

they were of the opinion that the process had to continue to be run so as to produce a certain proportion of calcium sulphite in the system, purely from a point of view of economics.

On the question of inhibitors being used to prevent corrosion, whilst that had been proving successful to a certain extent up to date, they were of the opinion that the final operation of the plant should be such that inhibitors should be unnecessary, and the elimination of brass interlacers was a step in that direction.

Messrs. Pearson, Nonhebel, and Ulander appeared to feel that Messrs. Parker and Clarke were putting some slight condemnation on the plant due to operating difficulties which had been experienced. Messrs. Parker and Clarke would like to hasten to contradict any such impression, and to say quite clearly that the plant was new, involving numerous novel features, and that it was only to be expected that difficulties would occur. However, as stated in the Paper the efficiency of the plant in removing sulphur oxides from the flue-gases could not be disputed, as it was working with an efficiency of approximately 99 per cent. The main difficulties being experienced, which would be overcome, were high maintenance-costs and the limited running hours that the plant would remain in operation.

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Paper No. 5154.

**“The Reconstruction of Main Road Bridges, Calcutta.” †**

By MALCOLM RAMSAY ATKINS, C.B.E., B.Sc.(Eng.), and DOUGLAS  
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**Mr. W. J. Doak**, of Brisbane, observed that the Authors had shown remarkable courage in the adoption of arched bridges on yielding foundations. The elaborate precautions taken to prevent horizontal movements suggested that costs had probably been high, but, apart from that, settlements of 5 inches were bound to have produced some undesirable results in the structures.

Regarding the bearing power of piles, Mr. Doak had frequently endeavoured to reconcile the bearing power, computed more or less in the

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† Journal Inst. C.E., vol. 9 (1937-38), p. 95 (June 1938).

manner given on pp. 106-107 §, with actual test-loads. The remarkable feature was that the latter had always been much the greater. That was still more remarkable when it was recalled that the frictional resistances to concrete caissons measured at the Zambezi bridge were less than Rankine's theory would indicate.

The figure of 110 lb. per cubic foot for  $w$ , the weight of soil, was very much suspect if the piles were in wet ground; the weight of soil would then be reduced by buoyancy to, say, 50 lb., which halved the figures both for bearing and frictional resistance. The source of resistance of driven piles was still an unsolved mystery.

In regard to the bowstring designs (which were actually tied arches), the method employed for computing moments in the arch-ribs would be of considerable interest.

**Mr. J. P. Porter** desired to confine his remarks to those parts of the Paper which referred to the supporting value of piled foundations, and to the calculations relating to the foundations of Beliaghatta bridge. He noted that the formulas for the supporting values of piles given on pp. 106 and 107 § were identical in form with those given in his own article on "The Supporting Value of Piled and other Deep Foundations\*." Beliaghatta bridge had been constructed before the publication of that article, but the Paper had not been published until 1938. He would be interested to learn whether the Authors had evolved those formulas independently or whether they had been derived from his article.

It appeared to be desirable to state the derivation of his formula, which was intended to give the ultimate supporting value of any prismatic deep foundation, whether driven as a pile, sunk as a cylinder or monolith, built up in an excavation as a pier, or formed as a pile bored in situ. The general formula was

$$U + P = \sqrt{l}A_b b + fp \frac{l^2}{2}$$

where  $U$  denoted the ultimate supporting value of the foundation-unit,

$P$  " " weight of the foundation-unit,

$l$  " " buried length of the unit,

$A_b$  " " effective bearing area (= 4 times the sectional area, in the case of a driven pile),

$b$  " " bearing-factor,

$f$  " " friction-factor,

$p$  " " perimeter of the foundation-member.

The value of  $b$ , the bearing-factor, was assumed to be  $w \left( \frac{1 + \sin \beta}{1 - \sin \beta} \right)^2$ ,

§ Page numbers so marked refer to the Paper. (Footnote (†) above.)—SEC. INST. C.E.

