a number of cubes were tested which had the same composition but which had been made at different times. Unless

Mr. Popplewell.

the conditions were made uniform in every respect, uniformity of results could not be guaranteed. As an example of this the following figures might be quoted as showing how specimens might vary in strength, although of the same composition and age, when the length of time between the mixing and the moulding was varied. These tests had been carried out by Mr. Popplewell for Mr. Corbett, of Manchester[1]:—

Age of Test-Specimen.	Time between Mixing and Moulding.			
	Fresh.	2 Hours.	4 Hours.	6 Hours.
2 months	192	174	150	126
4 months	234	183	172	167

Mr. Marsh's difficulty with regard to the gauge-length was apparently due to the fact that the actual measurements taken with the Martens extensometer were on 20 centimetres or 7·98 inches, and for convenience of reference and calculation the compressions given on the diagrams were their equivalents on lengths of 12 inches. Mr. Etchells had expressed surprise at the difference between the limit-of-proportionality stress and the yield-point stress of the steel used, but in the light of his experience of many samples of mild steel tested Mr. Popplewell did not consider this result in any way remarkable. The yield-point mentioned on p. 111 as 7·5 to 7·96 tons referred to the total load, while the yield-point of 18 tons, given on a previous page when describing the properties of the steel, would be seen on examination to refer to the load in tons per square inch. He heartily endorsed Mr. Faber's opinion as to the form which should be given to the ends of the test-columns in reinforced concrete, and agreed that the part of the column where the frictional grip was such as to transmit only a part of full load to the steel—so throwing on the concrete more than its share of the load—should be enlarged in cross section so as to reduce the stress in the concrete to something nearer its normal value. He could not admit the validity of the contention put forward by Messrs. Vawdrey and Yeatman that the concrete and steel in the middle of a column such as that described in the Paper could not do otherwise than compress together as one material, or, in other words, that the steel could not fail to carry its legitimate stress as fixed by the ratio of the two moduli. If there were no frictional bond between

[1] Journal of the Royal Institute of British Architects, 3rd series, vol. xviii, p. 18.

Mr Popplewell. the steel and concrete, or if this bond were very weak, then evidently the pressure of the concrete on the ends of the rods would not be sufficient to develop the full amount of stress in the steel. Either the frictional grip of the concrete on the steel must be sufficient to carry the full load of the bars at their centre, or, if this friction was not sufficient, the ends of the bars must abut against an unyielding surface, compelling equal strain of the concrete and the steel. If these conditions did not obtain, then it was conceivable that, as the load came on the column and the concrete was compressed, it would be pushed past the ends of the rods, which would compress the concrete immediately in contact with them. From the present knowledge of the properties of reinforced concrete most people would expect the concrete and steel to behave as they were shown to do in the experiments; but to say that they could not possibly do anything else was surely going too far. Mr. Yeatman's suggestion that measurements might have been taken nearer the ends was a good one, and it might be well in future experiments of this kind to go a step farther even than this and take measurements at, say, three places on the bar. This would furnish information as to how the frictional grip affected the stress in the bars throughout their length. The stress would be expected to be greatest in the centre of the column, tapering off to something very much less at the ends. Several speakers, notably Mr. Faber and Mr. Marsh, had remarked on the lowness of the modulus of the concrete. He had determined the modulus in question by means of Martens mirror observations taken on a prism of plain concrete having the same composition as that used in the columns. This particular concrete was not taken from the same mixing as any one of the columns, but was made of the same materials and as nearly as possible under the same conditions. At the same time it was quite possible that there was some variation in the conditions under which it was made that tended to lower the value of its modulus. He repeated his determination on the same prism about a year later than the second experiment mentioned in the Paper, that was to say, when it was about 18 months old: he found that the material had hardened still more, and obtained a modulus of 1,900,000 lbs. per square inch. With the modulus found for the steel, this would give a value of nearly 16 for the ratio of the moduli, m, which was much nearer the 15 mentioned by Mr. Faber and was the modulus found by Mr. Marsh's calculations. The modulus for steel had a value which varied only within comparatively small limits for all steels, so that the variation in the ratio depended almost entirely on the modulus of the concrete.

This was known to vary, even for concretes which were supposed to be the same in composition and age, within very wide limits, and he thought it unfair to base all calculations on an assumed constant value of m. In the absence of direct experiment at the time, it was impossible for designers to avoid making some such assumption, and, as Mr. Wentworth-Sheilds pointed out, even a large difference in the value of m made relatively small differences in the loads which columns could safely bear. In his opinion the values generally assumed for m had a tendency to err in the direction of being too low. In conclusion, he thought that further experiments might be made on similar lines to those described, especially in the direction of taking linear measurements at different parts of the bars, comparing the effects of central loads, eccentric loads, and loads uniformly distributed on the ends, and noting the effect of loadings repeated many times. It would also be of benefit to those carrying out reinforced-concrete work if the methods of carrying out crushing-tests on concrete could be standardized to some extent. Mr. Popplewell.

