Comparisons between Theoretical and Observed Behaviour of Pile Foundations

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SUMMARY The paper describes a number of case studies in which the deformation behaviour of single piles or pile groups has been calculated theoretically and compared with the observed behaviour. The cases include: (a) the settlement of two grout-injected test piles in sand, for which "Class A" (before the event) predictions were made, (b) the settlement of steel pipe piles and step-taper piles in a soil profile consisting of clay overlying dense sand, (c) the lateral response of two test piles, the predictions being based on pressuremeter data, (d) the lateral load-deflection behaviour of pile groups tested in Rumania, (e) the settlement of a large office block founded on rock-socketted piles. In the last two cases, the results of tests on single piles have been used to predict group behaviour. The theoretical approach used for the calculations is based on elastic theory and is outlined briefly in the paper. The selection of the required soil parameters is also discussed. It is concluded that the theory used can give quite acceptable estimates of pile performance, particularly if used in conjunction with test pile data.

1 INTRODUCTION

The past decade has seen considerable advances in the development of theoretical methods for predicting the behaviour of single piles and pile groups subjected to axial and lateral loads. Several techniques have been employed, ranging from simplified closed form solutions (Randolph and Wroth, 1978) and boundary element methods of varying degrees of complexity (e.g. Poulos and Davis, 1968); Mattes and Poulos, 1969; Butterfield and Banerjee, 1971; Poulos, 1971; Banerjee, 1978) to finite element analyses (e. g. Ellison et al, 1971; Desai, 1974; Valliappan et al, 1974; Balaam et al, 1975). A number of comparisons have been made between the theoretical and observed behaviour of piles (Poulos, 1974; Butterfield and Ghosh, 1977; Banerjee and Davies, 1979) and these have generally demonstrated the applicability of the theory to both model and full-scale piles. Nevertheless, further comparisons are desirable in order to gain a better appreciation of the capabilities and limitations of the theoretical approaches, and also to gain further experience in selecting soil parameters to use with the theory.

The present paper describes six case studies of comparisons between theoretical and observed behaviour of single piles and pile groups. In all cases, the theoretical behaviour has been derived from an analysis which employs elastic theory but allows for non-linear behaviour at the pile-soil interface. A brief review of the theory is given and then each case is described in detail, with particular attention being paid to the method of selection of the requisite soil parameters. Two of the cases involve "Class A" predictions, in the terminology of Lambe (1973) i.e. made before the measurements were taken, while the remainder involve "Class C" predictions, made after the test results were known.

2 BRIEF REVIEW OF THEORY

2.1 Single Piles

The analyses used in this paper are all derived from the theory of elasticity, using a simplified form of the boundary element method. The analysis of a single axially loaded pile using this approach has been described by Poulos (1977) and involves the discretization of the pile into a series of shaft and base elements, each acted upon by an unknown pile-soil interaction stress. An expression for the axial deflection of each element of the pile can be written in terms of these interaction stresses by assuming the pile to deform as an axially loaded cylinder. A corresponding expression for the axial deflection of the soil adjacent to each element can be obtained by integration of the appropriate Mindlin elastic equation for vertical subsurface loading. The expressions for soil and pile deflections can be equated and solved to give the distribution of interaction stress, and hence deflection, along the pile. To allow for the possibility of pile-soil slip, limiting values of the interaction stress can be specified at each element. An interative analysis may then be performed to evaluate the load-settlement behaviour of the pile to failure. By introducing simplifying assumptions, it is also possible to consider, with reasonable accuracy, piles in non-homogeneous soils (Poulos, 1979a).

Single laterally loaded piles may be treated in a similar fashion. The pile is idealized as a thin beam, with the horizontal pile deflections being evaluated from beam bending theory and the soil deflections from integration of the Mindlin elastic equation for horizontal subsurface loading (Poulos, 1971). Again, by specifying limiting values of lateral pile-soil interaction stresses at each element, non-linearity of the lateral response of the pile can be reproduced.

For hand calculations, parametric solutions have been presented for the settlement and lateral deflection of a single pile (Poulos and Davis, 1980). For more detailed computations and cases not readily covered by these parametric solutions, computer programs have been developed (TAPILE for axial analysis, PULL for lateral analysis).

2.2 Pile Groups

For pile groups subjected to axial loading, a simplified analysis has been developed which involves the superposition of "interaction factors" for two piles (Poulos, 1968). These interaction factors represent the relative increase in displacement of a pile due to an identical loaded pile, and depend on pile spacing, geometry and stiffness relative to the soil. This analysis enables a solution to be obtained for the distribution of load and settlement within a group of piles and provides a theoretical relationship between the group settlement and the settlement of an isolated single pile. Such an approach is very useful from a practical point of view in that it enables the results of a load test on a single pile to be used to predict the behaviour of a pile group.

A similar approach has been developed for laterally loaded pile groups except that consideration needs to be given to interaction factors for both pile head deflection and rotation due to lateral load and moment. Furthermore, depending on the head conditions imposed by the pile cap, it may be necessary to consider both the lateral and axial response of the piles.

While it is possible to obtain parametric solutions for the axial and lateral behaviour of pile groups, the range of variables which can be covered is necessarily limited, and hence more frequent resort must be made to a computer analysis than is the case for single piles. A program called DEFPIG has been developed for this purpose.

2.3 Soil Parameters

The major problem in the application of the above theories (or indeed any theory) to practice, is the selection of appropriate soil parameters. It has been found that conventional laboratory tests are of little direct value for predicting pile performance. While in-situ tests may provide a more reasonable basis for parameter selection, their utility still remains to be fully investigated. The most reliable means of making predictions is considered to be to interpret the results of a load test on a single pile in terms of the theory, and then to use the values of soil modulus and limiting pile-soil stress so derived to predict the behaviour of the prototype piles or pile groups. In the absence of pile test results, a number of empirical correlations have been developed for preliminary estimates of piles in clay or sand. For clay, these correlations relate the soil modulus to undrained cohesion while for sand, the soil modulus is related approximately to the relative density of the soil (Poulos, 1974). Values of limiting skin adhesion and lateral pile-soil pressure can be estimated from conventional theories of ultimate pile resistance.

