Behaviour of a Flexible Piled Foundation

Comportement d'une Fondation Flexible sur Pieux

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SYNOPSIS Detailed settlement records have been obtained over an eight year period relating to a building in central London supported on under-reamed bored piles. The building is of particular interest because of its large plan area and flexible superstructure, which together invite the possibility of large differential settlements.

It is demonstrated how relatively simple methods of elastic analysis can be used to calculate the settlement profiles of large piled foundations. Several different methods are examined, and the results of each compared with measured values. The effect of differential settlement on column loads is assessed using a generalised moment distribution method, and the interactive behaviour of an underpass retaining wall traversing the site is also discussed.

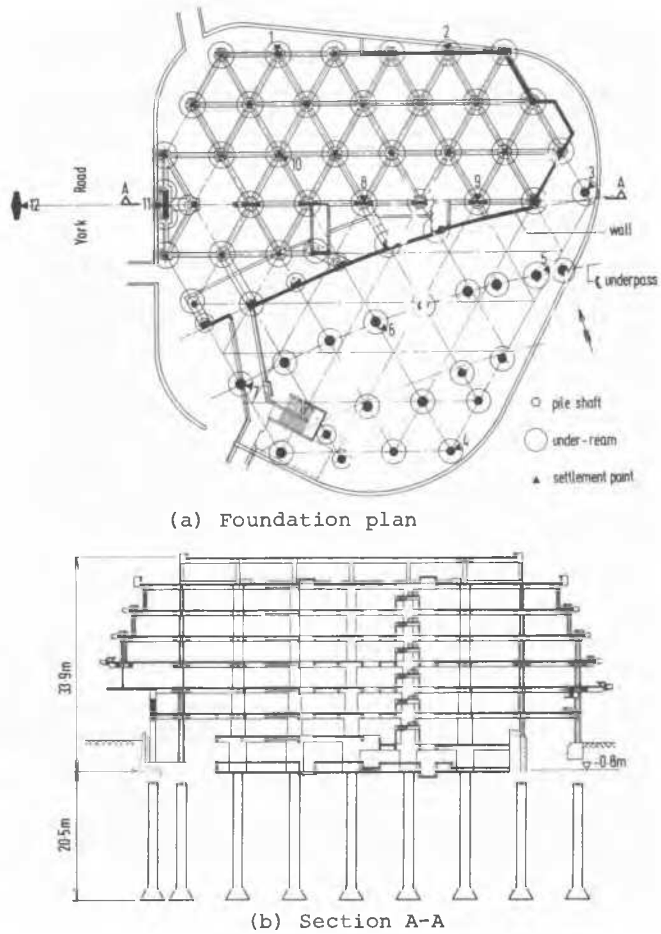
INTRODUCTION

Designers of large piled foundations are often faced with a rather perplexing array of methods for estimating the settlement characteristics of pile groups. It is usually the case that the more rigorous methods which treat the piles as discrete structural members turn out to be unsuitable, either because the pile group is too large or because of one or more important restrictions attached to such methods. Simpler and more approximate methods are not only desirable but often mandatory, especially in the early stages of design. It remains to be shown, however, that these alternative methods are capable of giving reasonable and reliable results.

The best way of testing methods of settlement analysis is to compare the measured and computed settlement profiles obtained for full-scale structures. This approach is often limited by the small differential settlements which generally occur in practice, and the consequent measurement difficulties. In contrast, the present building combines a large plan area with a flexible superstructure, and the measured differential settlements are of appreciable magnitude. This Paper deals with the interpretation of measured settlements by means of several different methods of simplified analysis.

THE BUILDING

The building comprises a 7-storey superstructure together with a single storey basement over part of the site. It provides office accommodation for the Greater London Council, and is located close to the south bank of the River Thames in central London. Structural details are shown in Fig.1.



(a) Foundation plan

(b) Section A-A

Fig.1 Structural details

The building is supported by 61 bored piles about 20m long and with under-reamed bases founded at the same level. Shaft and base diameters vary from 1.22 to 1.52m and 3.05 to 4.57m respectively. The top 5m length of pile passes through sand and gravel, the remainder being embedded in London clay. Most of the piles are positioned directly below the columns which are generally spaced 9m apart. The ground slab is only 225mm thick, and tie beams connect the small pile caps in the northern part of the building. In the southern part, provisions have been made for the future construction of a road underpass. The pile layout is modified in this area, and a reinforced concrete retaining wall (4.5m high, 375mm thick) traverses the building and forms the northern face of the underpass structure.