\* *Concrete and Constructional Engineering*, vol. 31 (1936), p. 319.

where  $\beta$  denoted the apparent angle of internal friction of the subsoil at the foundation-plane and  $w$  the density of the subsoil. The values of  $b$  suggested in his article had been derived from a detailed study of all data within his knowledge relating to the settlement of deep foundations and the test loading of piles.

The value of  $f$ , the friction-factor, was taken as  $w \tan \phi \left( \frac{1 - \sin \phi}{1 + \sin \phi} \right)$  for values of  $\phi$  not greater than 30 degrees,  $\phi$  being the average angle of internal friction of the subsoil in contact with the sides of the foundation-member. Where the apparent value of  $\phi$  exceeded 30 degrees, he had suggested provisional values of  $f$  ranging from 0.015 ton per square foot at 35 degrees to 0.025 ton per square foot at 45 degrees in the case of driven piles, and of 0.010 ton per square foot in the case of other deep foundations. The values of  $\phi$  and  $f$  given in his article were based on a detailed consideration of a large number of skin-friction coefficients derived from the sinking of cylinders, caissons and monoliths and from the withdrawing of piles, as given in standard text-books and in the Proceedings of The Institution.

The values of the bearing-factor  $b$  and the friction-factor  $f$  which he suggested for use in his formula were as followed :—

Subsoil.	Bearing-factor $b$ : tons per square foot.	Friction-factor $f$ : tons per square foot.
Compact gravel . . . . .	2.00 to 2.20	0.025 to 0.030
Gravel and sand, or very stiff clay . . . . .	1.60 to 1.80	0.022
Firm sand, hard sand-clay or water-bearing gravel . . . . .	1.20 to 1.40	0.020
Firm dry clay, chalk or moist sand . . . . .	0.80 to 1.00	0.015 to 0.020
Soft clay or sand-clay, or wet sand . . . . .	0.05 to 0.60	0.010 to 0.012
Peaty clay ("bungum") . . . . .	0.30	0.009
Soft wet clay, wet silty sand, peat or soft wet chalk . . . . .	0.20	0.008
Mud or silt . . . . .	0.15	0.007
Thin mud or silt . . . . .	0.10	0.005

(NOTE.—Values of  $f$  for all foundations other than driven piles should be taken as not greater than 0.010 ton per square foot.)

The use of the term  $\sqrt{l}$  in the formula for bearing resistance was based on practical observations of the test-loading of "Vibro" tubes as given by Mr. Alfred Hiley, M. Inst. C.E. The formula had been checked against all available data regarding the test-loading of driven piles, and in the majority of cases had been proved to be correct to within 20 per cent. The piles had been driven in strata of every type, and ranged from 8-inch timber piles carrying 10 tons ultimate load to 24-inch reinforced-concrete piles carrying 300 tons ultimate load.

It was to be noted that the factor of 4 applied to the bearing area of driven piles was equally applicable to piles cast in situ, provided that the

hole for the pile was formed by the dynamic driving of a tube or mandrel, but did not apply in the case of bored piles, where the subsoil was not pre-compressed.

With reference to Dr. Faber's remarks (p. 126 §), it should be noted that the formula allowed for a bearing resistance four times as great in the case of driven piles as in other cases. Thus the bearing resistance of a pile driven 40 feet and penetrating into compact ballast would be about 50 tons per square foot of pile section, whereas the bearing resistance of a non-driven member would be about 12 tons per square foot. Those figures were comparable with those cited by Dr. Faber.

Mr. Porter had recently had an opportunity, in the construction of piled foundations for large reinforced-concrete tanks, of testing the formula in practice against the ultimate-resistance values calculated from Mr. Hiley's formula. The results obtained had been such that he had been able to predict the ultimate resistance of piles of varying sizes and lengths driven into ballast and clay before the piles had been driven. By using those predicted values in Mr. Hiley's formula he had been able to estimate the probable driving sets for the piles at various levels. The sets so predicted had agreed very closely with the observed sets of the piles when driven. As the work was still in progress he was unable to give further particulars at present, but he hoped to have an opportunity of publishing them in due course.