3 SETTLEMENT OF GROUT-INJECTED PILE AT SURFERS PARADISE, QUEENSLAND

In 1977, a load test was performed on a grout-injected pile in Surfer's Paradise, Queensland, at the site of a new residential development. The details of the pile and the subsoil profile are summarized in Fig.1. Prior to the load rest results being revealed, a "Class A" prediction was made of the load-settlement behaviour of the pile. The only quantitative information on the subsoil properties consisted of SPT values, and on the basis of these values, the soil profile was classified as being very dense sand. Based on the suggestions of the SAA Piling Code (1978), a constant soil modulus of 100 MPa was assumed, while the corresponding distribution of ultimate skin friction is shown in Fig.1. The ultimate base resistance was calculated using a value of the bearing capacity factor, N_q, of 180. The load-settlement prediction was made using the program TAPILE.

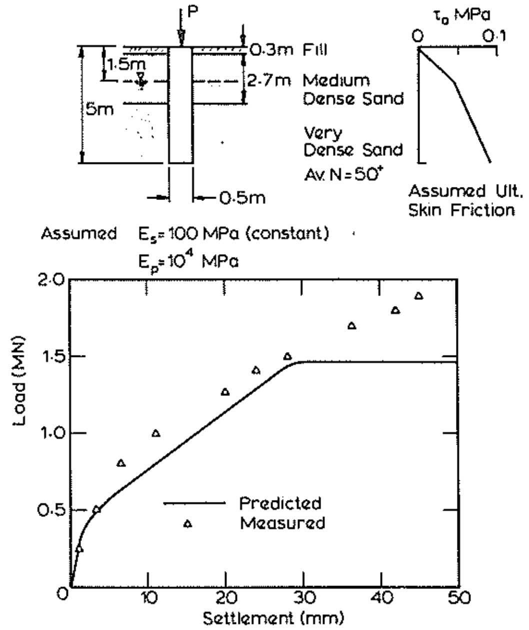


Figure 1 Predicted and measured settlements of grouted pile-Surfer's Paradise

The predicted load-settlement curve is shown in Fig. 1 together with the measured curve. The agreement is good up to a load of about 500 kN (the working load on the pile was to be about 550 kN). Beyond 500 kN load, the theory overestimates the settlement somewhat and predicts that full shaft slip occurs and that any additional load is carried by the base. The predicted ultimate load was 1460 kN whereas the actual pile carried a load of 2300 kN with a settlement in excess of 80 mm. Even if the failure load is defined as that to cause a settlement of 10% of the base diameter, it would be found to be about 2000 kN, still considerably in excess of the predicted value. The difference is believed to result primarily from an underestimate of the base bearing capacity, as the average soil modulus and ultimate skin friction values appear to have been chosen with reasonable (if fortuitous) accuracy.

Nevertheless, it may be said that, for a "Class A" prediction, based on very limited soil data, the theory provided excellent agreement with the measured behaviour up to and beyond the proposed working load.

4 SETTLEMENT OF TEST PILE IN HAMILTON, N.S.W.

In November, 1978, a load test was carried out on an instrumented grout-injected test pile in Hamilton, N.S.W. The soil profile consisted of layers of clay and sand to 6 m depth overlying an 8 m thick sandy clay layer which was in turn underlain by very stiff shaley clay (see Fig.2). The pile was 12 m long, 0.45 m diameter, and was reinforced with a helical spiral steel cage 11.8 m long. It also contained a central pipe to which resistance wire strain gauges were cemented at nine locations in order to deduce the load distribution along the shaft.

Prior to the load test, a "Class A" prediction was made of the load-settlement curve to failure and the load distribution in the pile at various load levels. Load-settlement predictions were made both by using a hand calculation procedure proposed by Poulos (1972), and also by use of the program

TAPILE. The latter program also produced the load distribution along the pile shaft. From the average value of undrained cohesion deduced from the cone

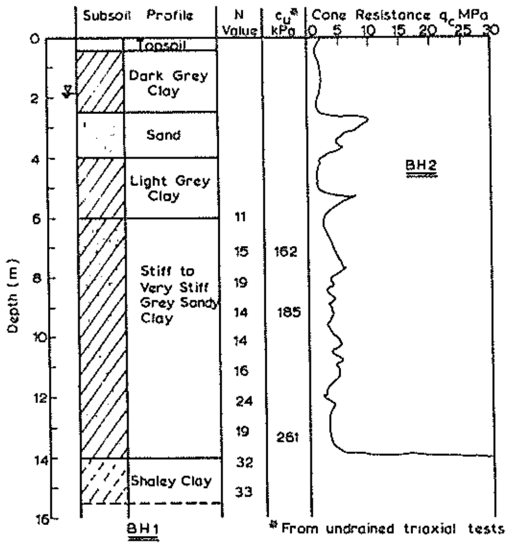


Figure 2 Geotechnical data for Hamilton site

resistance data (taking c_u=cone resistance/15), a constant soil modulus E_s of 70 MPa was selected for the hand calculations, based on the correlations of Poulos (1972). For the computer predictions, a value of E_s varying linearly with depth along the shaft from 40 MPa to 80 MPa was chosen. The ultimate skin friction τ_a was estimated on a total stress basis from the deduced c_u values, and was taken to be 40 kPa to a depth of 6 m, increasing linearly to 65 kPa at pile tip level. For the ultimate base resistance, a value of 9 c_u(=1660 kPa) was taken. The pile itself was assumed to have a Young's modulus of 21 GPa.