The basic layout of the superstructure consists of columns spaced 9m apart on a triangular grid, with floor beams spanning between the columns and supporting solid floor slabs. Cast in-situ concrete was used throughout, and the open-plan office floors have very few internal partition walls. At second floor level the building is connected to another office block by a walkway bridge.

The design pile loads vary from 3.40 to 8.42MN, with an average of 5.60MN. These loads relate to dead load plus reduced live load (excluding wind load), with no load assumed to be taken by the pile caps, ground beams and ground slab. The total design pile load is 342MN. This corresponds to an average applied pressure of 105kPa, based on a foundation plan area of $3260m^2$.

SOIL DATA

The general level of the ground surface varies only by about 1m across the site, which was previously occupied by low-rise dwellings. Six boreholes were sunk using shell and auger equipment, and the soil succession is 3.3m of made ground, 6.4m of sand and gravel, 31.2m of London clay followed by the Woolwich and Reading beds. The ground water level was found to be about 6.3m below the ground surface. Site investigation data for other nearby buildings suggest that the Woolwich and Reading beds are some 20m thick, followed by about 8m of Thanet sand and then chalk.

Several standard and cone penetration tests were carried out in the sand and gravel, and the average blow count was about 25. Laboratory test results for the London clay are shown in Fig.2. Plotted values of shear strength (c_u) are based on the average of three unconsolidated undrained tests on 38mm diameter specimens extracted from 102mm diameter samples. Values of the coefficients of volume compressibility (m_v) and consolidation (c_v) were obtained from 76mm diameter oedometer specimens loaded to 107kPa in excess of the effective overburden pressure at the sample depth.

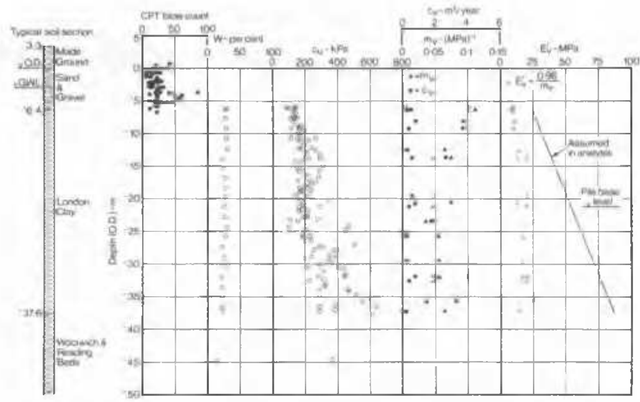


Fig.2 Soil data

MEASURED SETTLEMENTS

Settlements were measured at the 12 locations shown in Fig.1(a), commencing July 1972. At each location, a screwed socket was cast into the column just above ground floor level. Measurements were taken using a high precision instrument and the levelling closing errors, usually less than 1mm, were distributed on each survey.

Detailed settlement records are given elsewhere (Hooper, 1980), but measured settlement profiles across one section of the building are shown in Fig.3. The maximum angular distortion ($\Delta w/L$) appears to occur between levelling stations 3 and 9. Here Δw is approximately 9mm, based on the July 1979 readings, giving $\Delta w/L = 1/2000$. If allowance is made for the settlements occurring prior to the first set of readings, $\Delta w/L$ increases to about 1/1200. If further allowance is made for the column located mid-way between levelling stations 3 and 9, then the angular distortion of the outer bay is likely to be about 1/700.

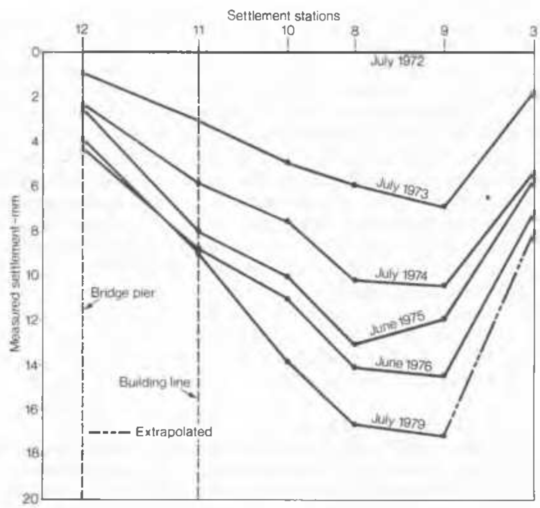


Fig.3 Measured settlement profiles

RETROSPECTIVE FOUNDATION ANALYSIS

Although a rigorous analysis of piled foundations in layered strata appears to be virtually intractable, a review of previous work (Hooper, 1979) suggests that various approximate methods of elastic analysis can be used to considerable effect. Two methods have been found particularly useful in assessing the settlement characteristics of large pile groups supporting building structures (Fig.4). The first is to completely ignore the presence of the piles and apply the vertical structural load at or near pile base level. The second is to replace the pile group by a block of pile - reinforced material of the same overall dimensions as the actual group.

