Lee, C. J., Bolton, M. D. & Al-Tabbaa, A. (2002). *Géotechnique* **52**, No. 5, 325–335

Numerical modelling of group effects on the distribution of dragloads in pile foundations

C. J. LEE*, M. D. BOLTON† and A. AL-TABBAA‡

Negative skin friction on pile foundations, predicted from the results of numerical analyses, is presented. Soil slip at the pile–soil interface has been found to be the most important factor in governing pile behaviour in consolidating ground. Reduction in dragload is predicted for piles in a group owing to interaction between soil and pile. It has been demonstrated that the group effect depends not only on the configuration of the pile group, but also on soil slip along the pile–soil interface, governed mainly by the interface friction coefficient and the soil settlement. Various factors should be included in an evaluation of the group effect, including the pile spacing, the number of piles in a group, the relative location of piles in a group, the pile type, the pile installation method, the surface loading and the stiffness of the soil. Existing design approaches result in overprediction of dragload for a single pile and of group effect for a pile group. Back-analysed dragloads and group effects considering soil slip are compared with a number of case histories.

KEYWORDS: case history; numerical modelling and analysis; piles; soil/structure interaction

Nous étudions le frottement superficiel négatif sur les fondations de piles, frottement calculé d'après les résultats d'analyses numériques. Nous avons trouvé que le glissement de sol à l'interface pile-sol était le facteur qui avait le plus d'influence sur le comportement des piles dans le sol de consolidation. Nous calculons la réduction de la charge de traînée pour les piles d'un groupe, réduction due à l'interaction entre sol et pile. Il a été démontré que l'effet de groupe dépendait non seulement de la configuration du groupe de piles mais aussi du glissement de sol le long de l'interface pile-sol, glissement principalement gouverné par le coefficient de friction d'interface et le tassement du sol. Plusieurs facteurs, comme l'espacement des piles, le nombre de piles dans un groupe, l'emplacement relatif des piles dans un groupe, le type de piles, leur méthode d'installation, la charge surfacique et la rigidité du sol devront être pris en compte dans l'évaluation de l'effet de groupe. En appliquant les méthodes d'étude existantes, nous obtenons des surévaluations de la charge de traînée dans le cas d'une pile seule et de l'effet de groupe. Nous comparons avec plusieurs histoires de cas les charges de traînée et les effets de groupe analysés rétroactivement en tenant compte du glissement de sol.

INTRODUCTION

The evaluation of negative skin friction (NSF), which occurs when the soil next to a pile settles more than the pile, is a common problem in the design and construction of pile foundations in soft ground. Various causes have been reported, which are related mainly to the increase in effective vertical stress in soil (e.g. Phamvan, 1989; Little, 1994; Lee *et al.*, 1998). The development of additional compressive force (dragload) in a pile and excessive pile settlement (downdrag) could cause difficulty in construction and maintenance of the structure supported.

Although the basic mechanism of and solution to the NSF problem are well established, there is some debate among engineers regarding various crucial aspects of the NSF phenomenon, which sometimes leads to confusion and misunderstanding (Lee, 2001). Hence in this paper the basic terms relating to the NSF problem are adopted according to the recent definitions proposed by Fellenius (1999):

(*a*) negative skin friction (NSF): soil resistance acting downwards along the pile shaft as a result of a downdrag and inducing compression in the pile
(*b*) downdrag: the downward movement of a deep foundation unit due to NSF and expressed in terms of settlement
(*c*) dragload: the load transferred to a deep foundation unit from NSF
(*d*) neutral plane (NP): the point at which equilibrium exists between the sum of the downward-acting permanent loads applied to the pile and dragload due to negative skin friction, and the sum of upward-acting positive shaft resistance (PSR) and mobilised toe resistance. The neutral plane is also the point at which the relative movement between the pile and the soil is zero.

Failure of foundations in terms of serviceability criteria due to downdrag is not uncommon in practice (Davisson, 1993). However, to date most of the current design approaches have been based on simplified methods and are not satisfactory. Dragload predictions presented by various researchers at the Wroth Memorial Symposium varied within a range of 98–515% of the measured value (Little & Ibrahim, 1993). It is also known that less dragload develops on piles within a group, owing to pile–soil interaction. Exaggerated dragloads in groups are predicted using current design methods, especially for central piles (Terzaghi & Peck, 1948; Combarieu, 1985; Briaud *et al.*, 1991; Jeong, 1992). However, lesser group effects have been reported from previous experimental measurements documented in the literature (Koerner & Mukhopadhyay, 1972; Denman *et al.*, 1978; Shibata *et al.*, 1982; Little, 1994; Thomas, 1998), although reliable measurements are rather limited.

Contrary to common design approaches, in which elastic analysis is thought to present approximate estimations of pile behaviour, soil slip is very likely to develop at the pile–soil interface, owing to large soil movements for piles in

Manuscript received 25 May 2001; revised manuscript accepted 15 March 2002.
Discussion on this paper closes 1 December 2002, for further details see p. ii.
* Visiting Scholar, Geotechnical Centrifuge Facility, The Hong Kong University of Science and Technology (formerly Cambridge University).
† Professor, Cambridge University, Engineering department, UK.
‡ Lecturer, Cambridge University, Engineering department, UK.

consolidating ground. Nishi & Esashi (1982) and Phamvan (1989) have pointed out the significance of the consideration of soil slip at the interface for single piles. Kuwabara & Poulos (1989), Chow *et al.* (1990), Teh & Wong (1995) and Chow *et al.* (1996) have also reported that the effect of soil slip at the pile–soil interface is a key factor affecting pile group behaviour. However, to date, the effect of soil slip in pile groups is not well understood. Although the behaviour of piles in a group is obviously a three-dimensional (3D) problem, there is only one reported 3D finite element analysis (FEA) of the behaviour of a pile group in the literature by Jeong (1992), neglecting soil slip at the pile–soil interface, which reported very large group effects. In this paper the results of two-dimensional (2D) and 3D FEA, incorporating the effects of soil slip, are presented. The results are compared with elastic solutions. Parametric studies are presented examining the major factors affecting soil slip behaviour and soil–group interaction. Finally the results from FEA are compared with field observations and an example case history.

FINITE ELEMENT MODELLING

Numerical analyses have been conducted examining single piles in 2D axisymmetrical conditions and pile groups in 3D conditions respectively. Although a 3D problem can sometimes be simplified to a 2D model, it is unrealistic for this research because the effects of the relative location of piles within a group cannot be modelled. The finite element package ABAQUS is used in the simulation of pile behaviour described in this paper.

Figures 1(a) and 1(b) present typical finite element (FE)