Fig.3a compares the predicted and measured load-settlement curves. The latter were corrected to exclude time-dependent movements during a period of maintained loading and load removal at a load of 380 kN. The measured settlements are in remarkably good agreement with both the hand and computer predictions up to a load of about 600 kN (the design working load for this pile was 390 kN). At larger loads, the measured settlements significantly exceed the predicted values, and it is obvious that the actual failure load was significantly less than the predicted value of 1055 kN. Both the ultimate skin and base resistances appear to have been overestimated in this case.

Fig.3b compares the measured and predicted axial load distributions at loads of 300 kN and 800 kN. The agreement is quite good for both cases, except that the actual load in the pile near the lower part of the pile is greater than predicted, suggesting that perhaps the soil beneath the pile tip may have been somewhat stiffer than assumed. It is interesting to note that, prior to the application of the test loads, the strain gauge readings indicated considerable tensile strains in the pile, due possibly to the swell of the grout during curing or the expansion of the grout due to the development of high temperatures during initial curing.

In summary, the settlement behaviour up to working load has been quite well-predicted, as in the case of the Surfer's Paradise pile. However, the ultimate load prediction has again been far from accurate; this finding is rather surprising, as it is often considered that ultimate pile loads can be more accurately predicted than pile settlement.

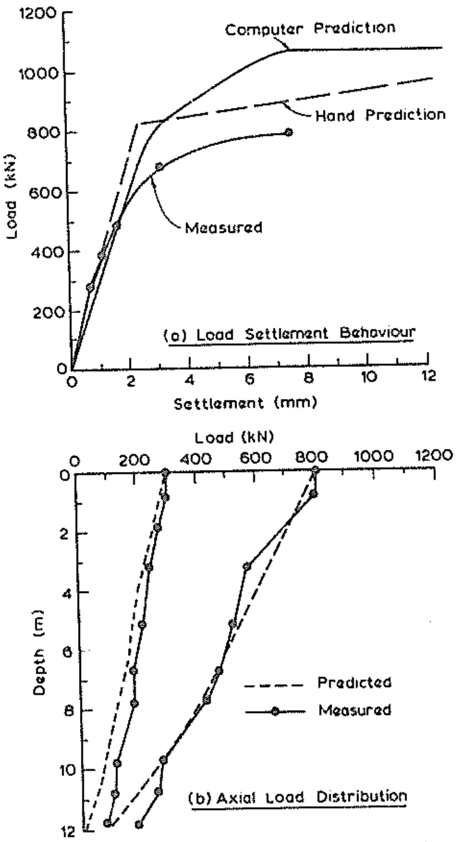


Figure 3 Measurement vs prediction-Hamilton test pile

5 TESTS OF SULLIVAN (1973)

Sullivan (1973) reported the results of load tests on four piles, (two steel pipe piles and two step-taper piles) in a soil profile consisting of stiff clays and silty clays overlying dense fine sand overlying further stiff clay. Fig.4 shows the profile and the available geotechnical data. "Class C" (after the event) predictions of the load-settlement behaviour of the four piles during maintained loading tests were made in two ways:

(i) using soil parameters selected on the basis of available soil information at this side and previously-developed empirical correlations;

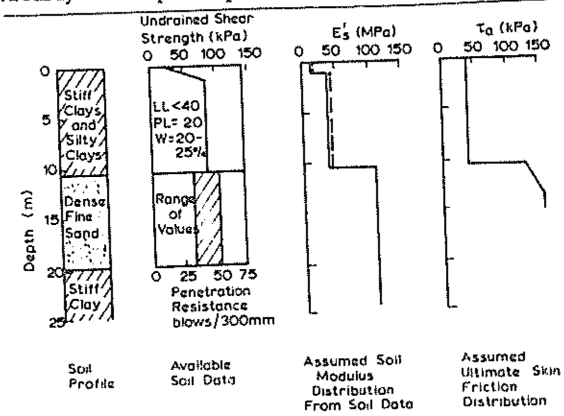


Figure 4 Soil conditions and parameters for tests of Sullivan (1973)

(ii) using soil parameters backfigured from the results of the load tests on one of the steel pipe piles (Pile 1).

For method (i), empirical relationships between drained soil modulus E'_s and c_u were used for the clay while E'_s for the sand was taken as 110 MPa. For both soils, the drained Poisson's ratio ν'_s was taken as 0.35. In the clay, the ultimate skin friction was taken as 43 kPa, and in the sand, τ_a was calculated on an effective stress basis as $F\sigma_v'$, where σ_v'=vertical effective stress and F was taken as 1.0 because of the dense nature of the sand. Based on the SAA Piling Code (1978), τ_a was assumed to reach a limiting value at a penetration of 8 diameters into the sand. Fig.4 shows the distributions of E'_s and τ_a thus determined. The ultimate base resistance P_{bu} was also calculated on an effecitve stress basis as $N_q.\sigma_v'$, with N_q=100, and σ_v' assumed to reach a limiting value at a penetration of 8 diameters in the sand.

For method (ii), the measured settlement of Pile 1 was fitted to elastic theory at a load of 670 kN to determine an equivalent average drained modulus of the soil profile. The resulting value of E'_s was 61 MPa.

Having derived the required input parameters, the program TAPILE was then used to predict the load-settlement behaviour of the four test piles. Fig.5 shows the curves thus obtained, together with the measured curves. The following observations may be made: (i) The predicted load-settlement curves from both methods are in reasonably close agreement.
(ii) The predicted settlements are in fair agreement with the measured values for Piles 1 and 3, but are more than the measured values for Piles 2 and 4. The reason for the somewhat stiffer measured behaviour of Piles 2 and 4 is not clear, as quick-loading tests carried out on the piles after completion of the maintained loading tests indicated that the settlement of all four piles at working loads was quite similar. This finding is in agreement with the theoretical predictions.