Initially, these two approximate methods were applied to the present foundation problem using axisymmetric finite element modelling. In each method the mesh remained the same, but the elastic parameters of elements above pile base level were varied to suit each case.

The bending stiffness of the superstructure was calculated to be that of a 1m thick 'equivalent' raft, and the value $K_r = 0.1$ is obtained for the relative raft stiffness. This low value of K_r suggests that the superstructure is having virtually no effect on differential settlements, provided that the interfacial shear tractions between the soil and ground floor slab are small; see, for example, Brown (1969) and Hooper (1974). In the axisymmetric analyses, therefore, superstructure stiffness was ignored and a uniformly distributed load of 105kPa applied directly to the soil.

Each soil layer, as well as the pile-reinforced block, was assumed to be transversely isotropic with a vertical axis of symmetry (Hooper, 1975), with drained elastic parameters of $E_h' = 2.3\ E_v'$, $G_{vh}' = 0.66E_v'$, $\nu_{vh}' = 0.1$, $\nu_{hh}' = -0.15$. Values of the drained vertical modulus E_v' were taken as 4000 MPa for the chalk, 200 MPa for the Woolwich and Reading beds and 100 MPa for the sand and gravel. These are the values used previously (unpublished work) in a number of other retrospective foundation analyses associated with buildings in the London area. A trial and error approach was adopted to estimate the variation of E_v' with depth for the London clay. Again based on previous work, the first relationship tried was $E_v'(z) = 25 + 2z$, where z denotes the depth below the upper surface of the London clay, and the units are in MN,m. In the undrained analyses it was assumed that $E_v = 1.65\ E_v'$, $E_h = 1.8\ E_v$, $G_{vh} = 0.4\ E_v$, $\nu_{vh} = 0.499$, $\nu_{hh} = 0.1$ for the London clay, with the remaining materials being considered as permanently drained.

The elastic properties of the pile-reinforced block were estimated using the method described by Hooper (1979). Let A denote the plan area of the foundation, and let E_p and A_p denote the modulus and total cross-sectional area of the pile shafts respectively. Then the vertical compression modulus of the pile-reinforced soil is given by

$$E_{ps} = \left[1 + \lambda(K_p - 1)\right] E_v' \tag{1}$$

where $\lambda = A_p/A$ and $K_p = E_p/E_v'$, the remaining four drained elastic parameters being set equal to those of the soil surrounding the piles. This in turn gives a value of about 10 for the relative vertical stiffness of the pile group, defined as

$$K_{ps} = \frac{E_{ps}}{E_v'} = 1 + \lambda(K_p - 1) \tag{2}$$

which is substantially lower than usual and reflects the widely spaced pile layout.

Computations based on these initially assumed soil parameters gave encouraging agreement with measured total settlements, as shown in Fig.5. In plotting the measured curves it was assumed that a settlement of 10mm occurred before the first readings were taken. The computed curves relate to the maximum settlements of the pile-reinforced block. It is noted from Fig.2 that the values of E_v' for London clay assumed in the analysis are much higher than those deduced from the results of laboratory oedometer tests.

The variation of settlement with depth below the centre of the foundation w(0) is shown in Fig.6 for the two approximate methods of analysis. The finite element (F.E.) results relate to models 1(b) and 2, although the same pattern of results was obtained for all three models 1(a),(b),(c).