Mr. Burr, in reply, remarked that, in view of some features of this highly interesting discussion, it seemed advisable to direct attention to the fact that his tests had not been intended to accomplish more than to secure information at one point in what might be called a particular problem in reinforced-concrete column-work. A number of circumstances rendered it advisable to use the relative sectional areas of concrete and steel found in the columns tested. It was realized that the percentage of steel section was high, although not as high as had been used in actual work where this type of column was employed for high buildings. Indeed, the percentage of reinforcement in some columns of the usual banded type described in the discussion was so nearly equal to that found in these columns that the difference was immaterial so far as the ratio between the steel and concrete sections was concerned. The percentages of concrete and steel in the sections employed were not in any sense indications of what he might consider most advisable for any particular case. He hoped eventually to extend this experimental investigation into a series in which the percentage of steel would range from the lowest values employed to perhaps higher than in these tests. Mr. Meik and some others had objected to the term "reinforced concrete" being applied to columns of the composite type; but it was a matter of little consequence what this type of column might be called. It was probably reasonable to consider any combination member or structure composed of two different materials a composite member, including what were universally known as reinforced-concrete beams. Abundant experience with Mr. Burr.

Mr. Burr. conflagrations in tall steel-frame buildings in New York City had shown the effectiveness of a shell of such material as concrete outside of metal columns as fire-resisting protection to the covered metal, although the impact of heavy fire-streams and the falling of debris would occasionally crack it and detach it in places. This partial destruction in a protracted fire would render it improper to consider such a shell as part of the supporting material of the column, although effective as the fire-protection for which it was designed. It was for this reason that the compressive carrying-capacity of the concrete within the exterior limits only of the steel framework of the column was recognized. There was a large area of continuous connection between the mass of concrete in the interior of the column and the exterior shell, sufficient to give the latter all the support required for protective purposes. This might be supplemented, if desired, by wire wound rather loosely about the steel reinforcement, which would act as a kind of wire lathing to hold the exterior concrete or mortar protecting shell in place during the destructive vicissitudes of a protracted fire. Mr. Vawdrey would probably not consider the round rods and spiral or other winding wire of the usual wire-wound reinforced column as capable of carrying any material amount of compression as a steel column in itself, and that was what was meant by the quotation which he made from the bottom of the first page of Mr. Burr's Paper. Such a cage of steelwork, acting independently of the concrete in which it was encased, would certainly be markedly ineffective as "a direct load-carrying element of a column." The point Mr. Burr desired to make was that for high buildings—perhaps five or six to thirty or forty stories in height—it was essential to use a column capable of sustaining the bending stresses developed in the use of long columns. It would probably be generally admitted that this could be satisfactorily accomplished only by the use of a form of steel reinforcement capable of meeting such requirements by its inherent qualities as an independent member. The presence of concrete would give enhanced resistance to direct compression, but certainly not to long-column bending, except possibly indirectly to a limited extent. In fact, Mr. Vawdrey essentially concurred in this view in his remarks. In this connection it was well to point out that the straight straps or lacing bars between embracing members of steel reinforcement of the type of column under consideration formed but one part, and frequently not the main part, of the lateral support given by the steel to the enclosed concrete. The corner steel angles or the steel channels, or other similar parts used in this type of column, were frequently and perhaps generally the main lateral supports of the concrete, continuously and materially embracing the latter. The point was raised in Mr. Faber's interesting observations that the mode of application of the load at the end of a reinforced-concrete column would affect the relative carrying-capacity of the steel and concrete; and that view was undoubtedly correct. It was Mr. Burr's opinion that the type of column covered by his tests possessed a marked advantage in being adapted to such detailing at floor-intersections and other points of application of loading as to convey more effectively the designed proportions of load to the steel and concrete than was the case with spirally-wound or other similarly reinforced members. Regarding the unit working-stress to be allowed on the concrete of reinforced-concrete columns, to which Mr. Marsh made a very apt and suitable reference, it must obviously depend upon the amount of lateral support given by the banding or encasing steel employed. If there was a small amount of banding, spiral or other, or if the load-carrying steel of the other type of column was very light, the working-stress should necessarily be smaller than would be justifiable with a much higher percentage of the reinforcing steel, either as banding or as an encasing column. Spiral or plain banding had been employed by Mr. Considère up to 12·9 per cent. of lateral and 1·2 per cent. of direct reinforcement, giving to the concrete an ultimate resistance as high as 25,600 lbs. per square inch; but that could scarcely be considered a practicable reinforced-concrete column for ordinary building purposes. Even sand might be so encased or held by a thin cylinder of steel, or even by close spiral winding, as to carry a relatively high load per square inch. He concurred with Mr. Marsh in the opinion that the concrete in reinforced-concrete columns ought to be so supported as to yield a higher working-stress, especially in large structures, than 500 to 700 lbs. per square inch. He believed that in some structures it should be possible to reach a working-load of 1,000 lbs. per square inch on the concrete of reinforced-concrete compression members. The observations of Mr. Etchells regarding the classification of reinforced-concrete columns appeared to be well grounded. As had already been stated it seemed to be of little consequence whether this particular type of column were called a composite column or a column of reinforced concrete, since, as already observed, any combination of two materials in one member made that member composite. He believed from his own experience of the use of this type of column in high-building construction that the care required to secure the satisfactory filling of the interior spaces was no more than was compatible with economy and rapidity of construction. A relatively wet concrete and thorough agitation would accomplish the desired end, the largest individuals of the