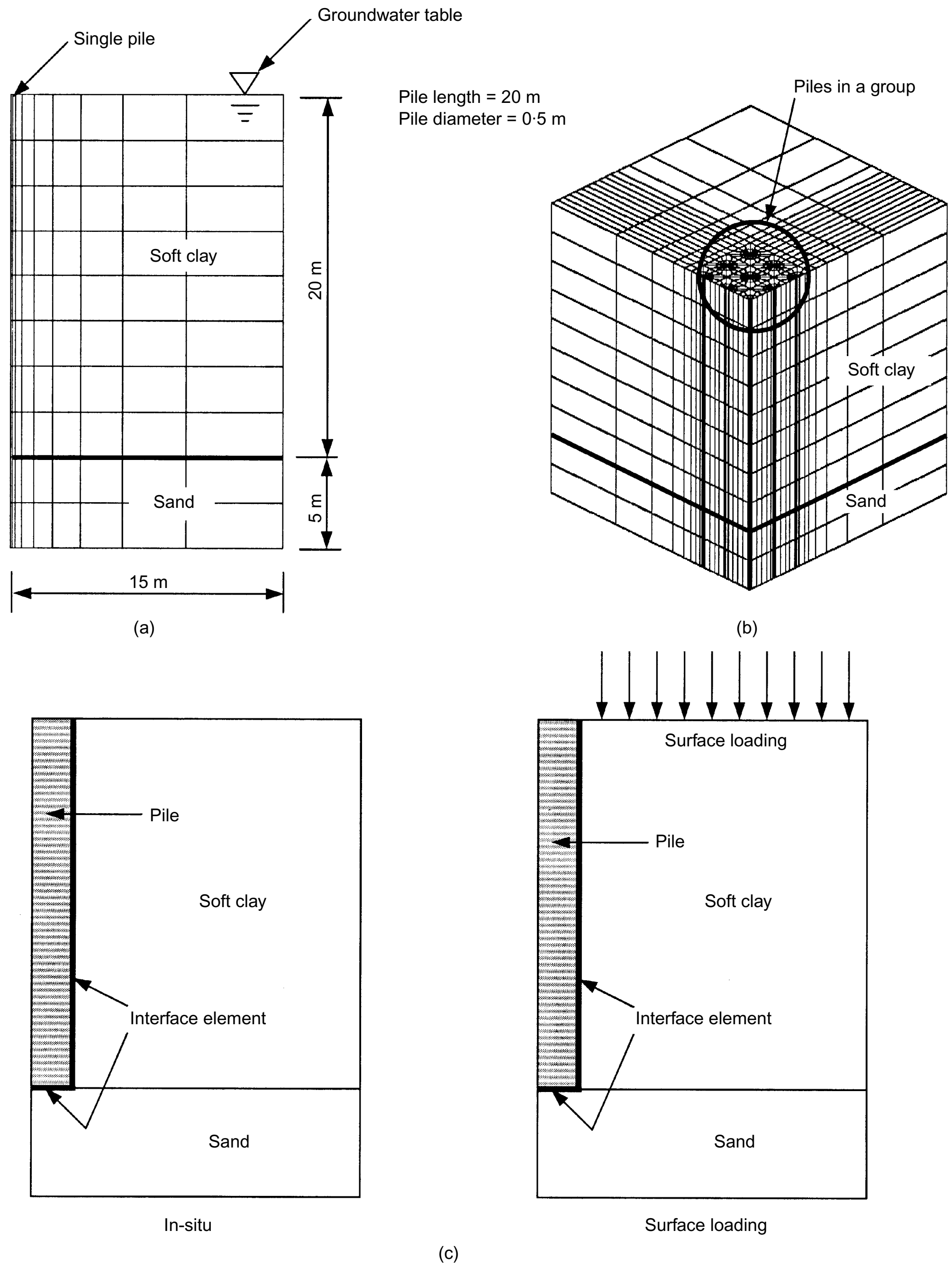

Fig. 1. Typical FE meshes for (a) 2D (axisymmetric) and (b) 3D (5×5 piles in 2·5D) analyses; (c) analysis sequences

meshes used in this analysis. Owing to symmetry, only a quarter of a whole mesh is used in the 3D simulation (Fig. 1(b)). A relatively fine mesh is used near the pile–soil interface, and it becomes coarser further from the pile. Various sensitivity studies have been performed in order to design the most appropriate FE mesh for 3D analyses. The piles are taken to be 0·5 m in diameter and 20 m long, and are not connected with each other. The piles are assumed to have been installed through soft clay with the pile tip located at the interface between the soft clay and the bearing sand layer. Changes in the in-situ stress in the soil and changes in soil stiffness resulting from pile installation are ignored. Therefore, before initiating each analysis, the piles are free of residual stress. Soil settlement, and hence the development of NSF, is initiated by the application of surface loading on the top of the clay surface, as demonstrated in Fig. 1(c). In the 3D analyses 3×3 and 5×5 pile groups have been considered with a typical minimum and maximum pile spacing, S, of $2{\cdot}5D$ and $5{\cdot}0D$ (where D is the pile diameter); surface loading, Δp, of 25–200 kPa; soil modulus, E, of 2–20 MPa; and interface friction coefficient, μ, (where $\mu = \tan\delta$) of 0·2–0·5. Eight-node second-order quadrilateral elements and 27-node second-order brick elements are used for the 2D and 3D analyses respectively.

Table 1 summarises the typical material properties used in this analysis. Unless otherwise stated, these material properties have been used throughout. Since the consolidation analysis requires more calculation time and memory space, only drained analyses have been conducted. An elastic model is used for the pile and a non-associated Mohr–Coulomb model for the clay and sand. For clay the internal angle of friction, ϕ, is set at 20°, typical of a critical-state angle with a very small dilation angle ψ, since large shear deformation developed at the interface. For sand the peak internal angle of friction is set at 45° with a dilation angle of 10°, consistent with the critical-state angle being about 35°. Because of space limitations in this paper, for detailed information on the numerical analyses and adopted input parameters readers should refer to Lee (2001).

The ABAQUS interface modelling technique is used to simulate slip at the soil–pile interface (ABAQUS, 1998). Duplicated nodes are used to form an interface of zero thickness. ABAQUS uses the Coulomb frictional law in which frictional behaviour is specified by an interface friction coefficient, μ, and a limiting displacement, γ_{crit}. A limiting displacement of 5 mm was assumed to achieve full mobilisation of skin friction, typical of field measurements reported by Broms (1979) (i.e. 1–8 mm). The normal effective stress, σ', between two contact surfaces was multiplied by the interface friction coefficient, μ, to give a limiting frictional shear stress, $\mu\sigma'$. If the shear stress applied along the surfaces was less than $\mu\sigma'$ the surfaces would stick. The nodes of the soil elements in contact with a pile could slide along it when soil slip occurs. It has been found that pile behaviour is governed mainly by interface behaviour. Fig. 2 presents the relationship of shear stress, displacement and soil slip.

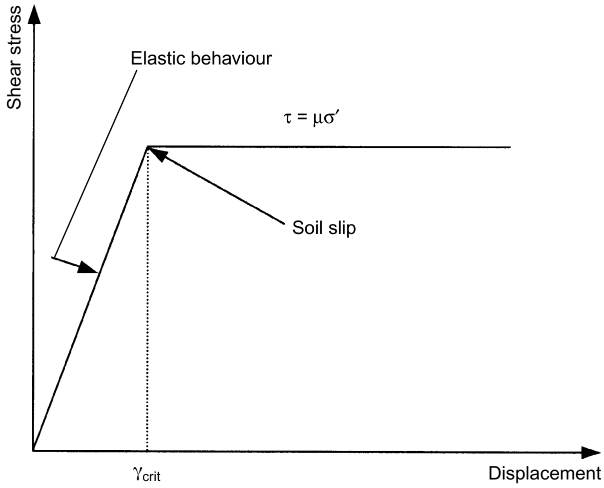

Fig. 2. Behaviour of interface element

BEHAVIOUR OF A SINGLE PILE

Elastic solution

Figure 3 shows distributions of the normalised dragload, $P_a/E_{clay}S_oL$ (where P_a is maximum dragload), with respect

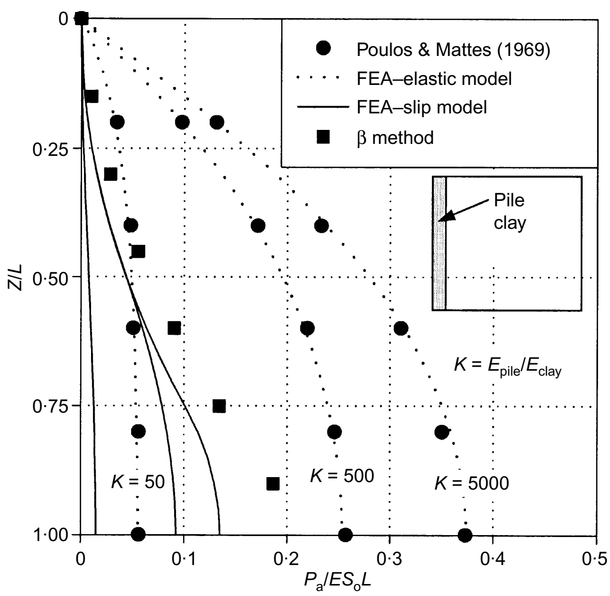

Fig. 3. Distribution of dragload as determined by various methods. $E_{pile} = 2$ GPa, $\Delta p = 25$ kPa, $\mu = 0{\cdot}3$, $\beta = 0{\cdot}3$

