Loading Test on an Unorthodox Concrete Cuff Pile

Essai de charge sur un pieu, en béton à manchette non orthodoxe

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SUMMARY

To cross the Weespertrekvaart Canal in Amsterdam a lift bridge was built; the bulkheads required piles with a high bearing capacity. The subsoil, below a layer of about 12 m of very weak soil, consisted of two sand layers at depths of about 13 and 15 m, separated by a layer of poor resistance. With a view to these soil conditions a special "cuff pile" was designed. This consisted of a reinforced concrete pile which was enlarged at a distance of about 2.5 m from the pile tip; the enlarged cross-section acted as a cuff. The tip and the cuff were placed in the centres of the two successive sand layers, thereby increasing the bearing capacity as was ascertained by loading a test pile provided with strain gauges.

SOMMAIRE

Pour les fondations sur pieux d'un pont, traversant le canal Weespertrekvaart à Amsterdam, il était nécessaire d'avoir des pieux à haute résistance. Les caractéristiques étaient les suivantes: sous une couche de sol à très faible résistance et d'une épaisseur d'environ 12 m, se trouvent deux couches de sable à une profondeur d'environ 13 et 15 m, séparées par une couche de sable argileux de faible résistance. Pour avoir le maximum avantage des conditions du sol, c'est-à-dire des deux couches sablonneuses, un type de pieu en béton armé fut utilisé avec une "manchette" à une distance de 2,5 m de la pointe du pieu. L'augmentation de la capacité portante a été confirmé par le chargement d'un pieu d'essai équipé de comparateurs.

IN AMSTERDAM, cone penetration tests are used to determine the desirable depth of piles. As a rule, strata having sufficient bearing capacity are found at depths of about 11 to 12 m; the overlying strata consist of very soft peat and more or less sandy clay. As a result, every building and structure has to be founded on piles. To determine the depth and also the bearing capacity of the necessary piles, about 1,000 to 1,500 cone penetration tests are carried out yearly by the city of Amsterdam.

At the site of a lift bridge to be built across the Weespertrekvaart Canal, a rather uncommon result was obtained for the cone penetration tests. As shown in Fig. 1, the usual sand layer is encountered at a depth of about 12 m, but it consists of two layers with maximum resistance at depths of about 13 and 15 to 16 m. In between is a layer of rather low resistance. From depths of 17 to 25 m, clayey sand layers are encountered and a firm sand layer starts only at about 26 m. It was felt that the bulkheads, with a view to the moving part of the bridge, should be extremely strong and show no settlements of any importance. Hence the bearing capacity of the piles should be as high as possible.

Driving piles to depths of 26 m would have been possible but expensive. To place the piles in the upper sand layer at a depth of about 13 m would have been the normal solution. However, settlements were expected in the clayey sand layer at a depth of 14 m. Because of the negative skin friction to be expected, the piles would have to be provided with large tips, which made it unlikely that the piles could have been driven into the second sand layer which was at a depth of about 16 m. Therefore it was decided, in order to make use of both upper sand layers, to leave out an enlargement at the tip proper and to place it higher along the shaft of the pile in the form of a cuff. Then the pile tip could be driven down into the deepest upper sand layer and the location of the cuff could be chosen in such a way that it came to rest in the highest sand layer.

TEST PILE

In order to see if the idea was sound, a test pile was made as shown in Figs. 2 and 3. The cross-section was 32 by 32 cm except at the cuff where it was 45 by 45 cm. The cuff was placed about 2.50 m above tip level. The site of the test pile is given in Fig. 4. The cone penetration test shown in Fig. 1 can be considered as representative. Other cone penetration tests taken at the site show a similar picture. The test pile was driven to a depth of 15.30 m below normal Amsterdam level where it met near refusal. A hammer weighing 3 tons and dropping from a height of 0.60 m was used. The last 30 strokes of the hammer caused a final penetration of the pile of 0.5 cm.

Arrangement of Loading Test

The arrangement for loading the test pile is shown in Fig. 5. The maximum load available was about 150 tons delivered by cast-iron kentledge and additional tension piles. The pile was successively subjected to increasing loads. Each load was maintained for about 21 hours, during which time the process of settlement could be observed as shown in Fig. 6. After that period the pile was unloaded and the same load repeatedly applied and taken off in order to study the behaviour of the pile under the conditions of repeated loading and unloading. The load was applied with the use of a hydraulic jack (Fig. 5). The load on top of the pile was measured with a dynamometer fitted with wire resistance strain gauges.