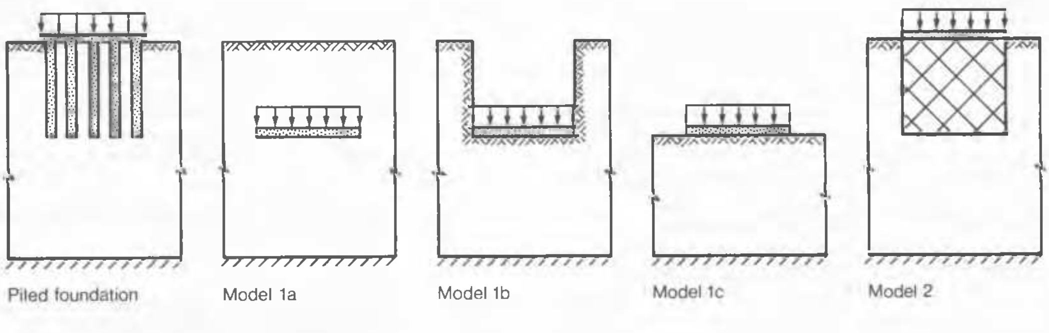


Fig.4 Approximate modelling of piled raft foundation

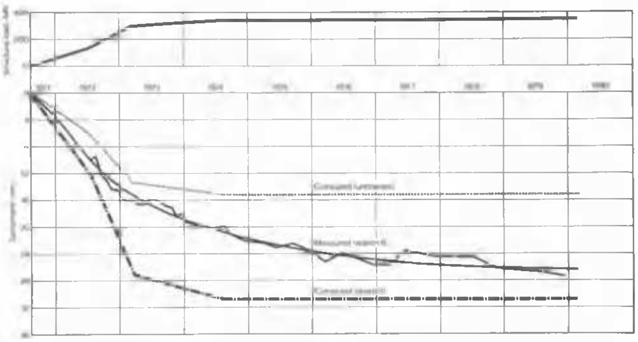


Fig.5 Measured and computed time-settlement curves

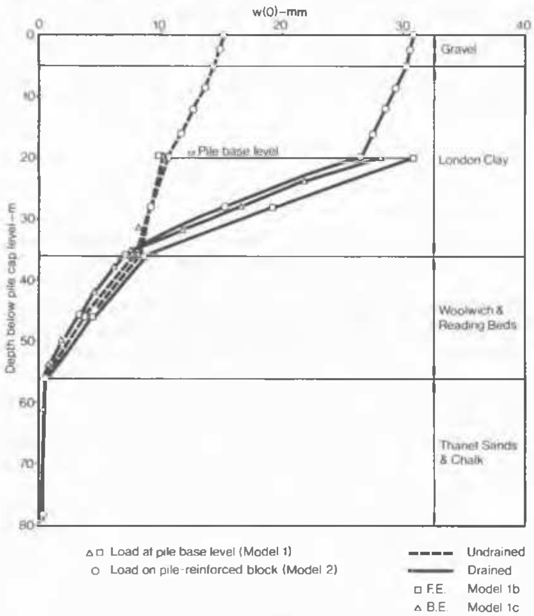
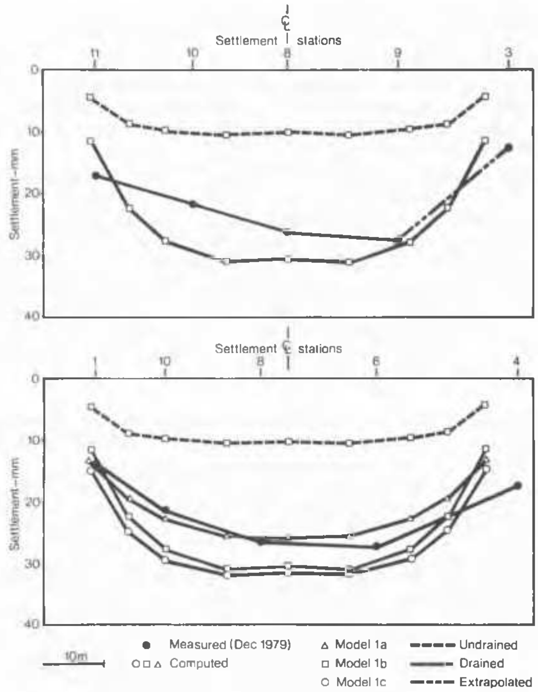
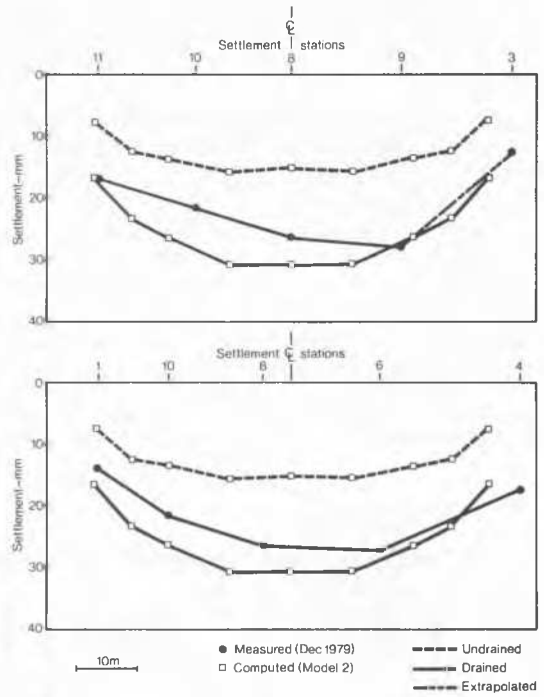