Table 1. Material properties used in the analysis

Material	Model	E: kN/m^2	c: kN/m^2	ν	ϕ_c: °	ψ: °	K_o	γ_t: kN/m^3
Concrete pile	Isotropic elastic	2 000 000		0·3			1·0	25
Soft clay	Mohr–	5000	3	0·3	20	0·1	0·65	18
Bearing sand	Coulomb	50 000	0·1	0·3	35	10	0·5	20

Notes:
1. Groundwater table is located on the top of the soft clay layer.
2. Hydrostatic water pressure distribution is assumed.

to the Young's modulus for clay, E_{clay}, the surface settlement, S_o, and pile length, L, based on formal elastic solutions for an end-bearing pile by Poulos & Mattes (1969). It also shows FEA results based both on a no-slip elastic model with interface friction coefficient $\mu = \infty$ and on an elasto-plastic slip model based on initially isotropic stress conditions (i.e. $K_o = 1$). The no-slip elastic FEA simulations agree closely with the elastic solutions, although both methods overestimate dragload. When soil slip is taken into account, however, a smaller dragload is predicted, such that the interface shear stress exceeds neither the clay nor the interface shear strength. Since the slip analysis ($\mu = 0{\cdot}3$) requires 5 mm of relative displacement prior to sliding, the slip analysis now also shows a reduced dragload in stiff soil ($K = 50$) due to partial shear mobilisation. The correction of these two effects, one numerical and one physical in origin, can be seen in Fig. 3 to produce a very significant effect on the dragload, P_a. Fig. 3 also shows dragload predicted from the β-method (Burland, 1973), assuming a β value of 0·3, which produces a single curve regardless of the value of K. This method would result in shear stress being somewhat overestimated, since the maximum shear stress is assumed to act along the entire length of the pile and hence partial mobilisation of skin friction near the pile toe could not be included. In addition, the actual vertical effective stress at the interface would be smaller than expected by the β method, owing to the transfer of some of the soil weight to the pile (Zeevaert, 1983; Jeong, 1992; Bustamante, 1999). In summary, both the β method and the elastic solutions predict excessively large dragloads, as would be expected.

Dragload and downdrag

Figures 4(a) and 4(b) show different distributions of typical shear stress (Fig. 4(a)) and dragload (Fig. 4(b)) for a friction pile and an end-bearing pile. In this analysis different stiffnesses ($E_{bearing\ layer}$) are used to represent the bearing layer (noted as a sand layer in Fig. 1) to model a friction pile or an end-bearing pile. The different stiffness ratios ($E_{bearing\ layer}/E_{soft\ clay\ layer}$) of 1 and 1000 are used for a friction pile and end-bearing pile respectively. For a friction pile, NSF is developed along a section extending from the top of the pile to the neutral plane, which is located at around 70% of the pile length measured from the surface. For an end-bearing pile, NSF is developed along the entire length of the pile. This can be explained by the relative settlement between the soil and the pile. When a friction pile settles more than the soil settles, positive shaft resistance (PSR) will be developed along the lower part of the pile shaft. However, only very small pile movement is possible for end-bearing piles. NSF is therefore developed along the entire length of the pile. Partial mobilisation of negative skin friction and positive shaft resistance are observed near the neutral plane, where the shear stress is less than the limiting shear stress, $\mu\sigma'$, owing to small relative movement between the soil and the pile. Slip lengths of about 63% and 93% of the total pile length are observed for friction and end-bearing piles respectively (see Fig. 4(a)).

More dragload (596 kN) and less downdrag (1·5 mm) are developed for an end-bearing pile, and less dragload (350 kN) and more downdrag (79·0 mm) are developed for a friction pile. It has been shown that, in most cases, dragload is not significant for piles shorter than 30 m (UniNews, 1998). However, there are still many cases where failure of structures is reported as a result of excessive pile settlement (downdrag) (Acar *et al.*, 1994). Excessive pile settlement is very probably developed for friction piles: therefore such piles should be installed deep into a stiff layer where possible. The structural integrity and driveability of the pile and piling system should therefore be thoroughly investigated. Where it is not possible to found the pile in a competent layer an alternative engineering solution should be considered.

Effect of soil slip on dragload

The development of the slip length along a pile shaft is dependent on the interface friction coefficient, surface loading and soil modulus, as shown in Fig. 5(a). When the interface friction coefficient and soil modulus are small, or surface loading is large, the slip length is increased, and vice versa. Fig. 5(b) shows the development of dragload considering the effect of interface friction coefficient, soil modulus and surface loading. The dragload is roughly proportional to the interface friction coefficient at all surface loading values. However, the effect of soil stiffness is insignificant, except for a small surface loading on stiff ground, since slip can develop under very small relative movements. The effect of the slip length on the group effect will be shown later.

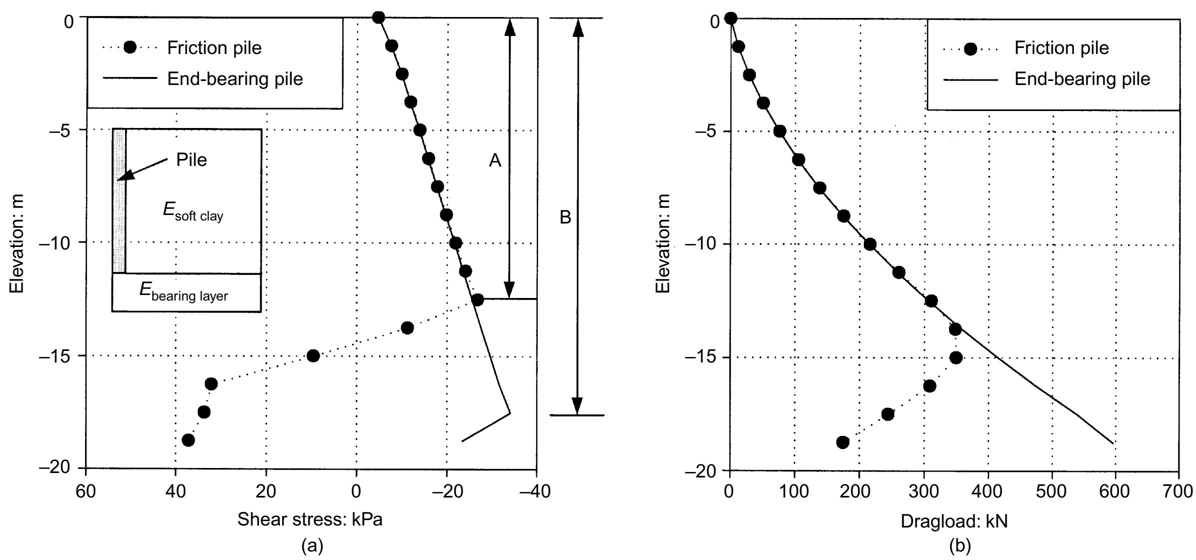

Fig. 4. (a) Shear stress and (b) dragload distributions. $E_{soft\ clay} = 5$ MPa, $\mu = 0{\cdot}3$, $\Delta p = 50$ kPa. $E_{bearing\ layer} = 5$ MPa (friction pile), 5 GPa (end-bearing pile). A, slip length for a friction pile; B, slip length for an end-bearing pile