For each loading step the stresses were measured in the tip of the pile and just below the cuff (Fig. 3). At these places wire resistance strain gauge elements were embedded in the pile. These elements, manufactured at the Delft Technological University, proved to be extremely reliable. Although they were subjected to heavy strains as the pile was driven down, there was no zero drift during the loading test. Also, the stresses during pile driving could be recorded accurately. These results are not given here.

Test Results

The result of the loading test is plotted in the load-settlement–time curve of Fig. 6. The loads measured at the

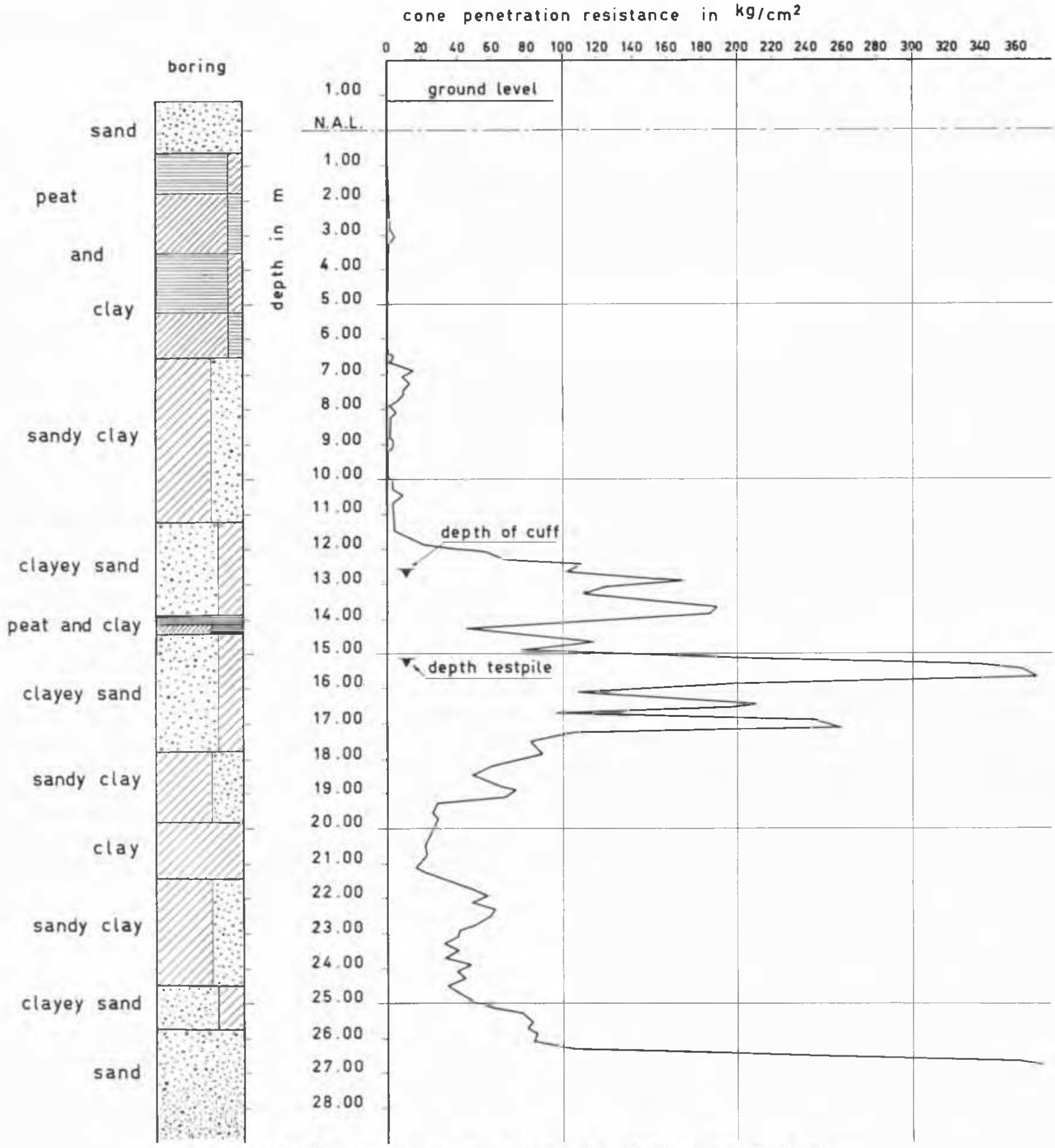

FIG. 1. Cone penetration test at the site of the test-loaded pile.