It might be argued, in view of recent investigations into the theory of soil-mechanics, that it was irrational to use Rankine's formula for determining bearing- and friction-factors in cohesive materials. In his investigation, however, he had found that for all subsoils except rocks and semi-fluids it was possible to select values of  $\beta$  and  $\phi$  which agreed reasonably with the results obtained from recorded data regarding foundations. There were indications, however, that the values used should be increased somewhat in the case of relatively deep foundations.

Mr. Remfry had apparently considered it advisable to check the supporting value of one complete foundation, firstly, as composed of a number of piles acting independently, and, secondly, as a pile-group in which the subsoil engaged by the piles was assumed to settle with them. Mr. Porter was doubtful whether the second assumption was justified in the instance under consideration, since (a) the average spacing of the piles in the main part of the foundations exceeded 5 feet between centres, (b) the silt had not been compacted more than about 4 per cent. by volume due to the pile-driving, and could hardly therefore be considered as being fully engaged by the piles, and (c) owing to the design of the raft, the surcharge of about 20 feet of soil at the rear, and the use of sheet-piling at the front, there was bound to be a considerable uplift under the raft.

The following remarks regarding the subsoil were to be noted. "...

the subsoil proved to be poor. . . . The soil below the raft appears to be fairly fine silt which extends for a considerable depth ; its angle of internal friction appears to be about 19 degrees . . . the level of the subsoil water is kept constant by the near proximity of the canal. . . . Apparently the mean pressure over the base area under the original superimposed earth loads was about 2.55 tons per square foot. The site must have been well consolidated. . . ." In view of these remarks, he preferred to use a value for  $\beta$  of 25 degrees, giving a value for  $b$  of 0.30 ton per square foot, and an average value for  $\phi$  of about 15 degrees, giving a value for  $f$  of 0.007 ton per square foot.

He would first consider the foundation as being so supported by a number of independent piles that there was no actual pressure on the subsoil under the raft, and also that the uplift-effect could be neglected. Each pile would be assumed capable of carrying the full load calculated in accordance with his formula. Taking the assumptions given on p. 107 §, excepting as regards values of  $\beta$  and  $\phi$  :—

$$\text{Average bearing resistance per pile} = \sqrt{52} \times 5.46 \times 0.30 \text{ tons} = 12 \text{ tons.}$$

$$\begin{aligned} \text{Average frictional resistance per pile} &= 0.007 \times 4.66 \times \frac{52^2 - 12^2}{2} \text{ tons} \\ &= 42 \text{ tons.} \end{aligned}$$

Therefore total supporting value of one hundred and thirty-seven piles  
 $= 137 \times 54 \text{ tons} = 7,400 \text{ tons.}$

$$\text{Therefore factor of safety on above assumptions} = \frac{7,400}{5,720} = 1.3.$$

The alternative method of treating the foundation as a single unit, in his view, was to assume that the average fluid pressure under the raft was that due to 20 feet of earth resting on saturated silt, that (in the worst case) the silt around the sides of the piles was, for the greater part of their length, fluid in nature and therefore virtually incapable of providing any support, and that the total bearing and frictional value of each pile in the apparently more compact material near the toe did not exceed 15 tons (the bearing value being 12 tons). The fluid uplift-pressure (due to 20 feet head of earth) under the raft was approximately 4,375 square feet  $\times$  1 ton per square foot = 4,375 tons. The probable minimum supporting value of one hundred and thirty-seven piles was about  $15 \times 137 \text{ tons} = 2,055 \text{ tons.}$

Therefore, on the above assumptions, the factor of safety was  $\frac{6,430}{5,720} = 1.1$ . It appeared, therefore, that the factor of safety slightly exceeded unity, whichever method of treatment was considered most applicable, but that it would have been advisable to have used longer piles.