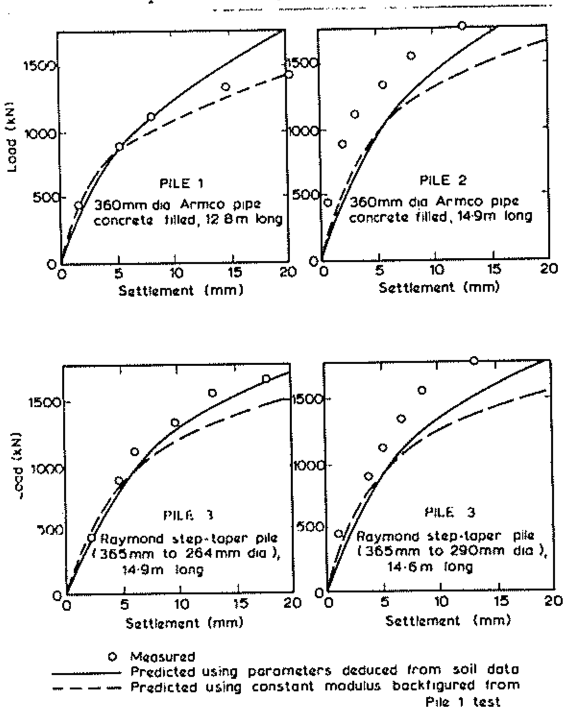


Figure 5 Comparison between measured and predicted load-settlement curves tests of Sullivan (1973)

(iii) The calculated ultimate load capacities are in generally good agreement with the values indicated from the load tests.

Piles 1 and 3 were each instrumented with "tell-tales", thus enabling the axial load distribution to be determined. Fig.6 shows comparisons between these measured distributions and those calculated from the TAPILE analysis, for a working load level of 530 kN. Bearing in mind the limited accuracy of the measured distributions because of the small number of tell-tales, the agreement with the theoretical load distributions is reasonable.

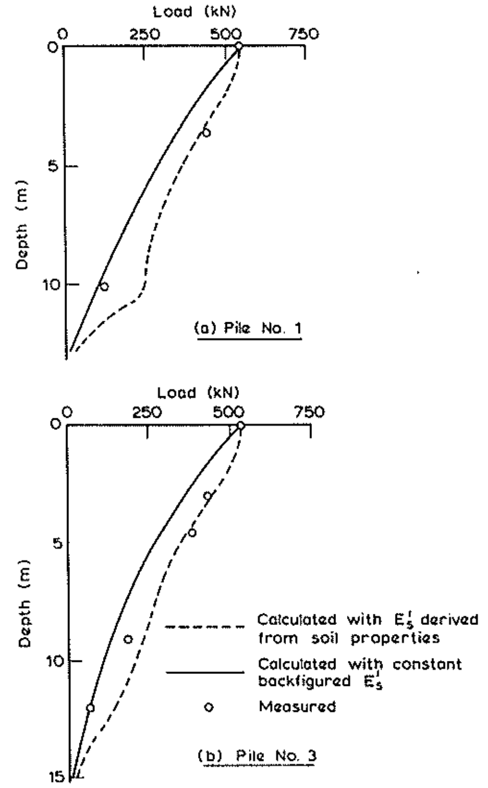


Figure 6 Measured vs predicted load distributions tests of Sullivan (1973)

This case study indicates that the theory can give reasonable, if somewhat conservative, predictions of load-settlement behaviour of both uniform diameter and step-taper piles, using a common set of soil data. The usefulness of the previously-derived empirical correlations of soil modulus is again demonstrated. As with the two previous case studies, the predicted settlements at loads well towards the ultimate become inaccurate, but from a practical viewpoint, such a deficiency in the theory is not significant.

6 TESTS OF FRYDMAN ET AL (1975)

Frydman et al (1975) have reported the results of lateral load tests on two prestressed concrete piles driven through highly plastic clay into fine dense sand in the Haifa Bay area of Israel. Fig.7 shows the soil profile, together with geotechnical data obtained from SPT, vane and Menard pressuremeter tests. One of the piles (Pile A) was instrumented with a slope indicator casing along its length, and in order to minimize its head rotation, had its upper part encased in a rigid concrete block supported on knife edge supports resting on the ground

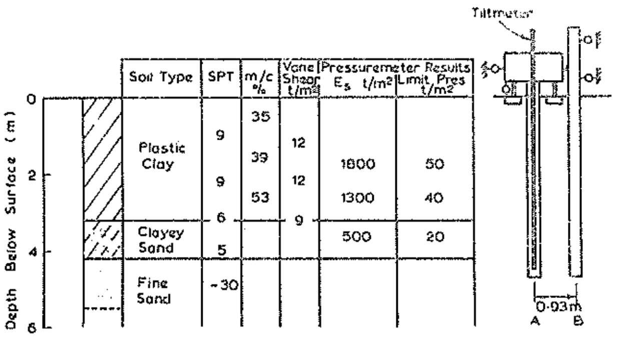


Figure 7 Soil profile and test pile arrangement (Frydman et al, 1975)

surface. The other pile (Pile B) was free-standing and had dial gauges at two points above the ground surface in order to measure both deflection and rotation (see Fig.7). Loading was applied by means of a jack acting between the piles.

In calculating the theoretical behaviour of the piles, it was decided to attempt to use the Menard pressuremeter tests directly to obtain the soil modulus E_s and the lateral pile-soil yield pressure P_y. E_s was taken to be the value determined directly from the pressuremeter results and reported in the paper. Py was taken to be $c_u.N_c$, where c_u was determined as the pressuremeter limit pressure divided by 5.5, and N_c varied between 2 at the ground surface to 9 at a depth greater than or equal to 4 diameters. For Pile A, the presence of the concrete block caused some difficulty in the choice of an appropriate head condition in the analysis. It was felt that the concrete block, while reducing head rotation, would not completely eliminate it, and it was therefore decided to assume that the block carried the moment load, while the shear was taken by the pile itself.