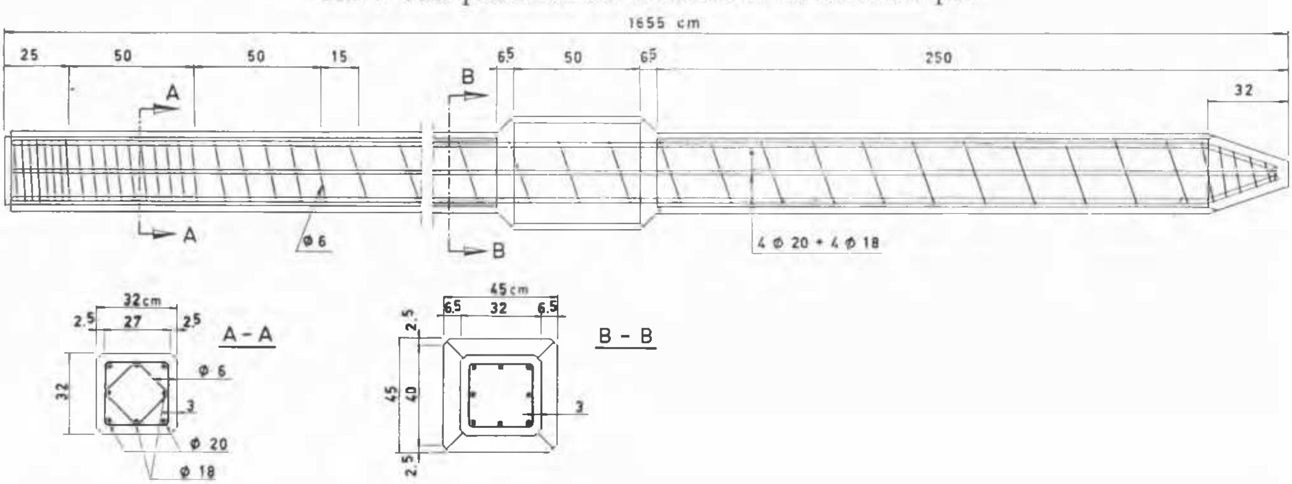

FIG. 2. Test pile with cuff.

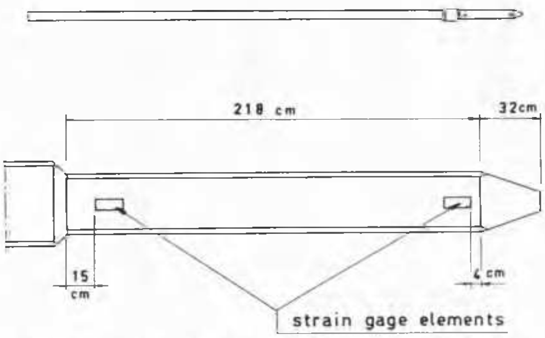

FIG. 3. Position of strain-gauge elements in test pile.