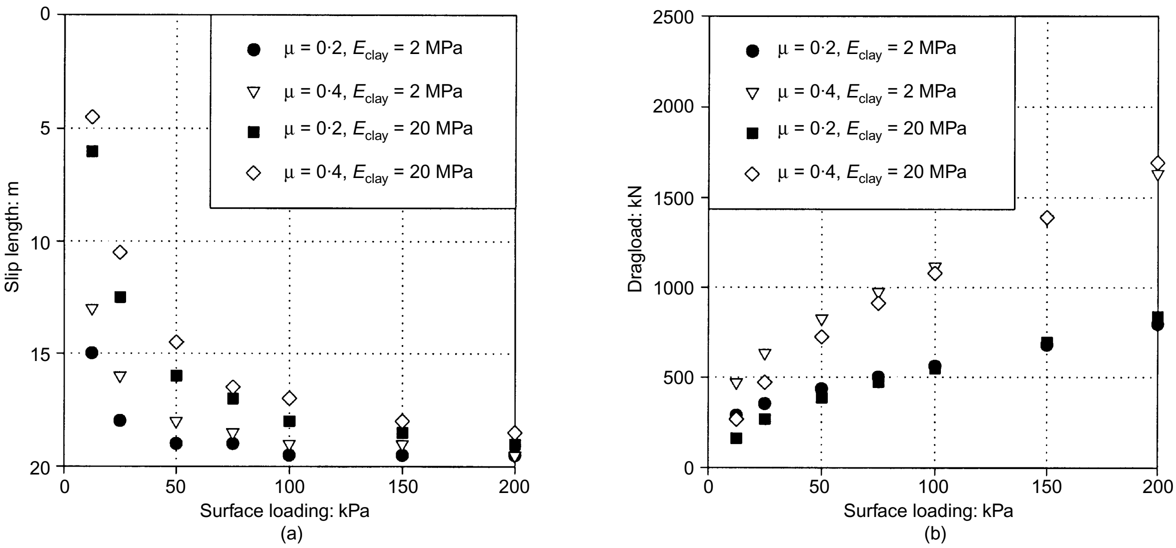

Fig. 5. Effect of soil slip at the interface: (a) slip length; (b) dragload

BEHAVIOUR OF PILES IN A GROUP

Elastic solution

Figure 6 presents the development of dragload for end-bearing piles in a 5×5 pile group with a pile spacing of $5D$ as determined from an FEA using a no-slip elastic model. It also shows an elastic solution presented by Kuwabara & Poulos (1989). The positions of the piles referred to in Fig. 6 are shown in Fig. 7(a). The piles are modelled as free-headed piles in a group, which can be regarded as similar to piles in a flexible pile cap. No constraint is provided at the pile heads, and no axial load is applied to them. It is assumed that piles are not connected to each other, so that each pile can respond separately in response to NSF and in accordance with its magnitude.

The trends for dragload distribution are very similar from both analyses, although the solution from Kuwabara & Poulos (1989) predicts smaller dragloads, particularly at the pile toes, a discrepancy also reported by Chow *et al.* (1996). These small dragloads, which amount to only 10–50% of that predicted for a single pile, are not supported by previous observations with similar pile configurations (Denman *et al.*, 1978). Fig. 6 also presents the results from an elasto-plastic analysis, which included soil slip ($\mu = 0{\cdot}3$). Relatively small reductions in the dragload (i.e. 40–70% compared with a single pile) are observed.

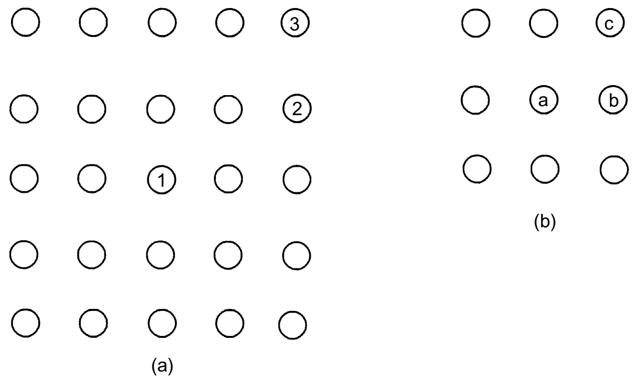

Fig. 7. Position of piles in a group: (a) 5×5; (b) 3×3 (referred to in Figs 6 and 8 and Table 2)

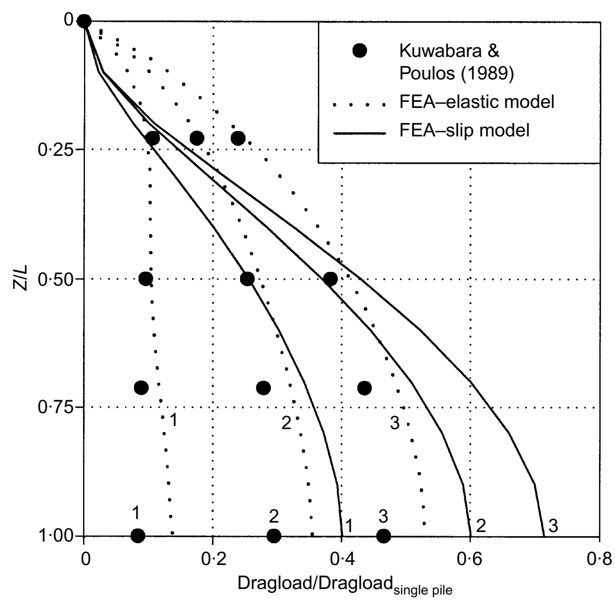

Fig. 6. Comparison of dragload with and without slip. $S = 5{\cdot}0D$, $E_{clay} = 20$ MPa, $\Delta p = 25$ kPa, $\mu = 0{\cdot}3$. See Fig. 7(a) for position of the piles

Figure 8(a) shows the distributions of shear stress in piles in a 5×5 group with a $2{\cdot}5D$ pile spacing. When soil slip develops, similar shear stresses are generated along the upper part of the piles, but shear stresses then reduce significantly with depth. There is less dragload on the piles in a pile group, as shown in Fig. 8(b), because the soil between the piles suffers less influence from the surface loading. The vertical effective stress in these soil pockets is comparatively decreased, so the soil does not consolidate as much as it would have done if there had only been one pile adjacent to it. The soil trapped between the piles is therefore capable of generating less ultimate downward shear stress, owing to decreased effective stress levels, and also lower mobilised shear stresses, because there is less relative displacement between the soil and the piles. This phenomenon is more noticeable for the central pile, where the slip length is only 35% of that of an isolated pile. Similar observations,

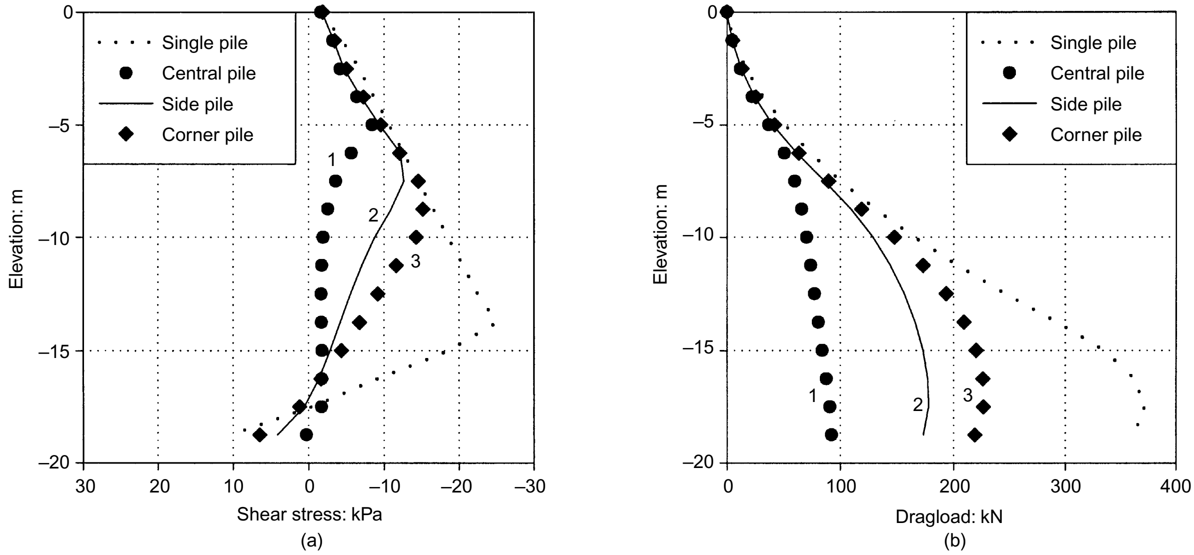

Fig. 8. Development of group effect (see Fig. 7(a)): (a) shear stress; (b) dragload. $S = 2{\cdot}5D$, $E_{clay} = 5$ MPa, $\mu = 0{\cdot}3$, $\Delta p = 25$ kPa

though attenuated, apply to external piles. Overall group effects of 76%, 52% and 39% are observed for the centre, side and corner piles respectively, where group effect is defined as the reduction in maximum dragload compared with an isolated pile carrying the same load per capita. If we had neglected soil slip at the soil and the pile interface, we would have artificially increased the bond between the soil and the pile, and we would have overestimated the group effect.