Fig.6 Computed settlements below centre of foundation

Profiles of measured settlement along two sections crossing the building are shown in Fig.7, together with results from various axisymmetric analyses. The smallest and largest differential drained settlements are obtained using models 1(a) and (b) respectively. Each model type 1 gave similar undrained settlements. Drained settlements obtained using model 2 fall between the values given by models 1(a) and (c), although undrained settlements are substantially higher. By way of contrast, the computed values of total and differential settlement for a uniformly loaded surface foundation, obtained by setting E_{ps} to E_v or E_v' in model 2, are about double those for model 1(c).



(a) load applied at pile base level



(b) load applied to pile-reinforced block

Fig.7 Measured and computed settlement profiles; axisymmetric finite element analysis

An alternative approach to piled raft analysis is to use model 1(c) but to represent the soil strata by some means other than finite elements. The raft is modelled by quadrilateral or triangular finite elements of the plate bending type, and is assumed to be in frictionless contact with the plane surface of a layered continuum located at pile base level. Settlements are calculated assuming a Boussinesq-type stress distribution and summing the strains based on moduli appropriate to the various soil layers. The required strain equations have been derived for a transversely isotropic continuum (unpublished work), and have been used successfully in other foundation studies; see, for example, Hooper (1978). The overriding advantage of this approach is that it can handle foundations of any plan shape and flexibility, any distribution of vertical applied load, and can take account of the presence of the superstructure.

The results of the drained analysis have been used to draw the settlement contours shown in Fig.8, which also gives spot values of measured settlement. The results relate to the 1m thick 'equivalent' raft, although the corresponding settlements with no raft were almost identical, entirely as expected in view of the low value of K_r. Measured and computed settlement profiles along two sections of the building are shown in Fig.9. Computed settlements below the centre of the building are included in Fig.6. They are labelled B.E. (boundary element) - despite the rather loose relationship of the present method with orthodox boundary element methods - in order to distinguish them from the results of the axisymmetric analyses.

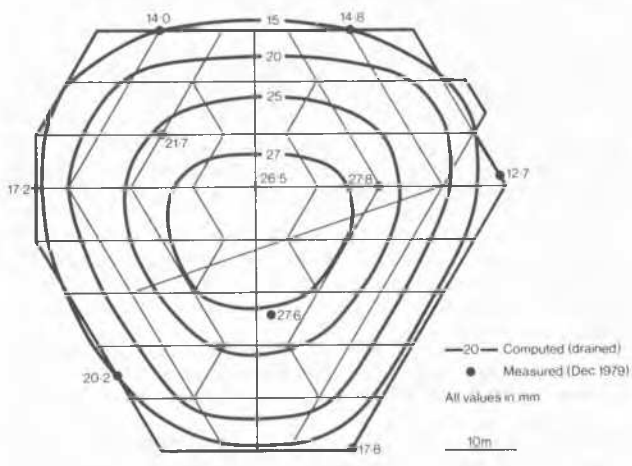


Fig.8 Drained settlement contours from boundary element analysis and spot values of measured settlement

An alternative way of examining the effect of superstructure stiffness on settlements is to determine the changes in column load associated