Mr. Burr.

Mr. Burr. aggregate being small enough to cause no difficulty in securing a dense concrete core. In commenting upon the failure of the two types of steel columns Mr. Etchells appeared to have fallen into an error in the computations relating to the steel-channel columns. He gave the ratio of the length of the steel channel between wrappings, divided by the least radius of gyration of the channel section, as 50. As a matter of fact, the 4-inch by $\frac{1}{4}$-inch wrappings were 16 inches apart between centres, or 14 inches between the centres of the fastenings which determined the length of column. As the least radius of gyration of the 3-inch channel was 0·41 inch, the ratio l/r was 34, and not 50; hence Mr. Etchells's observation regarding the failure of this column in detail was not sustained. Further, as stated in the Paper, these channel-steel columns failed by general flexure rather than by detail flexure of any part of the channels. Mr. Wentworth-Sheilds had touched upon an important point in raising the question of a proper ratio between the moduli of elasticity for steel and concrete. Undoubtedly there were many well-established experimental results which indicated that this ratio might be taken as low as 12 or even less, while, on the other hand, other reliable experimental results might justify taking the ratio as high as 20, even for 1 : 2 : 4 concrete with cement of excellent quality. Obviously the value of the ratio would depend upon the age of the concrete, other things being equal. It would appear reasonable, however, in view of the large number of determinations which had been made so far, and in view of the further fact that reinforced-concrete members or structures must frequently sustain considerable loading at an age not greater than 30 days, to conclude that the prevailing value of 15 was well considered and worthy of its wide use. Mr. Burr was pleased to notice that Mr. Fitzmaurice concurred in his view as to the classification of the two types of columns considered in his Paper. It was difficult to imagine any substantial reason for not calling both types reinforced concrete. The same materials were elements of both types, and there was not the slightest doubt as to the mode of combination or as to the significance of any reference to either. Mr. Fitzmaurice's familiarity with high-building construction in New York City had prompted him to remark very aptly upon the adaptability of the two types of columns. The space occupied by columns in the lower floors of buildings ten to thirty or forty stories high was a matter of the utmost importance where floor-space was so valuable as it was in New York City. This fact, together with the capacity of the so-called composite type of column to resist column flexure, gave it adaptability to buildings of any height; whereas the larger area of cross section required for the spiral-wound or other banded column of the usual type and its lack of capacity to resist the bending action arising in long columns practically ruled it out of use in high-building construction, especially for buildings intended to contain heavy running machinery such as printing-presses, which set up vibrations that were frequently severe. In answer to Mr. Read's question how much of the extra strength of concrete-filled columns was relied upon in the calculations for the columns in the high buildings in New York, it was customary to use the full working-load permitted on concrete. In one eleven-story building in New York City where the columns and other structural work had been designed and constructed under Mr. Burr's direction, a working-load of 750 lbs. per square inch had been adopted for the concrete part of the composite columns, that load being permitted by the regulations of the Bureau of Buildings of New York City. If no additional load-carrying capacity could be credited to the concrete part of the section, few or no American engineers would use the concrete, but would design the regular or standard steel columns without any filling whatever. Mr. Burr.

Correspondence.

Mr. W. M. Bergin considered that Mr. Ellis was to be congratulated on having carried out a work of such magnitude in reinforced concrete in China, but regretted that on some points a little more detail had not been given. Nothing was said as to how the piles were handled; as the largest must have weighed 7 or 8 tons it must have been no small matter to move them about and place them in position ready for driving. In the design of the wharf-deck there was what seemed to be a serious fault, namely, the absence of any provision for expansion and contraction. Apparently an attempt had been made to construct a reinforced-concrete monolithic slab measuring 1,160 feet by 174 feet and subject to a range of temperature of about 150° F. (Shanghai winter-night temperature to summer-sun temperature), with the inevitable result that cracks occurred at or near the points where expansion-joints for the concrete should have been made, and exposure to corrosion and ultimate destruction of the reinforcement at these places was only a matter of time. The design of the reinforcement—a succession of connected cantilevers—excluded the Mr. Bergin.