Consideration of various factors

Table 2 presents the influence of various parameters—interface friction coefficient, pile group configuration, relative position of piles in the group, pile spacing and surface loading—on the group effect. Group effects gradually reduce as surface loading is increased, because of an increase in slip length. Larger group effects are observed with an increase in the interface friction coefficient, which reduces soil slip. Larger group effects are observed for the 5 × 5 group

Table 2. Change of group effect (see Fig. 7 for position of the piles)

Group effect: %, $\Delta p = 50$ kPa

μ	Pile spacing	3 × 3			5 × 5		
		Position of piles					
		a	b	c	1	2	3
0·2	2·5*D*	17	12	9	50	29	20
0·3		30	18	14	66	40	29
0·4		41	25	19	74	49	36
0·5		48	27	19	79	54	40
0·2	5·0*D*	10	8	6	26	19	14
0·3		15	12	10	37	26	20
0·4		22	18	14	48	35	27
0·5		26	18	15	56	41	31

Group effect: %, $\mu = 0{\cdot}3$

Δp: kPa	Pile spacing	3 × 3			5 × 5		
		Position of piles					
		a	b	c	1	2	3
25	2·5*D*	48	31	23	75	52	39
50		30	18	14	66	40	29
100		16	11	10	55	31	25
150		11	8	6	49	25	21
200		9	6	5	45	23	19
25	5·0*D*	29	22	17	52	40	30
50		15	12	10	37	26	20
100		9	8	7	22	16	13
150		5	4	4	14	11	10
200		4	4	4	10	8	7

than for the 3×3 group. The maximum group effect is obtained on central piles, whereas the minimum group effect develops on corner piles. Relatively small group effects are observed for groups with a wider spacing (5·0*D*). The effect of the soil stiffness modulus on the group effect is found to be very small, since soil slip can develop at very small soil settlement, as discussed previously. An increase of 5–10% in the group effect is noted, with an increase from 2 to 20 MPa in the stiffness modulus of the soil. Therefore surface loading and interface friction coefficient are the most important factors governing the group effect.

In a situation where the interface friction coefficient amounts to 0·3 and a surface loading of 50 kPa is applied, group effects of 14–30% and 29–66% can be expected for the 3×3 and 5×5 pile groups respectively, each with an internal spacing of 2·5*D*. Smaller group effects will develop when larger pile spacing is considered. Table 3 summarises various factors influencing the group effect.

COMPARISONS BETWEEN MEASURED AND PREDICTED DRAGLOADS AND GROUP EFFECTS

Based on a number of case studies, some comparisons between measured and predicted dragloads for single piles and group effects for pile groups are presented below. Since detailed information regarding soil properties and stress histories is not provided in case studies 3 and 4, numerical simulations have been made based on the best assumption of appropriate soil properties, stress histories and boundary conditions. However, when modelling case studies 1 and 2, the Modified Cam Clay soil model has been used since all the required information is provided by the respective researchers from the experimental measurements and the Mohr–Coulomb soil model has been used in case studies 3 and 4.

Case1: Phamvan (1989)

Phamvan (1989) reported the development of dragload on a single pile due to embankment loading. After construction of a 2 m high embankment, a pile was driven through weathered crust, soft and medium stiff clay until the pile toe located on a bearing layer of stiff clay. Detailed soil properties and stress histories were measured, which enables an FE simulation of the problem to be performed (Phamvan, 1989). Fig. 9 shows the distribution of dragload as determined from

Table 3. Change of group effect due to various factors

1. Configuration of pile group		Group effects
Pile spacing	Close	More
	Wide	Less
Position of pile	Inside of group	More
	Outside of group	Less
Number of piles	Less	Less
	More	More
2. Interface friction		
Pile type	Concrete pile	More
	Steel pile	Less
Installation method	Driven pile	More
	Bored pile	Less
3. Soil settlement		
Surface loading	Small	More
	Large	Less
Stiffness of soil	Soft	Less
	Stiff	More

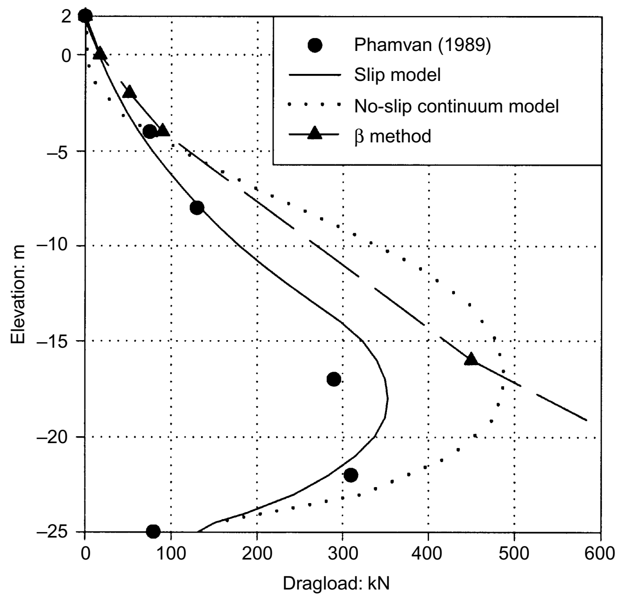

Fig. 9. Comparisons of the development of dragload

both field measurement and numerical analyses. The no-slip continuum analysis and the conventional β-method, taking an average β value of 0·2 as measured in the field, overestimate the dragloads. The β-method might have produced a better prediction if partial mobilisation of NSF and positive shaft resistance near the neutral plane had been considered. Good agreement with the field measurement is obtained when soil slip is taken into account in the FE simulation (slip analysis), although the position of the predicted neutral plane is slightly above its measured location.

Case 2: Lee et al. (1998)

Lee *et al.* (1998) presented measurements of dragload in a model pile from a single centrifuge test. The model pile had a diameter of 0·03 m and a length of 0·45 m. Complete data regarding the soil properties and stress histories of the soil and the boundary conditions during the test were presented. A large interface friction angle of 25·8° was measured from the experiment. However, since the pile was installed at 1*g* (where *g* is the centrifugal gravity), the pile behaviour would have been similar to that of a bored pile (Fioravante *et al.*, 1994). Therefore in this analysis friction coefficients of 0·3 and 0·4 are used to investigate a possible reduction in the interface friction coefficient. Two factors contribute to the development of NSF mechanism. First, a small amount of dragload developed owing to the increase of self-weight of the soil during the acceleration of the centrifuge test package from 1*g* to 50*g*. Then, after consolidation had taken placed, owing to dewatering, the effective vertical stress in the soil had increased by roughly 62 kPa.