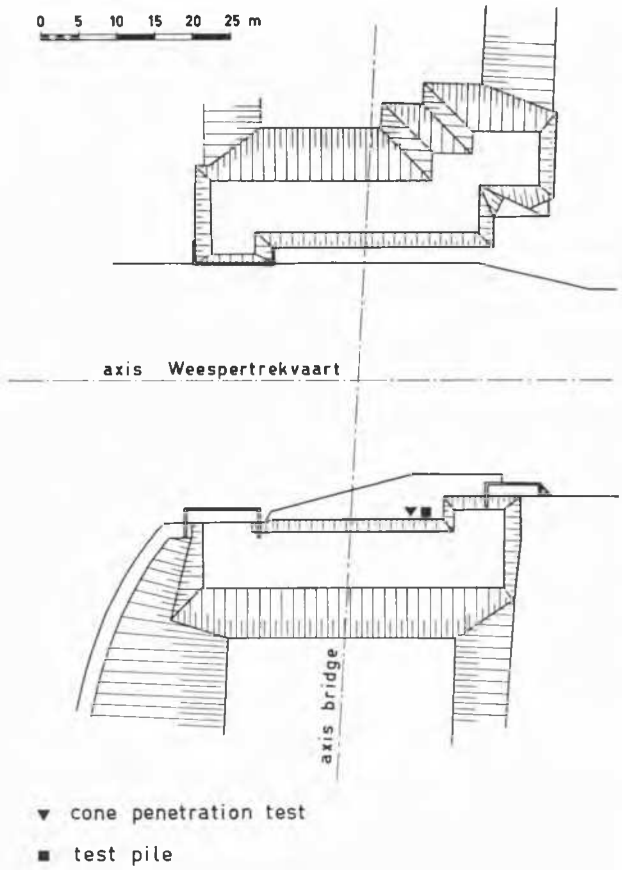

FIG. 4. Site of test pile and cone penetration test.

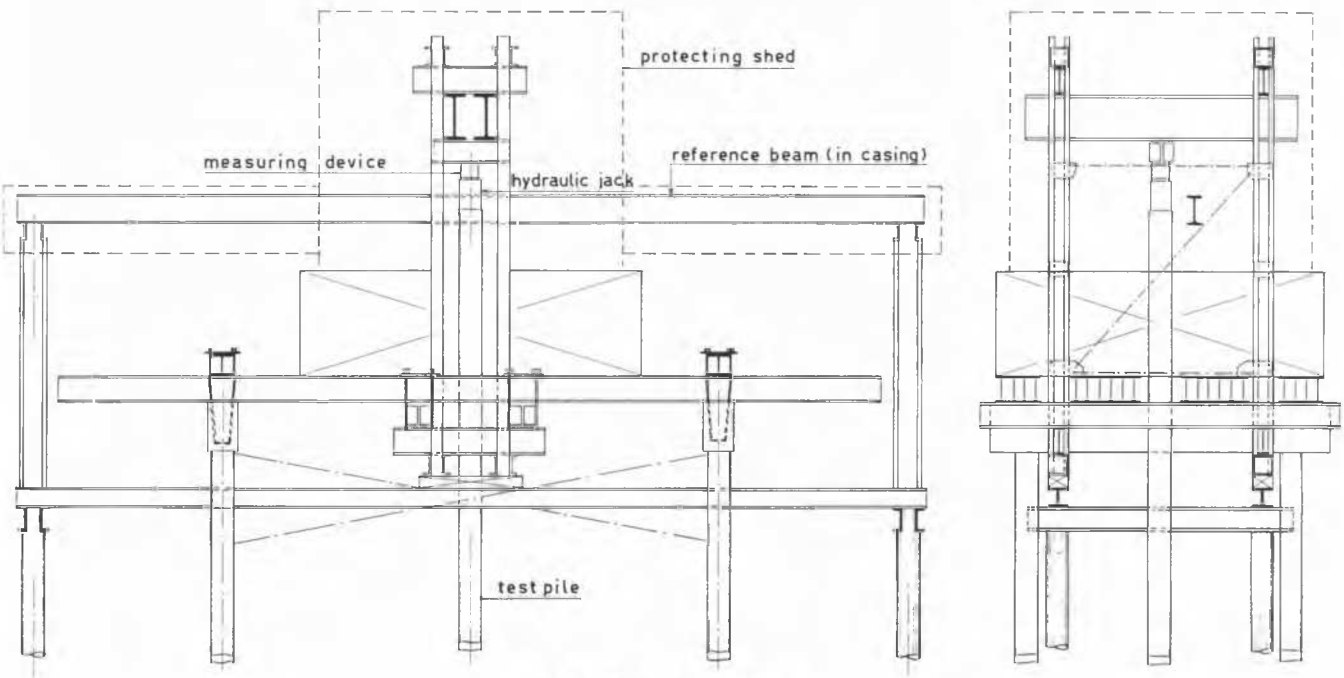

FIG. 5. Loading platform.

