resistance was practically eliminated. This was of value where the bearing-value of the piles was the chief factor in supporting the load. In many cases, however, as in clays which extended to a considerable depth, the frictional support of the pile was the prime consideration. The Author would suggest in such a case that a cruciform steel pile should be driven to a suitable depth, and after an interval for resettlement, should be loaded as a test-pile. As the bearing-action would be practically eliminated, the frictional force might be accurately determined. In designing extensive piled foundations, economy in design depended on a careful estimation of the bearing-value and the friction value of the pile and the probability of the latter increasing or decreasing during the period in which it had to support its load. The Author.

In conclusion, he desired to thank all who had taken part in the discussion, and in particular those members who had supported his suggestion of a Committee on Design to deal with problems such as the design of piled foundations.

Correspondence.

Mr. R. D. Brown agreed with the Author that the present position of the design of piled structures was not altogether satisfactory. Some of the factors which had to be considered as affecting the problem to a greater or less degree were the following :— Mr. Brown.

I. The Pile :

(*a*) Shape—whether tapered or parallel; (*b*) Material—wood, concrete, or steel; (*c*) Length; (*d*) Sectional area; (*e*) Perimeter; (*f*) Weight; (*g*) Young's modulus; (*h*) Nature of shoe.

II. The Dolly (if used):

(*a*) Material; (*b*) Length; (*c*) Sectional area; (*d*) Young's modulus.

III. The Hammer :

(*a*) Simple drop hammer, with monkey release; (*b*) Simple drop hammer, with release clutch at winch; (*c*) Steam hammer, with steam uplift only; (*d*) Steam hammer, with steam downward impulse ; (*e*) Diameter of cylinder; (*f*) Pressure of steam; (*g*) Adequacy or otherwise of escape valves; (*h*) Time-interval between blows; (*k*) Drop; (*l*) Weight of moving parts; (*m*) Weight of fixed parts.

Mr. Brown.

IV. The set with final blows.

V. The rebound of the hammer.

VI. The rebound of the pile-head.

VII. The strata, and their recuperation after rest.

VIII. The grouping of the piles.

IX. The order of driving.

X. The nature of the working load, vibratory or otherwise, and the proportion of "dead" to "live."

No formula could possibly embrace all these variables. Using the same data throughout in the case of one pile, he had arrived at the following diverse results :—

Formula.	Safe Load: Tons.
Rankine [1]	40
New York Building Code	45
Trautwine	85
Saunders	43
Dutch Rule	18
Weisbach	90
Boston Rule. (Surface contact method)	35

The New York Building Code specified a factor of safety not less than 6, while Rankine recommended "from 3 to 10, depending upon the judgment of the engineer." A good deal must be left to the decision of the engineer. In some recent work with which Mr. Brown was associated, the "factor of safety" had been as low as $2\frac{1}{2}$, but in that case the decision had been confirmed by actual test-loading, which was, in the last resort, the only sure method of arriving at a reliable result. As a first step in simplification it was possible to cut out four of the variables he had given by discarding the dolly, at least for the last few blows. Bare-headed driving had been done successfully on some very large contracts for the driving of concrete piles carried out on the Manchester Ship Canal under Mr. H. A. Reed, M. Inst. C.E. To enable the pile to withstand this severe condition, it was necessary specially to reinforce its head, and to clamp it for about 2 feet with a split steel collar. The damage done, however, was of little moment, because a large portion of the pile-head had to be stripped in any case for bonding with the upper work. Of the formulas he had mentioned, Rankine's was the only one based mathematically upon the elastic reaction of the pile and strata to the energy given out by the last blow. Unfortunately, it was too unwieldy and cumbersome

[1] "Manual of Civil Engineering." London, 1863.

for use in general practice. That no doubt could be remedied by the preparation of a series of Tables to cover different conditions. He strongly supported the Author's suggestion that a committee of The Institution should investigate this matter by obtaining standardized data from members engaged on pile-driving. It was desirable that precise measurements should be made (by, say, photo-electrical methods) of the actual behaviour of piles under the final blows of the hammer, and these should be compared with actual test loads that would cause settlement. Until some further progress had been made in this matter, pile-driving formulas must be regarded as merely approximate aids. Great care must be exercised in adopting the loads to be carried, and due regard should be given to the fact that fortunately in the great majority of cases piles driven further after an interval of some months would be found to have acquired much better sets. Mr. Brown.