Figure 10 presents the distributions of dragloads as determined from the centrifuge test and numerical analyses. A closer prediction is observed when a friction angle of 0·3 is adopted. Overall, a reasonable prediction of dragload is obtained from the slip model. Although the model pile is intended to have a fixed base, a pile movement of 1·5 mm has been measured during the centrifuge test. Therefore some positive shaft resistance is measured near the pile tip, where the settlement of the soil is smaller than that of the pile. Hence the maximum dragload is developed at 80–85% of the pile length measured from the top of the pile. In the numerical analysis the model pile is assumed to be an

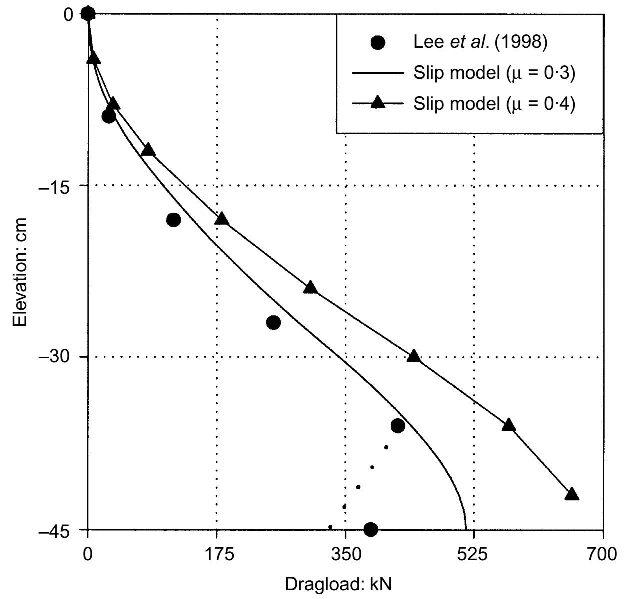

Fig. 10. Fitting the slip model to centrifuge data

end-bearing resting on a rigid base. Therefore, by allowing some pile movement (i.e. by modelling a fictitious soil layer below the pile tip), a better prediction of the distribution of dragload would be expected (shown in Fig. 10 by the dotted line).

Case 3: Okabe (1977)

Okabe (1977) reported the results from full-scale field measurements of dragload in a pile group, resulting from a combination of dewatering and surcharge loading. The pile group consisted of 38 piles spaced at $2{\cdot}1D$ (see Fig. 11). There were 14 external piles for protection, which were to take most of the negative skin friction and were free to move, and 24 internal end-bearing piles, which were connected to a rigid pile cap. This observation is found in the literature reporting the largest group effects for piles in a pile group. A group effect of roughly 85–88% was reported for the inside piles. Almost no skin friction developed along the length of the piles with little compressive force on the pile head. A 51% group effect was also measured for the external protection piles. A tensile force of approximately 605 kN was measured at the pile head of the protection piles. However, since the protection piles were free to move, little or no tensile force should have been developed. The reliability of this measurement is therefore questionable. This has also been discussed by Teh & Wong (1995).

Since information regarding the soil properties, loading sequence, exact pile configuration, structural load on the footing and interaction between the top soil and the footing is not available, some simplifications have to be made for the purpose of FEA. In this analysis the pile cap is not in contact with the soil surface, for the purpose of simplicity in the FE simulation and for comparison with a previous theoretical study by Kuwabara & Poulos (1989). Therefore the pile cap is above the soil surface at the beginning of the analysis. No external load is applied on the pile cap or pile heads. In this analysis the pile group is modelled as a 6×6 group with a pile spacing of $2{\cdot}1D$ (see Fig. 12). Therefore only the most important features of the real situation could be approximately simulated. The heads of the inner pile (i) and the central pile (c) are connected to a rigid footing, while the outer protection piles are vertically separated from the footing, as reported by Okabe (1977). For predictions, the soil properties and surface loading are estimated by fitting the measured dragload distribution in a single pile to that computed by the FEA. Therefore a soil stiffness modulus, E, of 10 MPa, an interface friction coefficient, μ, of 0·425 and a soil surface loading of 250 kPa, resulting from the combined effects of an increase in effective vertical stress in the soil due to dewatering and embankment loading, are assumed. 3D analyses are carried out to model rigid pile caps properly.

The dragload of piles in a group from the field measurement and the numerical analysis is presented in Fig. 13. A certain amount of tension develops at the pile head for the inner piles (i), and compressive force is observed along the central piles (c), since the outer piles try to move more than the inner piles as shown in Fig. 13 (see Figs 11 and 12 for the positions of piles). Group effects varying from 44% to 72% are predicted. The maximum dragload is predicted for the protection piles, whereas the central pile has the least dragload. The prediction of group effect for the protection piles (44%) is very similar to the values reported by Okabe

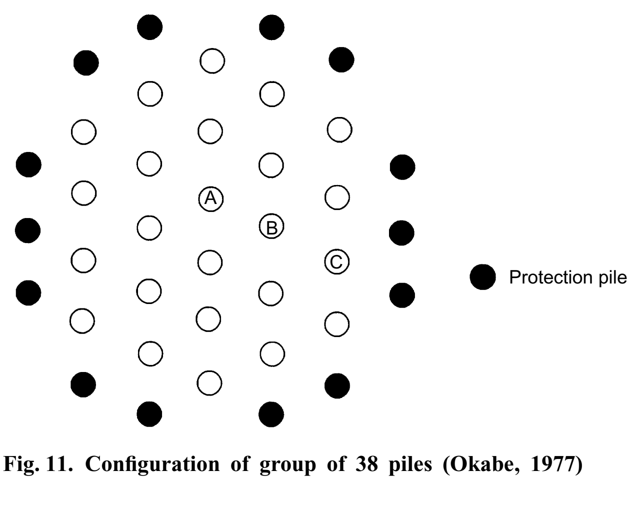

Fig. 11. Configuration of group of 38 piles (Okabe, 1977)

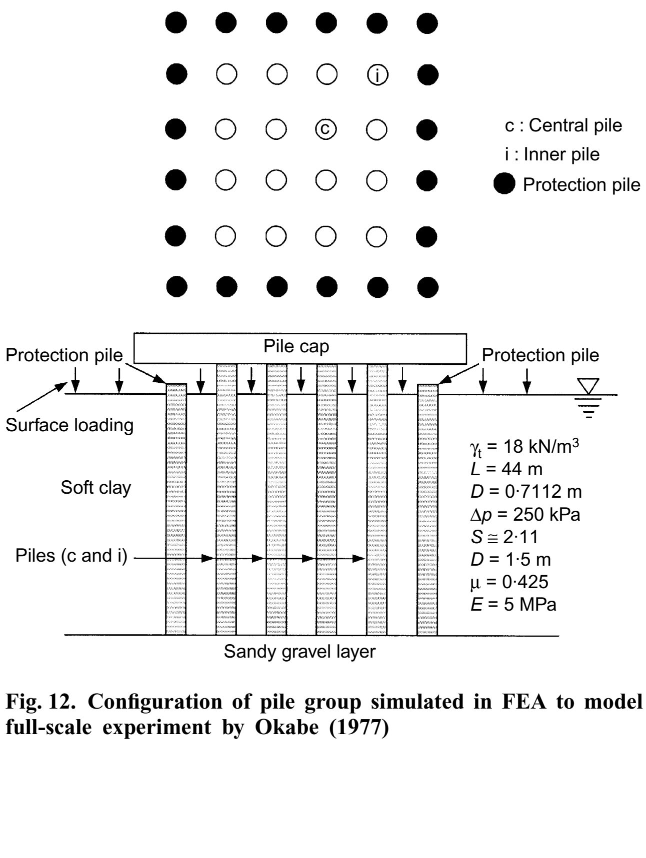

Fig. 12. Configuration of pile group simulated in FEA to model full-scale experiment by Okabe (1977)

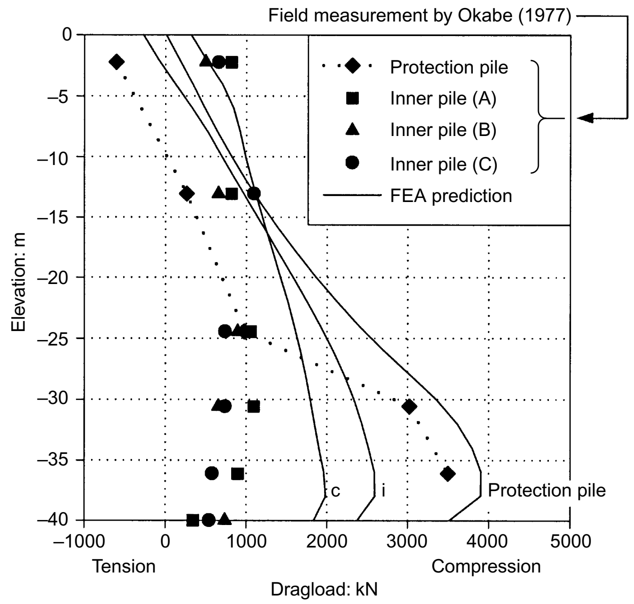

Fig. 13. Distribution of dragload (see Figs 11 and 12 for positions of the piles)

(1977) (51%), whereas smaller group effects (more dragload) are obtained for inner (60%) and central piles (72%) than the range 85–88% measured by Okabe (1977).