Two sets of calculations were carried out: (i) assuming Piles A and B are isolated and do not influence each other; the program PULL was used for this calculation, (ii) assuming Piles A and B interact; the program DEFPIG was used and the piles analyzed as a two-pile group, Pile A being subjected to shear only and Pile B to both shear and moment.

Fig.8 compares the theoretical and measured load-deflection curves for Pile A at the ground line. As would be expected, the theoretical curve allowing for interaction gives smaller deflections and rotations than if the piles are assumed isolated. The effect of interaction on rotation is less than on deflection. For loads up to about 60 kN, the agreement between both the theoretical curves and the measurements is quite good, but at larger loads, the measured values are larger than the theoretical values. Fig.9 compares theoretical and measured deflection profiles along Pile A at two different load levels. At the lower load of 43 kN, excellent agreement is found between the measurements and the theoretical curve including interaction effects. However at the higher load, the agreement is not as good, particularly near the surface. Whether this discrepancy is due to the assumed parameters or the pile head condition changing as the load increases, cannot be stated with certainty, although the latter appears to be more likely in view of the good agreement at lower load levels.

Detailed measurements were not reported for Pile B, but at a load of 43 kN, the measured deflection of Pile B 470 mm above the groundline was 4.1 mm. The

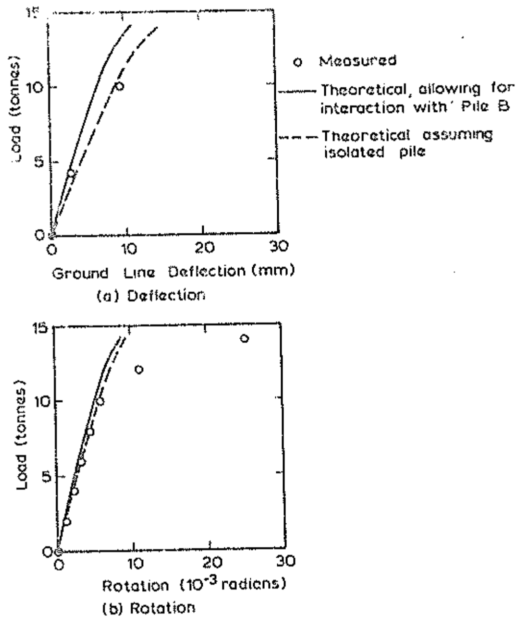


Figure 8 Comparison between theoretical and observed behaviour of Pile A (Frydman et al, 1975)

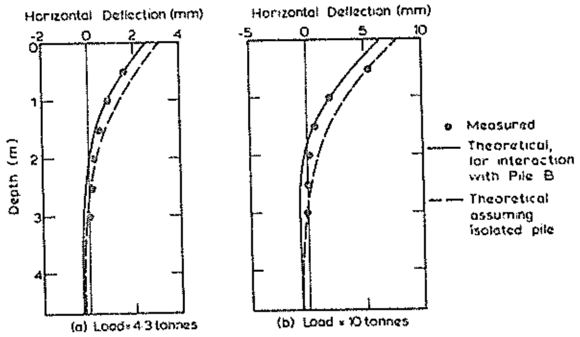


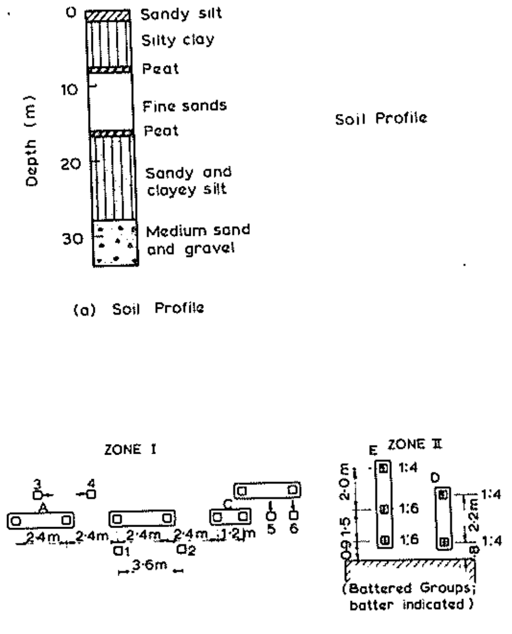
Figure 9 Comparison between theoretical and observed deflection profiles for Pile A (Frydman et al; 1975)

calculated deflection, assuming Pile B to be an isolated pile, was 6.3 mm; taking interaction into account gave a deflection of 5.8 mm, which is in slightly better agreement with the measurements.

This case study therefore suggests that the results of pressuremeter tests may be applicable to the prediction of lateral pile performance, although more case studies would be required in order to gain confidence in this application. In addition, it illustrates the importance of considering the interaction between two test piles jacked apart.

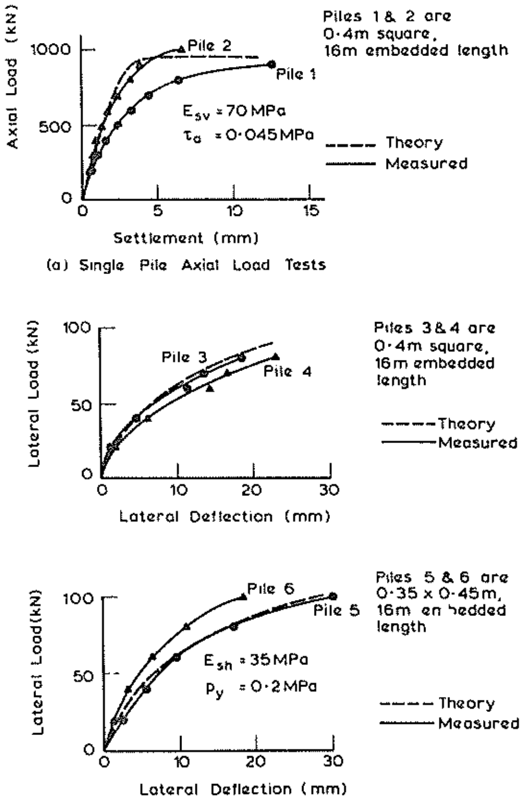
7 PILE TESTS IN RUMANIA (Manoliu et al, 1977)