Mr. William Cleaver complimented the Author on the concise arrangement and practical value of the information he gave, particularly with regard to the piling records. Although the adoption of the monolith foundation in the north-east abutment of the Gade bridge and the concrete piles in the Colne bridge might be accounted for by local conditions, etc., it would be of interest if the Author would further explain matters. It was not quite clear why a monolith foundation had been adopted for one abutment only, particularly in view of the fact that pile-driving plant, etc., was already in use and available. Not only was pile-driving generally more satisfactory than monolith-sinking, but in the case under review he imagined that it would have been considerably cheaper and equally effective, unless there were local conditions, not indicated in the Paper, necessitating the exclusion of piles on that particular portion of the site. There did not appear to be any specific reason for using concrete piles at the Colne bridge when timber piles had been decided on for the Gade bridge, or vice versa; perhaps the Author would throw some further light on this. He could understand the adoption of timber piles in one case and concrete piles in the other on two absolutely separate schemes, each individual engineer being possibly prejudiced in favour of one or other material; but in the case under review prejudice for one material or the other would not apply. Mr. Cleaver.

Mr. W. G. Miles observed that the information given in the Paper was extremely useful, especially in conjunction with that contained in the Paper[1] by Mr. H. A. Reed, M. Inst. C.E., on the Mr. Miles.

[1] Minutes of Proceedings Inst. C.E., vol. 221 (1926), p. 67.

Mr. Miles. reconstruction of Trafford Wharf, Manchester. The Author stated that investigation of the site of bridge No. 2 (Colne bridge) showed that no good bearing strata for the abutment-walls were reached. What methods had been used in those investigations? From the Paper it would appear that, instead of the safe load on piles being calculated as was usually done, the resistance of trial piles should alone determine the amount of driving for the supporting-power of the other piles; this would take into account all the pile dimensions. If M denoted the weight of the monkey in tons, H the drop of the monkey in feet, and S the penetration of the pile per blow in feet, then the energy expended in driving the pile through any measured distance into the ground was $\Sigma M(H+S)$ foot-tons. Let F denote the frictional resistance in tons per square foot, P the perimeter of the pile in feet, l the length of the pile below ground-level in feet, N the number of blows given to the pile during the last stages of the driving, in which period the penetrations were measured, and K a constant depending on the amount of cushioning, then $M(NH + \Sigma S) = KFPl$, and it only remained to find K before the actual resistance and ultimate bearing-capacity of the ground was determined. To find the value of K, No. 1 trial pile should be driven without a dolly, and No. 2 about 10 feet away with a new elm dolly, say 2 feet long; then, provided the two piles were driven into the same strata, it would be easy to find K. As the dolly became broken, the engineer might reasonably consider the reduced cushioning-effect and vary K accordingly; for example, if it appeared that the dolly had only 75 per cent. of its original efficiency, then the original K should be reduced accordingly. This method dealt only with the resistance of the ground at the time of driving, and to allow for the gain of supporting-power after 12 to 18 months, a further factor K_1 should be included in the calculation for the safe load on the pile. The formula then took the form L (safe load in tons) $= FPlK_1$. Now K_1 depended on F. As the pile was driven farther into a stratum, F increased, and, according to Rankine, this depended on $\{(1-\sin\phi)/(1+\sin\phi)\}^2$, hence the graph would always be a curve—as shown in *Figs. 5–9* (pp. 249–251). On most works it was impossible to test piles 12 months after driving, but if a committee were formed, as the Author suggested, it should be possible to determine the value of K_1 for a number of cases.