Case 4: Example case study by Combarieu (1985), later extended by Jeong (1992)

In this example a rectangular pile group of 3 × 4 presented by Combarieu (1985) is considered. Information on the configuration of the pile group and on the soil properties is shown in Fig. 14. It is assumed that a relative displacement of 5 mm would fully mobilise skin friction. Predictions of dragload on single piles and on piles in a group determined from various design methods have been presented based on Combarieu (1985) and Jeong (1992). Additional results from the FEA described in this paper, as well as work by Shibata *et al.* (1982), Chow *et al.* (1990), Teh & Wong (1995) and Chow *et al.* (1996) are discussed here.

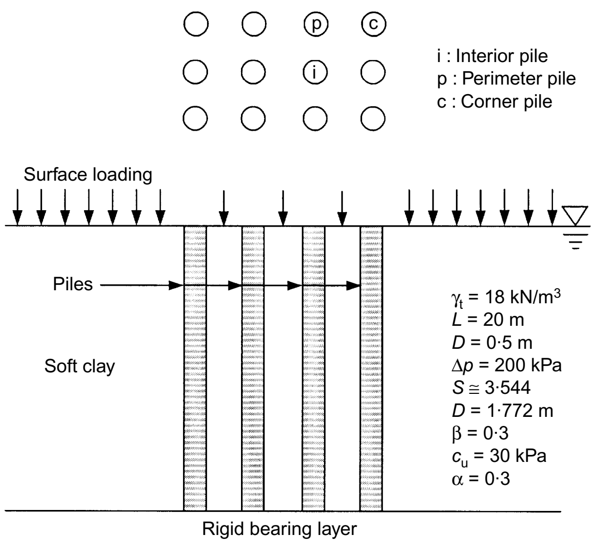

Fig. 14. Configuration of a pile group by Combarieu (1985)

Since detailed soil properties are not presented, typical soil properties are assumed for the purpose of this analysis. Assuming a K_0 value of 0·6–0·7 for normally consolidated clay and a β value of 0·3 as given by Combarieu (1985), a rough estimate of interface friction coefficient, μ, of 0·4 and 0·5 is obtained. The stiffness modulus, E, of clay is taken to be 5 and 10 MPa respectively. Both 2D and 3D FEA are carried out for a single pile and for piles in a group respectively.

As shown in Table 4, less dragload is predicted for a single pile from the analyses described here than from the conventional β method. Since the pile tip is located on a rigid bearing layer, the neutral plane is very close to the pile toe. Therefore reduced skin friction due to partial mobilisation near the neutral plane does not have a significant effect. Also, since the surface loading is very large (200 kPa), the maximum shear stress could have been developed along the entire pile length. However, when considering the effective vertical stress near the pile, stresses that are smaller than expected are observed owing to the transfer of soil weight to the pile shaft, and hence less shear stress developed along the pile shaft. Therefore the dragload estimate based on the β method should give an upper-bound estimate, as discussed previously.

For piles in a group, very large group effects are predicted from conventional approaches (1–5 in Table 4), 3D FEA based on continuum analyses (6), a simplified method (7) and a graphic method (8). Extremely large group effects (54–84%) are predicted for the central pile. However, such large group effects would be possible only for small pile spacing, for large surface loading, and when the pile numbers are large, as shown in the previous comparison. A smaller reduction in dragload (2–14%) is obtained from the slip model. Similar observations are made by various researchers (9–11 in Table 4), although slightly different pile configurations are considered: that is, a 3 × 3 pile group and a pile spacing of 3·0D. Despite the number of piles being more (12), pile spacing is wider (3·5D) in this example. Therefore it can be assumed that the difference would be insignificant. An overall small group reduction of (2–25%) is obtained from these analyses (9–11), which is similar to the prediction based on the slip model. Furthermore, it had been shown from the experimental observations that the group effect was relatively small for most cases. It was found that conventional design methods normally overpredict dragload and group effects.

DISCUSSIONS AND CONCLUSIONS

Should the potential exist for the development of NSF on piles in soft ground, dragload (compressive force) is normally not a major problem in terms of the design strength of the pile material. However, downdrag (pile settlement) could present some difficulties from a serviceability viewpoint. Piles should therefore be installed to a stiff layer in order to reduce downdrag, depending on driveability and dragload (Lee, 2001). Friction piles should therefore be used with great care, since a strong connection to a stiff pile cap or raft may be required to prevent differential settlement.

The results of the numerical analyses described here compare well with elastic solutions, recent theoretical studies, a number of field observations and an example case history. It has been found that the estimation of dragload and group effects from current design methods is not satisfactory, nor is it realistic. Dragload is normally overestimated from empirical methods and elastic and continuum analyses. If a granular surcharge creates an increment of vertical stress, current practice is to assume that this remains constant through the compressible clay. However, some of this extra

Table 4. Predicted group effects

	Group (total): kN	Dragload (kN) & group effect (%)			Single pile (kN)
		Corner*	Perimeter*	Interior*	
β-method					2640
(1) Terzaghi & Peck (1948)	β-method: 33 130				
	α-method: 11 832				
(2) Zeevaert (1957)	26 620	2640	2480	590	
		0%	6%	78%	
(3) Broms (1966)	Case 1: 31 680	(4775)[1] (0%)	1730 (34%)	590 (78%)	
	Case 2: 33 130	2640			
	Case 3: 22 120				
		0%	34%	78%	
(4) Broms (1976)	27 580	2640	2640	590	
		0%	0%	78%	
(5) Combarieu (1985)	10 448	1265	758	420	
		52%	71%	84%	
(6) Jeong (1992)	18 746	2054	1541	642	2568
		20%	40%	75%	
(7) Briaud *et al.* (1991)	17 096	1980	1320	628	
		25%	50%	76%	
(8) Shibata *et al.* (1982)	17 636	1663	1426	1214	
		37%	46%	54%	
(9) Teh & Wong (1995)	Pile configuration 3×3,	2%		10%	
	$3{\cdot}0D$ for cases				
(10) Chow *et al.* (1990)	9–11	15%	18%	22%	
(11) Chow *et al.* (1996)		10–25%	10–25%	10–25%	
(12) Present study	$\mu = 0{\cdot}5$, $E = 5$ MPa	1878	1802	1722	1916
	21 768	2%	6%	10%	
	$\mu = 0{\cdot}5$, $E = 10$ MPa	1809	1769	1674	1945
	21 198	7%	9%	14%	
	$\mu = 0{\cdot}4$, $E = 5$ MPa	1491	1474	1445	1534
	23 478	3%	4%	6%	
	$\mu = 0{\cdot}4$, $E = 10$ MPa	1486	1465	1442	1558
	17 618	5%	6%	7%	

* See Fig. 14 for position of piles
Notes:
1. Calculated dragload for the corner pile (4775 kN) is larger than for the single pile (2640 kN).
2. For calculation of dragloads for the single pile and for cases (1)–(6), see Combarieu (1985) and Jeong (1992).

load is transferred to the piles, and this can best be accounted for in a proper numerical analysis. Current methods also overemphasise group effects by failing to account for soil slip (soil yielding), which reduces the protection offered to a pile inside a group by its neighbours. Numerical simulations of the field observations confirm that more realistic interface and soil slip modelling must be introduced if analyses are to be at all accurate.