Manoliu et al (1977) have presented the results of a program of field tests in Rumania on single piles and pile groups subjected to both axial and lateral loading. The subsoil profile at the site is shown in Fig.10a. Six single piles were tested, two under axial load, and four under lateral loading. The piles were of reinforced concrete, 17 m long (embedded length 16 m), and generally of 0.4 m square cross-section, although some were of rectangular section (0.35 m by 0.45 m). Five different groups were subjected to lateral loading, three groups of two vertical piles, one group of two battered piles, and



(b) Layout of Test Piles

Figure 10 Tests of Manoliu et al (1977)



(b) Single Pile Lateral Load Tests

Figure 11 Single pile tests

one group of three battered piles, as shown in Fig. 10b. The published data thus provided a good opportunity for the "Class C" prediction of the load-deflection behaviour of the pile groups, using the single pile behaviour in conjunction with the theoretical analysis of pile group response.

The first step in the prediction process was to attempt to backfigure the average soil modulus and limiting pile-soil stress values for axial loading by fitting the theoretical solutions to the measured load-settlement curves. This involved a trial and error process using the program TAPILE. Fig.11a compares the best-fit theoretical load-settlement curve and the two measured curves, and indicates the parameters used for the fit. Both the soil modulus, E_{sv}, and limiting skin friction τ_a, were assumed constant with depth. While a reasonable fit is obtained at relatively low load levels, the theoretical curve shows too stiff a response at higher loads. This is a previously-observed characteristic of analyses with constant E_{sv} and τ_a, and a better fit could have been obtained by altering the distributions of these parameters. However, the discrepancy should not lead to serious inaccuracy in group deflection response predictions until relatively high load levels are reached.

The second step was to backfigure the average lateral soil modulus, E_{sh}, and limiting lateral pile-soil pressure p_y by a similar process of trial and error, using the program PULL 2B to generate the theoretical curves. Figs.11b and c show the fit obtained between the theory and the four measured curves, using constant values of E_{sh} of 35 MPa and p_y of 0.2 MPa. In the light of the variability of the measured results, the fit obtained is quite reasonable. It is interesting to note that the backfigured modulus value for lateral loading, E_{sh}, is one-half of the value for axial loading, E_{sv}. This difference is probably attributable largely to the effects of pile-soil separation at the back of the laterally loaded pile.

The first step in the prediction procedure was to use the backfigured parameters from the single pile tests to predict the entire lateral load versus deflection behaviour of the pile groups, using the program DEFPIG. This necessitated the estimation of the lateral efficiency factor η_L by which the value of p_y was reduced to allow fro group effects. Based on a small amount of previous data and experience, values of 0.85, 0.6, 0.9 and 0.7 were chosen for Groups A, C, D and E respectively.

Fig.12 shows the predicted and measured load-deflection curves for the above four groups. The agreement for the two groups with vertical piles (Groups A and C) is very good over the entire load range. For the groups containing battered piles (Groups D and E), the agreement is less satisfactory; the theory overpredicts deflections at lower load levels and underpredicts at higher load levels. Nevertheless, the agreement is not unreasonable, bearing in mind the possibility of some variation in soil conditions from Zone II, where these groups are located, to Zone I, where the single test piles were located.

In summary, this case suggests that the philosophy of using the single pile tests to evaluate the pile-soil parameters and then using the theory to predict group response, is basically sound.

8 COVENTRY POINT OFFICE BLOCK (Cole and Stroud, 1977)

Cole and Stroud (1977) have described some aspects of the foundation design of a medium rise office development, Coventry Point, in the city of Coventry, U.K. The development consists of two inter-linked office blocks of fifteen storeys (Block A) and sixteen storeys (Block B) which have been supported on groups of rock-socketted piles. The subsoil profile consists of up to 5 m of fill and firm silty sandy clay overlying siltstones and sandstones of varying strength, interbedded with extensively-weathered