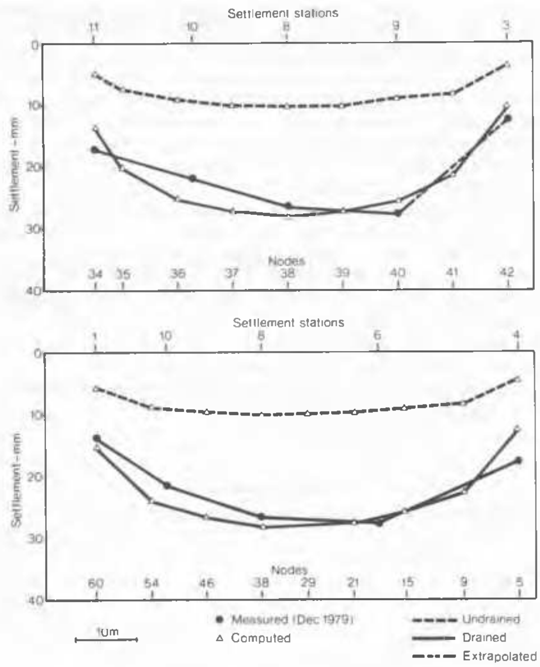


Fig.9 Measured and computed settlement profiles; boundary element analysis

with the differential settlements computed from the axisymmetric analysis of the pile-reinforced block. This was done as a hand calculation using a generalised moment - distribution method, assuming a symmetric 8-bay frame with equal spans and taking some account of the torsional stiffness of the floor beams. The estimated additional loads applied to the foundation by the superstructure are given in Fig.10, and range up to about 5% of the average column load.

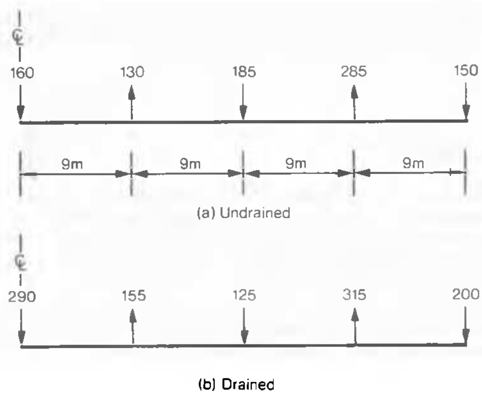


Fig.10 Estimated changes in column load (kN) induced by foundation settlements

It is also instructive to study the influence of the underpass wall on foundation settlements. This substantial reinforced concrete wall (4.5m high, 375mm thick) traverses the site and is monolithic with the adjacent columns and floor slabs. In the so-called boundary element method, the wall can be represented most conveniently by a series of beam elements attached to the raft, with element properties which model the bending and shear stiffness of the combined wall and floor slab components. The computed profile of drained settlement along the line of the wall is shown in Fig.11, together with the measured settlements of nearby levelling stations. These computed settlements are very similar to those obtained for the case of no wall, further suggesting that the stiffening effect of the wall is negligible.

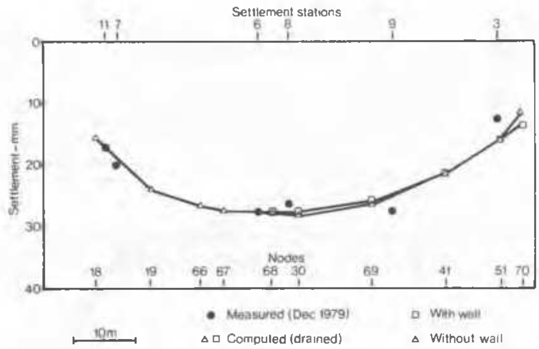


Fig.11 Computed drained settlements along line of underpass wall, and nearby measured settlements

An independent check on this stiffening effect can be made by considering the wall as a strip footing and calculating its stiffness K_s relative to the soil. This gives $K_s = 0.0015$ which, according to Brown (1975), corresponds to a very flexible footing. Thus it appears that the underpass wall does little to stiffen the superstructure, although the method of superposing beam elements on a plain raft can often be used to advantage in more general cases.

CONCLUSIONS

On the basis of measured values of total and differential settlement, together with the absence of any cracking in the primary structure, the piled foundation can be said to have performed entirely satisfactorily. It is likely that, because of the open-plan floor layout and the consequent flexibility of the superstructure, there has been very little redistribution of structural load resulting from differential settlement of the foundation.

Calculated settlements based on approximate methods of pile group analysis are in reasonable agreement with measured values. Particularly good correlation is obtained using the analogy of a pile-reinforced block, despite the widely spaced pile layout. Good results are also obtained using the so-called boundary element method, which is probably the most powerful and versatile of all the approximate methods of piled raft analysis. This has suggested that these approximate and relatively simple methods can be usefully employed in design to assess the behaviour of large pile group foundations. Recent experience on several projects ranging from low-rise housing to tall building structures has amply justified this approach, although the stiffening effect of the superstructure has been usually much more pronounced than in the particular case examined herein.

ACKNOWLEDGEMENTS

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