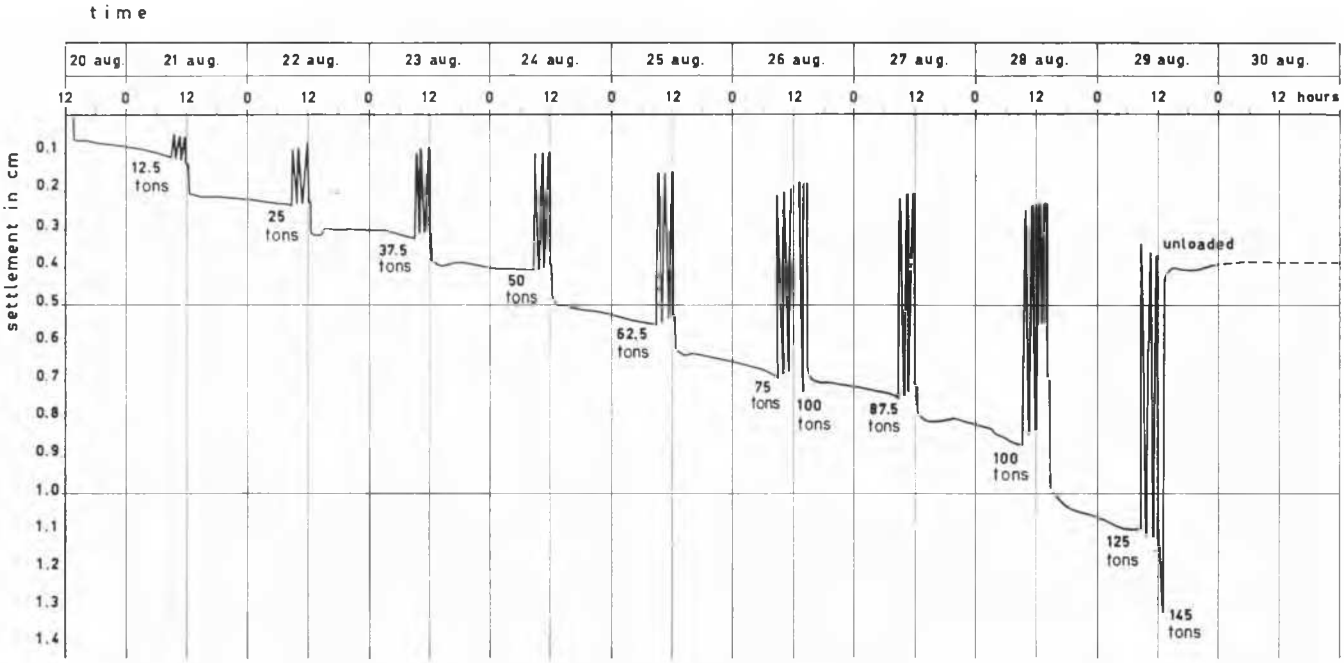

FIG. 6. Results of loading test.

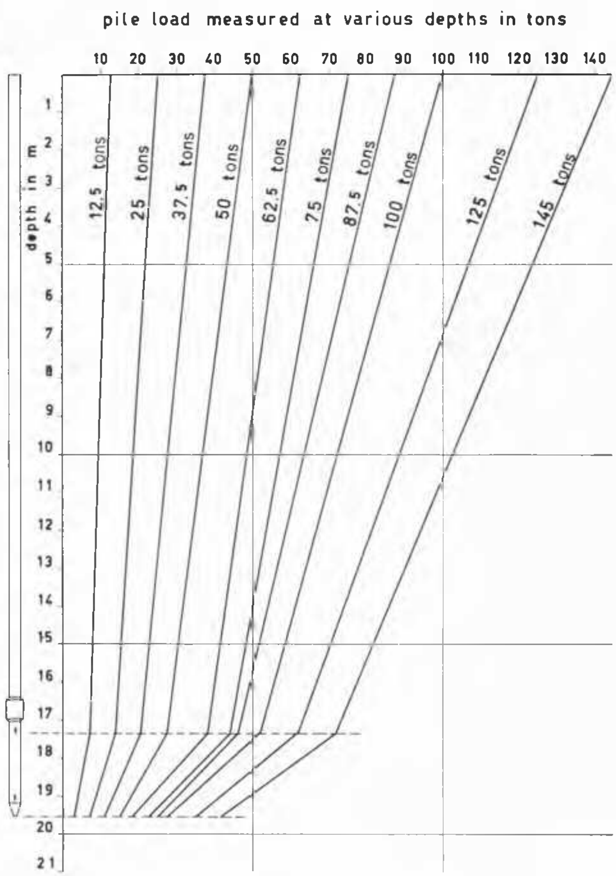

FIG. 7. Pile load measured at various depths.