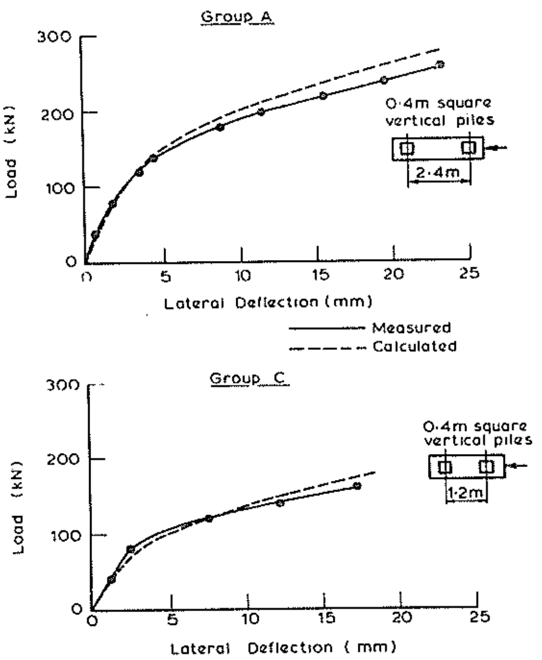


Figure 12 Comparisons between measured and theoretical group behaviour

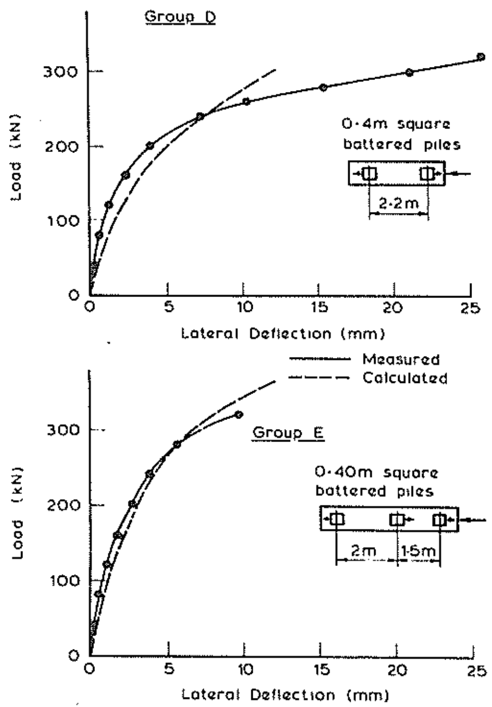


Figure 12cont'd. Comparisons between measured and theoretical group behaviour

bands of mudstone (Fig.13a). Typically, SPT values in the upper layer ranged between 10 and 50, whereas below 5 m, values between 100 and 400 (with occasional larger values) were experienced. To aid the foundation design, a contract pile, 1.06 m diameter and approximately 8.5 m long, was test loaded. This pile settled 12 mm under the design load of 4.5 kN.

In order to assess the applicability of pile settlement theory to this case, the above test pile data was interpreted to obtain modulus values for the

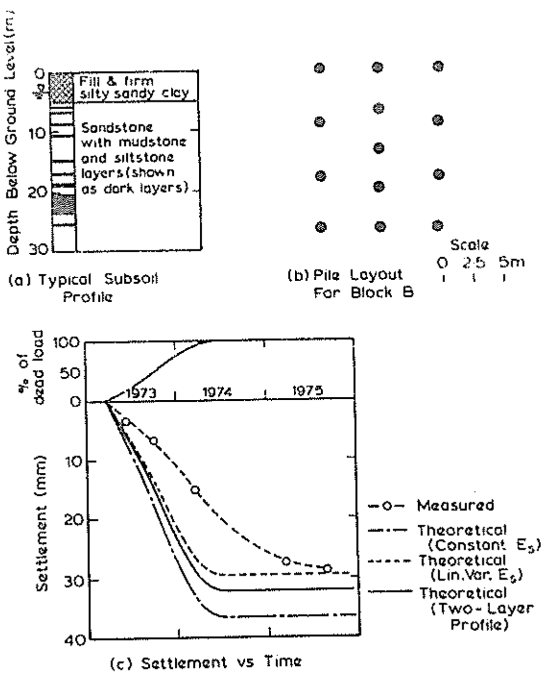


Figure 13 Comparison between observed and theoretical settlement - Coventry Point Block B (Cole & Stroud, 1977)

subsoil profile, and a group settlement analysis was then performed for Block B. Three different assumptions were used in interpreting the test pile results: (a) a constant modulus E_s with depth, (b) a linearly increasing modulus with depth, (c) a two-layer profile, with the upper 5 m having a modulus one-fifth of the value for the underlying material (this assumptions was based on the SPT results).

In each case, the program TAPILE was used to develop relationships between the pile head settlement and modulus, from which the modulus to give a 12 mm settlement was determined. The following values were obtained for the three assumptions listed:

(a) E_s = 51.3 MPa
(b) E_s at 8.5 m depth = 78.1 MPa
(c) E_s (0-5 m) = 12.6 MPa
E_s (5 m^+) = 63 MPa.

The program DEFPIG was then used to compute the settlement of Block B, which was founded primarily on 13 piles, 1.22 m diameter, and penetrating to 8 m below the ground surface (Fig.13b). A rigid pile cap was assumed, and based on the available information, a dead load of 65.5 MN was assumed to act on the foundation. Table 1 compares the three predicted settlements with the measured settlement approximately 18 months after completion of construction. Good agreement is found with the values from assumptions (b) and (c). The settlement predicted on the assumption (a) of a constant modulus is too large, as a result of the larger settlement interaction factors which occur in this case (Fig.14).

A closer examination of the rate of settlement during and after construction reveals that the actual settlement developed more slowly than given by the theory if the settlements are assumed to occur instantaneously upon application of the load (Fig. 13c). This slower development of settlement and the continued increase after completion of construction are probably a consequence of consolidation of the interbedded mudstone and siltstone layer beneath the pile tips.

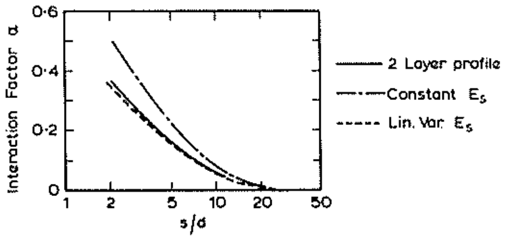


Figure 14 Comparison between theoretical interaction factors from various assumptions

Two interesting points thus emerge from this case study: (a) the theory appears to provide a useful link between the settlement of a single pile and a pile group, provided that a judicious assumption is made regarding the distribution of modulus with depth, (b) the time-dependence of settlement is more pronounced with pile groups than with single piles, as predicted theoretically by Poulos (1968). However, practical solutions for the rate of settlement of pile groups have not yet been published; moreover, in this case, the interbedded nature of the soil profile would make accurate prediction of time-settlement behaviour difficult.

TABLE 1

COMPARISON BETWEEN MEASURED AND THEORETICAL SETTLEMENTS. COVENTRY CENTRE - BLOCK B

Method	Settlement mm
Calculated for Constant E_s	36.5
Calculated for Linearly Increasing E_s	29.3
Calculated for 2-Layer Profile E_s (0-5m) = 1/5 E_s ($5m^+$)	32.0
Measured, 1½ years After Completion of Construction	28.6

9 CONCLUSIONS

The case histories examined in this paper have involved single piles and pile groups subjected to axial or lateral loading, and a common theoretical approach has been applied to the prediction of the load-deflection behaviour in all cases. The requisite soil parameters have been estimated in a variety of ways, including empirical correlations based on previous experience, the interpretation of load test results, and the application of pressuremeter data. The agreement between the measured and theoretical behaviour has been found to be generally satisfactory, even for the two "Class A" predictions where the soil parameters were based on empirical correlations. For the pile groups, successful predictions of load-deflection behaviour have been made by using soil parameters backfigured from single pile load tests.