In accordance with the second treatment suggested, the majority of the load was transmitted to the subsoil at raft-level and exerted a downward pressure no greater than that due to the adjoining embankment. The load carried by the piles produced an additional pressure at toe-level of less than 0.4 ton per square foot. Mr. Remfry's contention that the foundation-plane should be capable of supporting a slightly increased pressure appeared to be justified from practical considerations, and Mr. Porter agreed that the method of treating pile-groups given in his own article was not, in that instance, applicable. It was, however, to be inferred from the care with which Mr. Remfry had examined the foundations in question that he entertained some doubts as to their suitability as foundations for a reinforced-concrete arch bridge. Those doubts were, in view of his calculations, fully shared by Mr. Porter.

**Mr. Atkins**, in reply, wished to assure Mr. Doak that no undesirable results had been produced on the structure of the Dum Dum bridge by the settlement of 5 inches which had taken place. The clearance at the hinged joints had in fact allowed for considerably more movement than had been observed. The lowering of the level of the roadway by 5 inches was of little importance, as the programme of operations allowed a period of 2 or 3 years to elapse between the completion of the bridge and the construction of the permanent approaches. With regard to Mr. Porter's remarks he confessed that some doubt had been entertained by both Mr. Remfry and himself regarding the suitability of the subsoil for resisting the thrust of a reinforced-concrete arch. He submitted, however, that the precautions taken had proved completely successful, and that the adoption of the design had been justified in practice, if not in theory. The approximate cost of the temporary bridges was: road-bridges, £2,000—£2,500 each; foot-bridges, £800 (Alipore) and £350 (Chitpore).

**Mr. Remfry**, in reply, observed that the maximum total settlement of the crown of the Dum Dum bridge was  $2\frac{1}{2}$  inches, which might be the settlement due to the shrinkage of the concrete, plus 2 inches. In construction the centering for the ribs had been given a small excess rise which would take care of part of that settlement. The bowstring arches had been calculated as two hinged arches, the constraint due to the connexion of the tie beams having been disregarded.

In presenting the Paper the pile formula used was the one suggested by Mr. Porter, as Mr. Remfry considered Mr. Porter's views to be very well thought out and clearly presented. They did not, however, satisfactorily explain the supporting power of a pile-group under the particular conditions of the site. In the original design work it had been assumed that a raft on Calcutta soil would safely carry  $\frac{1}{2}$  ton per square foot loading above the load carried at that level normally, provided that the raft was kept fairly high within the upper hard crust of soil. As a raft to carry the full load at that assumed loading would have been too large, piles were relied upon to supply the extra supporting power. No satisfactory

theory seemed to exist to determine the carrying power of a raft plus a pile-group.

The normal depth of soil in the bank above the bottom of the raft was about 14 feet, producing a pressure of 0.70 ton per square foot. At that depth the bottom of the raft should carry 1.20 ton per square foot, or a little more than the 1.0 ton assumed by Mr. Porter if an earth surcharge of 20 feet were assumed at the back of the foundation. As a matter of fact the earth surcharge of the roadway behind varied from about 18 feet at the back of the abutment to about 28 feet near the front edge, or an average of 23 feet, which might have given an average uplift pressure of 1.15 ton per square foot. In some cases there was no sheet-piling along the front edge of the foundations; that was used only at Dum Dum bridge and at Beliaghata bridge, so that it might not be safe to assume the full uplift. Assuming an uplift of 1.15 ton per square foot at Beliaghata bridge, the support would be  $4,410 \times 1.15 = 5,072$  tons, leaving the 137 piles to support about  $4\frac{3}{4}$  tons each, or 648 tons, giving a total of 5,720 tons. There was no doubt that the piles used could offer very much more support than that.