The numerical analyses described here predict a reduction in dragload due to group effects varying from 5% to 48% and from 19% to 79% for 3×3 and 5×5 groups in 2·5D spacing respectively. These group effects are significantly smaller than the previous research works, as discussed in the current work, which should therefore be reconsidered. The development of dragload and group effects is found to be heavily dependent on soil slip at the pile–soil interface. This is governed by interface friction, pile configuration and soil settlement. Various factors, including the relative position of piles within a group, the number of piles, the pile spacing, surface loading, soil stiffness and pile type should therefore be considered in the determination of dragload and group effects.

While non-linear finite element analysis must always be validated against field or centrifuge test data, such data are in fact increasingly available, and show that simple models of foundation behaviour are misleading and inaccurate. The general lesson to be drawn from this work is that the pile–soil interactions within a pile group, together with corresponding stiffness and soil slip, must be considered if the serviceability of the foundation system is to be properly assured.

ACKNOWLEDGEMENTS

The first author is grateful to Professor S. Springman of the Swiss Federal Institute of Technology and Dr M. S. P. G. Madabhushi of the University of Cambridge for technical advice.

NOTATION

c	cohesion of soil
c_u	undrained shear strength of clay
D	diameter of pile
E	Young's modulus
g	centrifugal gravity
K	relative pile stiffness (E_{pile}/E_{clay})
K_o	earth pressure coefficient at rest
L	pile length
P_a	dragload
Δp	surface loading
S	pile spacing
S_o	surface settlement
Z	distance from the pile head
α	total stress parameter
β	effective stress coefficient
γ_t	total unit weight of soil
γ_{crit}	limiting displacement

δ interface friction angle between soil and pile
μ friction coefficient ($\tan\delta$)
ν Poisson's ratio
σ' effective normal stress on pile surface
τ shear stress
ϕ_c friction angle of soil at critical state
ψ dilation angle of soil
Group effect $(\max(P_{\text{single pile}}) - \max(P_{\text{pile under consideration within the group}}))/\max(P_{\text{single pile}})$, where P is dragload

REFERENCES

ABAQUS (1998). *ABAQUS User's and Theory Manuals*, Version 5·8. Rhode Island: Hibbitt, Karlsson & Sorensen, Inc.

Acar, Y. B., Avent, R. & Taha, M. R. (1994). *Downdrag on friction piles: A case history*, Geotechnical Special Publication No. 40, pp. 986–999. New York: ASCE.

Briaud, J. L., Jeong, S. & Bush, R. (1991). *Group effect in the case of downdrag*, Geotechnical Engineering Congress, Geotechnical Special Publication No. 27, pp. 505–518. New York: ASCE.

Broms, B. B. (1966). Methods of calculating the ultimate bearing capacity of piles – a summary. *Sols-Soils*, **5**, No. 18–19, 21–32.

Broms, B. (1976). Pile foundations – Pile groups. *Proc. 6th European Conf. of Soil Mech. and Found. Engng, Vienna, Austria*, **2.1**, 103–132.

Broms, B. (1979). Negative skin friction. *Proc. 6th Asian Regional Conf. Soil Mech. Found. Engng, Singapore* **2**, 41–75.

Burland, J. B. (1973). Shaft friction of piles in clay: a simple fundamental approach. *Ground Engng* **6**, No. 3, 30–42.

Bustamante, M. (1999). Current French design practice for axially loaded piles. *Ground Engng.*, 38–44.

Chow, Y. K., Chin, J. T. & Lee, S. L. (1990). Negative skin friction on pile groups. *Int. J. Numer. Anal. Meth. Geomech.* **14**, 75–91.

Chow, Y. K., Lim, C. H. & Karunaratne, G. P. (1996). Numerical modelling of negative skin friction on pile groups. *Comput. Geotech.* **18**, No. 3, 201–224.

Combarieu, O. (1985). *Frottement négatif sur les pieux*, Rapport de recherche LCPC No. 136. Paris: Laboratoire Central des Ponts et Chaussées.

Davisson, M. T. (1993). Negative skin friction in piles and design decision. *Proc. 3rd Int. Conf. Case Histories Geotech. Engng, Rolla* **3**, 1793–1801.

Denman, K. J., Nicholls, R. A. & Symons, M. V. (1978). Model studies of negative friction on pile groups. In *Proc. Large Ground Movements and Structures* (ed. J. D. Geddes), Cardiff, pp. 252–271.

Fellenius, B. H. (1999). *Piling terminology.* http://www.geoforum.com/info/pileinfo/terminology.asp

Fioravante, V., Jamiolkowski, M. & Pedroni, S. (1994). Modelling the behaviour of piles in sand subjected to axial load. *Proc. International Conference, Centrifuge 94*, Rotterdam: Balkema, 455–460.

Jeong, S. (1992). *Nonlinear three dimensional analysis of downdrag on pile groups*. PhD thesis, Texas A & M University.

Koerner, M. K. & Mukhopadhyay, C. (1972). Behaviour of negative skin friction on model piles in medium-plastic silt. *Highway Res. Rec.* **405**, 34–44.

Kuwabara, F. & Poulos, H. G. (1989). Downdrag forces in group of piles. *ASCE J. Geotech. Engng* **115**, No. 6, 806–818.

Lee, C. J. (2001). *The influence of negative skin friction on piles and in pile groups*. PhD thesis, Cambridge University.

Lee, C. J., Chen, H. T. & Wang, W. H. (1998). Negative skin friction on a pile due to excessive ground water withdrawal. *Proc. International Conference, Centrifuge 98*, New York, 513–518.

Little, J. A. (1994). Downdrag on piles: review and recent experimentation. *Proc. Settlement 94: Vertical and Horizontal Deformations of Foundations and Embankments*, Rotterdam: Balkema, 1805–1826.

Little, J. A. & Ibrahim, K. I. (1993). Predictions associated with the pile downdrag study at the SERC soft clay site at Bothkennar, Scotland. In *Predictive Soil Mechanics (Wroth Memorial Symposium)*, pp. 796–818. London: Thomas Telford.

Nishi, K. & Esashi, Y. (1982). Field measurement and prediction of negative skin friction in piles. *Proceedings international symposium on numerical models in geomechanics*, Zurich, pp. 776–784.

Okabe, T. (1977). Large negative friction and friction-free pile methods. *Proc. 9th Int. Conf. Soil Mech. Found. Engng, Tokyo*, 679–682.

Phamvan, P. (1989). *Negative skin friction on driven piles in Bangkok subsoils*. PhD thesis, AIT, Bangkok, Thailand.

Poulos, H. G. & Mattes, N. S. (1969). The analysis of downdrag in end-bearing piles. *Proc. 7th Int. Conf. Soil Mech. Found. Engng, Mexico City*, 203–208.

Shibata, T., Sekiguchi, H. & Yukitomo, H. (1982). Model test and analysis of negative friction acting on piles. *Soils Found.* **22**, No. 2, 29–39.

Teh, C. I. & Wong, K. S. (1995). Analysis of downdrag on pile groups. *Géotechnique* **45**, No. 2, 191–207.

Terzaghi, K. & Peck, R. B. (1948). *Soil mechanics in engineering practice*. New York: John Wiley & Sons.

Thomas. J. (1998). *Performance of piles and pile groups in clay.* PhD thesis, University of Western Australia.

UniNews (1998). *Downdrag and dragload: aren't they synonyms?* http://www.unisoftltd.com/news98.htm.

Zeevaert, L. (1957). Foundation design and behaviour of Tower Latimo Americana in Mexico City. *Géotechnique*, **7**, No. 3, 115–133.

Zeevaert, L. (1983). *Foundation engineering for difficult subsoil conditions*, 2nd edn. New York: Van Nostrand Reinhold.