Mr. Mitchell. Mr. James Mitchell remarked that the Author stated that formulas depending on observation of the set of a pile could only be justified so long as the pile was wholly supported by friction, but he gave no reason for the statement. Every pile-driving formula depended on the set of the pile, but in general the nature and distribution of the supporting-forces were largely matters of conjecture. It might be said that during a portion of the driving of a particular pile the resistance was due to the friction against stiff clay, and that during a further portion it was due to the shearing of hard chalk, but the difference was one of degree rather than one of kind. None of the formulas dealt in any way with the question of the lateral support of the pile, but solely with its sufficiency to resist an axial thrust. For this, among other reasons, the Author's contention that formulas should be expressed in terms of ultimate resistance, the engineer being required to determine a suitable factor of safety, was sound. It was stated on p. 258 that "the variations met with in the number of blows required per foot of penetration over a fair range are sometimes so considerable that they call for the average set to be read from an 'averaged' graph." It could not be too clearly kept in view, however, that the only thing that concerned the engineer was the final set, and that the only reason for continuing for five or twenty blows of the hammer was to make quite sure that the desired set had indeed been reached, and was not merely temporary, owing perhaps to some small obstruction. On p. 257 reference was made to the fact that it had "been noted by many authorities" that the time-interval between blows was a vital factor in determining the set of a pile for a given expenditure of energy. There was, however, a marked dearth of evidence in support of this statement, except where the time-interval was expressed in hours or days. Although a pile that had been allowed to rest for half an hour might be found to have "stiffened up" considerably when driving was resumed, it was very doubtful whether the difference of time-interval between the blows of say a drop-hammer and those of a single-acting steam-hammer had any appreciable effect on the driving of the pile. Assuming eight strokes per minute in the former case, and thirty-five strokes in the latter, the difference between the periods of rest in the two cases was only about $5\frac{3}{4}$ seconds. Comparing the single-acting steam-hammer with a double-acting one striking 150 blows per minute, the difference of time-interval was about $1\frac{1}{3}$ second. While it would be rash to assume that even the latter small difference had no effect on the amount of energy required to drive a pile, it was far from clear that the advantages claimed for the double-acting over the single-acting steam-hammer, or the drop-hammer, were materially due to this factor. It was stated (p. 257), with reference to the Wellington formula, that a variation of the constant was used "to allow for the quicker blows of a steam-hammer." With regard to the

Mr. Mitchell.

Mr. Mitchell. single-acting steam-hammer, however, the effect of the back pressure of the exhaust steam probably more than counterbalanced any advantage due to the greater rapidity of the blows, as compared with a drop hammer. In the case of the double-acting steam-hammer, the efficiency was greatly affected by variations in the boiler-pressure, by heat-losses due to condensation of the steam in long lengths of steam-pipe, and by variations in the air-temperature and in the velocity of the wind. Referring to the question of the ratio between the weight of the pile and that of the hammer, the elastic compression of both the pile and the hammer had to be taken into account. A certain quantity of work might be applied to the head of a pile by means of a projectile from a gun, without moving the pile as a whole, while the same quantity of work applied by means of a hammer of ample weight would drive it freely, the elastic reaction in the former case acting chiefly upwards, and in the latter case chiefly downwards. The difference between the two cases was analogous to that between forging done by means of a steam-hammer and that done by means of a forging-press. The same reasoning applied to the elastic compression of the ground and its reaction on the pile and the hammer. With reference to the statement that the weight of the hammer should never be less than that of the pile, a pitch-pine pile 50 feet by 14 inches by 14 inches would weigh about 24 cwt., but for anything like stiff driving a 24-cwt. hammer was quite inadequate for such a pile. The provision of heavier hammers involved slightly greater first cost, and also stronger piling-frames and staging, while they necessitated more labour in moving them to and from their work. Those drawbacks were, however, more than counterbalanced by the greater ease and certainty of driving, and the reduced risk of splitting or crushing the head of the pile.

Mr. Monk. Mr. J. E. MONK remarked that on the Gade bridge square girders had been used on skew spans. That appeared to have led to increased pier and abutment dimensions. He had known bad running at the ends of open-decked skew girders, arising out of the end sleepers being supported at one end on a girder and at the other on ballast; but in this case, where the girders were decked and the ballast was continuous, it was difficult to account for the girders being kept square. The flood spill openings had been built square to the line, and this had entailed heavy expenditure on extra masonry and piled foundations in pier No. 2, which would have sufficed to build these two spans on the skew and in line with the current. It would be of interest to know if the possibilities of reinforced-concrete rafts for these foundations had been investigated.

Mr. Monk.