However, the predictions for battered pile groups appear to be less satisfactory than for groups of vertical piles, and there may be some scope for improving the theory for battered pile groups. In addition, it is desirable, when predicting group behaviour, to assume a reasonable distribution of soil modulus with depth, as the interaction factors may be overestimated if a homogeneous soil mass is assumed.

Provided that an appropriate measure of engineering judgment is employed, the elastic-based theory appears to provide a reasonable practical basis for predicting the performance of pile foundations subjected to axial or lateral loading.

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11 REFERENCES

BALAAM, N.P., POULOS, H.G. and BOOKER, J.R. (1975). "Finite element analysis of the effects of installation on pile load-settlement behaviour". Geot.Eng. Vol.6, No.1, pp.33-48.

BANERJEE, P.K. (1978). "Analysis of axially and laterally loaded pile groups". Ch.9 of Developments in Soil Mechanics, Ed. C.R. Scott, Appl. Science, U.K.

BANERJEE, P.K. and DAVIES, T.G. (1979). "Analysis of some reported case histories of laterally loaded pile groups". Proc. Conf. on Num. Methods in Offshore Piling, I.C.E. London, pp.83-90.

BUTTERFIELD, R. and BANERJEE, P.K. (1971). "Elastic analysis of compressible piles and pile groups". Geotechnique, Vol.21, No.1, pp.43-60.

BUTTERFIELD, R. and GHOSH, N. (1979). "The use of a numerical analysis to correlate soil shear moduli determined from model tests on single piles and groups of piles in clay". Proc. Conf. on Num. Methods in Offshore Piling, I.C.E. London.

COLE, K.W. and STROUD, M.A. (1977). "Rock socket piles at Coventry Point, Market Way, Coventry". Piles in Weak Rock, I.C.E., London, pp.47-62.

DESAI, C.S. (1974). "Numerical design-analysis for piles in sands". Jnl. Geot. Eng. Divn. ASCE, Vol. 100, No.GT6, pp.613-635.

ELLISON, R.D., D'APPOLONIA, E. and THIERS, G.R. (1971). "Load-deformation mechanism for bored piles". Jnl. Soil Mechs. Fndns Divn, ASCE, Vol.97, No.SM4, pp.661-678.

FRYDMAN, S., SHA'AL, B. and MAZURIK, A. (1975). "Analysis of an instrumented laterally loaded pile". Proc. 5th Asian Reg. Conf. Soil Mechs. Fndn. Eng., Bangalore.

LAMBE, T.W. (1973). "Predictions in soil engineering". Geotechnique, Vol.23, pp.151-202.

MANOLIU, I., BOTEA, E. and CONSTANTINESCU, A. (1977). "Behaviour of pile foundations submitted to lateral loads". Proc. 9th Int. Conf. Soil Mechs. Foundn. Eng., Tokyo, Vol.1, pp.637-640.

MATTES, N.S. and POULOS, H.G. (1969). "Settlement of single compressible pile". Jnl. Soil Mechs. Divn., ASCE, Vol.95, No.SM1, pp.189-207.

POULOS, H.G. (1968). "Analysis of the settlement of pile groups". Geotechnique, Vol.18, No.4, pp.449-471.

POULOS, H.G. (1971). "Behaviour of laterally loaded piles: I - single piles". Jnl. Soil Mechs. Fndns. Divn., ASCE, Vol.97, No.SM5, pp.711-731.

POULOS, H.G. (1972). "Load-settlement prediction for piles and pier". Jnl. Soil Mechs. Fndns. Divn., ASCE, Vol.98, No.SM9, pp.879-897.

POULOS, H.G. (1974). "Some recent development in the theoretical analysis of pile behaviour". Ch.7 of Soil Mechanics - New Horizons, Ed. I.K. Lee, Newnes-Butterworths, London, pp.237-279.

POULOS, H.G. (1977). "Settlement of pile foundations". Ch.10 of Numerical Methods in Geotechnical Engineering, Ed. C.S. Desai and J.T. Christian, McGraw-Hill, New York.

POULOS, H.G. (1979a). "Settlement of single piles in non-homogeneous soil". Jnl. Geot. Eng. Divn., ASCE, Vol.105, No.GT5, pp.627-641.

POULOS, H.G. (1979b). "An approach for the analysis of offshore pile groups". Proc. Conf. on Num. Methods in Offshore Piling, I.C.E., London, pp.91-98.

POULOS, H.G. and DAVIS, E.H. (1968). "The settlement behaviour of single axially loaded incompressible piles and piers". Geotechnique, Vol.18, No.3, pp.351-371.

POULOS, H.G. and DAVIS, E.H. (1980). Pile foundation analysis and design. John Wiley and Sons, New York.

RANDOLPH, M.F. and WROTH, C.P. (1978). "Analysis of deformation of vertically loaded piles". Jnl. Geot. Eng. Divn., ASCE, Vol.104, No.GT12, pp.1465-1488.

SAA (1978). "SAA piling code AS2159-1978". Standards Association of Australia.

SULLIVAN, R.A. (1973). "Load tests of pipe and step-taper piles". Paper presented to ASCE, Texas Section Spring Meeting, Austin, Texas, April 5-7, 1973.

VALLIAPPAN, S., LEE, I.K. and BOONLUALOHR, P. (1974). "Settlement analysis of pile in layered soil". Proc. 7th Bien. Conf. Aust. Road Res. Bd., Adelaide, Vol.7, Pt.7, pp.144-153.