In regard to the carrying power of the piles at Beliaghata bridge, the transition between individual action and group-action was apparently bound to depend upon the relative bearing support and the frictional support. If the piles acted individually there was a concentrated bearing support around the point of each pile, but the soil between those bearing points had to be capable of carrying the frictional support transferred from the sides of the piles. In fact, the soil between the piles might be considered as columns transferring the frictional support to the general area of the sub-grade adjacent to the points of the piles; there had to be sufficient area for that to be the case. It was scarcely reasonable to suppose that the intensity of pressure carried on the sub-grade between the bearing areas would be quite as great as that on the bearing areas; it would be absurd to expect that it would be greater. The following assumption regarding the relative supporting power of an average pile was therefore made: support from bearing area of 5.46 square feet, at a pressure of 1.37 ton per square foot = 7.5 tons, frictional support = 51.4 tons, total 58.9 tons per pile; hence from 137 piles, the total supporting power was 8,110 tons.

If that assumption were correct, then for the whole area, at pile-point level:—bearing of 137 piles with 745 square feet of bearing area, at a pressure of 1.37 ton per square foot = 1,030 tons; frictional support spread over 5,850—745 = 5,105 square feet, at a pressure of 1.39 ton per square foot = 7,080 tons; total = 8,110 tons. That meant that between the bearing areas at the pile-points the excess pressure transmitted to the sub-grade was about the same as at such a bearing, the average spacing of the piles being 6.5 feet.

The conditions would be worse if a strip 9 feet were taken, where the pressure from the arch ribs was greater and where the piles were closer. Under such conditions the pile-spacing was 5.7 feet and the supporting area per pile at the pile-points was

32.3 square feet; hence 5.5 square feet of bearing at a pressure of 1.37 ton per square foot=7.5 tons, and 26.8 square feet of frictional support, at a pressure of 1.92 ton per square foot=51.4 tons. The frictional support could not exceed the bearing support, and undoubtedly a group-action would develop if the assumption made were correct.

From Mr. Porter's calculation of the supporting power of an average pile (namely, bearing support=12 tons and frictional support=42 tons (total 54 tons)), 137 piles carried 7,400 tons. If the whole side were considered, then 137 piles, with 745 square feet of bearing at a pressure of 2.2 tons per square foot =1,644 tons; 137 piles, with 5,105 square feet of frictional support, at a pressure of 1.12 ton per square foot=5,756 tons; total=7,400 tons.

Mr. Remfry considered that, if Mr. Porter's figures regarding the relative supports offered by bearing and friction were the more correct (as he thought that they might be), then the piles would act individually. If, however, the relative proportions as estimated in the Paper were correct, the piles would act as a group. He would suggest that when the frictional support per square foot between the bearing areas at the points of the piles exceeded 75 per cent. of the bearing pressure per square foot, then group-action might be considered as about to start. It was uneconomical to allow group-action to start, and hence it was important to ascertain correctly the relative frictional support and bearing support. It would have been better to have used fewer but longer piles. The handling difficulties, limited space, etc., precluded, however, the use of longer pre-cast piles, whilst the length of driving tubes available made it difficult to use longer cast-in-situ piles. Mr. Remfry did not believe that group-action had actually developed in the foundations. Mr. Porter had pointed out that the consolidation was insufficient for group-action, and he estimated the consolidation as only about 4 per cent.; possibly, however, it was nearer to 5 per cent. throughout the greater part of the area below the ribs. A direct connexion should be traceable between the amount of consolidation and the commencement of group-action in any particular soil. In any case, the bottom of a group had to be capable of carrying a greater concentration of load than obtained at that level before the group was formed.

The conditions in Calcutta for arch bridges were not ideal, but the difficulties could be, and had been, overcome. At Dum Dum bridge the settlement had been unexpected, but it was due almost entirely to the disturbance of the soil below the foundations which occurred in removing the existing foundations of a previous bridge.