Pier No. 3 rested on practically the same depth of ballast as the south-west abutment and appeared to be a suitable case for the adoption of a reinforced-concrete slab to reduce the pressure, instead of relying on the somewhat doubtful distribution of the total weight between the piles and the bearing on the ballast of the concrete cap. He had used reinforced concrete for turntable pedestal foundations with satisfactory results in Bengal, where the intensity of pressure on soft alluvial clay had to be reduced to 1 ton per square foot. The adoption of monoliths 30 feet by 22 feet in section, as against smaller units, and the design of the monoliths, called for remark. The use of a big monolith implied unnecessary sinking and expenditure on unrequired masonry and concrete if, after the major portion had reached satisfactory material, a soft pocket was left affecting only a part of its area. The sinking of a large monolith on a timber or even a reinforced-concrete curb would have to be carefully done if the cracking of the masonry described were to be avoided. If, owing to influx of water or a bad blow from the river, the Author had been compelled to adopt sinking by grabs, it appeared probable that the whole of the masonry steining would have collapsed. It was difficult to understand why a timber curb had been adopted for a structure in which the stresses must be heavy and vary greatly. The cutting edge was only screwed on to the timber curb, and though that might suffice where it could be carefully and evenly undercut by hand, the attachment would quite possibly have sheared if the cutting edge had been called on to make its own way through any hard material. A better design would be a steel plate with an angle riveted inside, or angle cleats closely spaced, and bearing against the under side of the timber curb. The most serious omission, however, in the design of the monolith appeared to be that of any provision for tying the curb and masonry together vertically. An integral part of the design of a monolith, whatever its shape, should be vertical rods, 1 to 1¼ inch in diameter and 4 or 5 feet apart between centres, passing up through the curb and masonry and connected at intervals of 6 to 8 feet by 4-inch by ½-inch horizontal flat bars. The vertical rods should be in lengths of 6 to 8 feet, with screwed ends, and the horizontal bars placed so that the long coupling nuts of the vertical rods bore on them. A monolith or, as it was termed in India, "well," held together in that manner had never in his experience shown signs of failure or parting from strains due to big and irregular variations in skin-friction during sinking, or from the curb being unevenly supported. From the thickness of outer wall adopted it might be presumed that it had been intended to strut it from the commencement.

Mr. Monk. Why had not a greater thickness of suitably reinforced brickwork been adopted, with the consequent elimination of a large quantity of timbering and all the advantage from the extra weight of the thicker masonry and reduction in handling kentledge? Engineers in England might, he thought, learn much from those in India about the design and handling of foundations of that type. In India the foundation would have consisted of two rows of circular "wells," 10 to 12 feet outside diameter, with working-shafts 5 to 6 feet in diameter, on steel, reinforced-concrete, or timber curbs, built three-ply and with a steel cutting edge. They would be fitted with tie-rods and bond-rings as already described, and the masonry would consist of well-burnt second-class bricks in lime mortar, with the exception perhaps of the corbelled courses above the curb, which might be built in cement. If dry, they would have been plugged with lime concrete up to the top of the corbelling, the shaft filled with sand or sand and stone metal well rammed, and another plug of 3 feet of lime concrete filled in at the top. If there were no danger of scour, some engineers might replace the bottom plug of concrete by sand filling. It was, he thought, to be regretted that one effect of the piling in piers Nos. 1 and 2 had been to break up a compact bed of gravel which appeared to be 8 to 12 feet thick, especially when a depth of 8 feet of this same gravel had been considered good enough to carry the south-west abutment without any help from piling. The bonding of the abutment and wing walls together, resting as they did on very different foundations and with different loading, was, he considered, a mistake. A "butt" joint in such cases was preferable.

In the Colne river bridge the appearance of the fascia would have been greatly improved if a springing-line normal to the intrados could have been indicated.