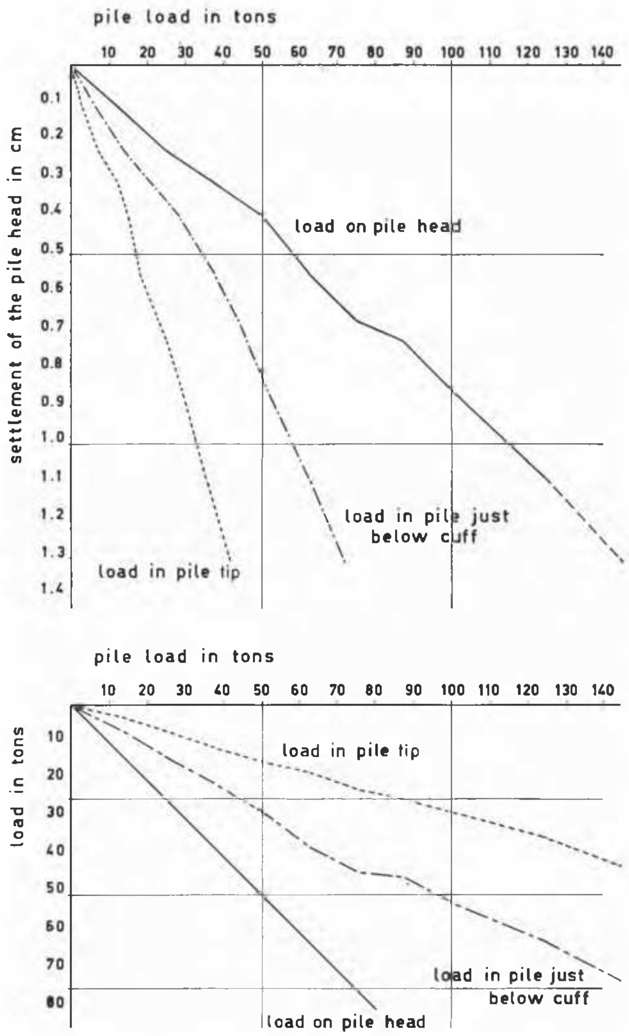

FIG. 8. Load settlement curve of test pile.

top, cuff, and tip of the pile are given in Fig. 7. The measurements have been arranged in a different way in Fig. 8, where the different loads in the three places are plotted as a percentage of the pile load at the top of the pile. It appears that the percentages are more or less constant throughout the test for each successive loading step. On an average, the pile tip takes about 30 per cent of the total pile load. As there is no friction, because of the poor soil layers above the cuff, the cuff itself takes about 50 per cent of the pile load. The remaining 20 per cent is taken by friction along the shaft of the pile beneath the cuff.

It appears that a high percentage of the bearing capacity is taken by the cuff. This confirms the expectation that the sand layer in which the cuff is placed would be compacted by the penetration of the lower pile end. This is also illustrated by the increased difficulty in driving the pile once the cuff entered the upper sand layer.

The highest test load was 145 tons. For that load the pile tip took 42 tons as its share, which is about 30 kg/sq.cm. The cuff in that case carried 73 tons which amounts to 55 kg/sq.cm., and is nearly double the resistance at the pile tip. Friction along the shaft below the cuff was 31 tons; this is about 11 tons/sq.cm., which is fairly high in the partly clayey sand layer penetrated by the lower pile end.

CONCLUSIONS

It is believed that the cuff pile may be used to advantage when the firm layer, in which the pile is to be placed, shows a fairly low resistance over part or all of the depth. In such a case the lower end of the pile serves to compact the sand and to provide a more rigid foundation layer for the ensuing cuff. Also, this type of pile transfers the pile load to the bearing soil stratum over a greater height than through a pile with an enlarged tip at the very end of it. If normal piles, without enlarged areas, had been used, they would have had to be driven much deeper in order to gain the same amount of bearing capacity through increased friction along the shaft.