The Author. The AUTHOR, in reply, remarked that he was pleased to note the interest taken in the Paper. He had, during the driving of the first trial piles for the Gade Bridge, attempted, as Mr. Brown had done, to calculate the bearing-value of the piles from various standard formulas, with a similar diversity of results. He noted Mr. Brown's agreement with him that in the majority of cases piles redriven after some months would be found to have acquired much better sets. That, however, must not be allowed to mask the fact that in certain cases it was possible that the bearing-value under working-conditions might in time be appreciably reduced, owing to frequently recurring dynamic loading on a structure carrying a small dead load. The careful comparison of test-pile driving-charts with borings in the same vicinity would usually be of value in enabling the engineer to determine the factor of safety to be used in a given case. With reference to Mr. Cleaver's remarks on the use of timber piles at the Gade bridge and concrete piles at the Colne bridge, these, as mentioned before, were entirely separate schemes. Timber piles had been used at the Gade bridge primarily to expedite the work, and, being completely below ground-water level, their long life was considered to be assured. For the Colne bridge the trial holes had been carried down 5 or 6 feet, and had revealed conditions similar to those encountered in the open excavations at the Gade bridge, including traces of disintegrated chalk. That had been confirmed by the bore-holes, by the very similar geological structure of the Colne and Gade valleys, and by the trial piles, before the casting of concrete piles was ordered. The formula given by Mr. Miles was of considerable interest, but it appeared to depend on the value of F (the frictional resistance). The Author had attempted by calculations based on Rankine's formula to evaluate the frictional resistance and the bearing resistance from the driving results, but he had found that that was of little value except during the driving through gravel, which might be considered as, in theory, a granular mass composed of more or less spherical particles. When the pile had penetrated the gravel and was being driven through the disintegrated chalk, the frictional resistance appeared to decrease rapidly, owing probably to lubrication of the sides and loosening of the gravel. The calculation was, in addition, complicated by the varying bearing-resistance. Mr. Mitchell had questioned the Author's statement that "set" formulas should only be used for friction-supported piles. In the extreme case of a pile penetrating soft clay or mud and then being driven on to a moderately hard rock, any desired set might be obtained by hitting the pile hard enough or often enough, but it would be due to the crippling of the pile. In effect the pile would be a column resting on a fixed base and receiving a heavy dynamical load. In such a case, the set would obviously be of no value in determining the supporting-power. It was for the engineer to decide from the borings and the piling-charts whether he should calculate the bearing-value of the piles as laterally restrained columns or as friction-supported piles liable to settle. The importance of the "time interval between blows" was due to the action which took place in driving. The blow of the hammer caused a wave of energy to travel down the pile at a rate depending on the nature of the blow, the material of the pile, and the nature of the resistance. The resistance encountered and the elasticity of the pile material The Author.

The Author. reversed the direction of this wave of energy after a very short but definite interval of time. If the force of the reversed wave was sufficiently great it would cause the hammer to bounce. The energy wave fluctuated along the pile and quickly died out. The resistance of the ground was in the nature of a frictional force, and, as Mr. Wellington had recognized in evolving his formula, the static friction overcome, if the pile was at rest when struck by the blow, would be considerably greater than the dynamic friction while the pile was moving. When the driving was done with a drop hammer the pile would undoubtedly come to rest between the blows, and in consequence, some degree of static friction must first be overcome in each case. The amount of this increased static resistance would depend on the time-interval. In the case of the single-acting steam-hammer, the static resistance would be very considerably reduced. When the driving was by means of a double-acting steam-hammer the resistance was practically wholly dynamic in character. The pile was kept on the move, and the effect of its inertia was also reduced to a minimum. There was therefore an essential difference in the necessary energy to secure a given penetration in the three cases, the actual quantities depending on the factors mentioned. If reinforced-concrete rafts had been used for the Gade bridge, as suggested by Mr. Monk, they would have had to be of such an area that their spreading effect plus the spreading effect of the thin layer of gravel left beneath them would together reduce the total pressure on the disintegrated chalk from 3 tons per square foot to not more than $\frac{1}{2}$ ton per square foot, which would have involved very wide inverted cantilevers of impracticable dimensions in the case of the piers Nos. 1 and 2, where the gravel was only about 5 feet thick. At the south-west abutment the gravel was more than 12 feet thick, its extent being 3 to 15 feet below ground. The depth of undisturbed gravel below the foundation was therefore about 8 feet, which had been deemed sufficient, in that case only, to permit the use of a raft foundation. In conclusion, he desired to thank the Chief Engineer, Metropolitan Railway, and the County Engineer, Hertfordshire County Council, for kindly sanctioning the presentation of the Paper.