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Pile foundations: Experimental investigations, analysis and design

Fondations sur pieux: Recherche expérimentale, analyse et projet

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ABSTRACT: Selected topics in the field of pile foundations are addressed. The effects of the installation technique on the bearing capacity and the load-settlement response of a single pile are discussed. The latter effect is shown to be less significant; a settlement controlled design is thus less dependent on the technological factors. Monitoring of the installation parameters shows some potential for controlling the pile response. The available experimental evidence on the behaviour of pile foundations under vertical loads (settlement, load sharing, bearing capacity), by monitoring of full scale structures or by research experiments, is reviewed. Simple empirical methods for a preliminary evaluation of the settlement are suggested. The (more limited) evidence about horizontal loading is also reviewed and discussed. The methods for the analysis of pile foundations under vertical load are next reported. They may be considered satisfactory for engineering purposes, provided they are used paying due attention to the correspondence relations between theories and reality. The criteria for an optimum design, achieving maximum economy while keeping satisfactory performances, are different for different kinds of pile foundations (small groups, large rafts). Safety against a bearing capacity failure, average settlement, differential settlement, moment and shear in the raft and cost are the quantities to be controlled. It is claimed that the conventional capacity based approach, still prevailing in practice, is not suited to develop a proper design. Present codes and regulations, essentially based on this approach, at the time being act as a restraint rather than a stimulus and need some revision.

RESUME: On présente une sélection de thèmes concernant le domaine des fondations profondes. On discute de l'effet du mode d'installation sur la capacité portante et sur la réponse charge-tassement d'un pieu isolé. On montre que ce dernier effet est moins significatif ; un dimensionnement fondé sur des critères de tassement est donc moins dépendant des facteurs technologiques. Le suivi des paramètres d'installation représente une voie potentielle pour le contrôle de la réponse du pieu. Une revue des données expérimentales sur le comportement des fondations sur pieux sous charges verticales (tassement, distribution de la charge, capacité portante) est présentée à travers les mesures effectuées sur des ouvrages réels ou celles sur des expériences de recherche. On propose des méthodes empiriques simples pour une première évaluation du tassement. Le cas (plus limité) du chargement horizontal est également passé en revue et discuté. Les méthodes d'analyse des fondations sur pieux sous charge verticale sont ensuite discutées. Elles peuvent être considérées comme satisfaisantes pour les besoins de l'ingénieur, pourvu qu'elles soient utilisées en faisant bien attention aux relations de passage de la théorie à la réalité. Les critères pour un dimensionnement optimum, le plus économique tout en gardant la meilleure performance, sont différents selon les différents types de fondations sur pieux (groupes à faible nombre de pieux, radiers de grande dimension). Les paramètres à contrôler sont la sécurité vis à vis de la capacité portante limite, le tassement moyen, les tassements différentiels, les moments et cisaillements dans le radier, et enfin le coût. Le dimensionnement classique fondé sur la capacité portante, qui prévaut encore dans la pratique, n'est pas adapté pour développer un dimensionnement approprié. Par conséquent les codes et règlements actuels, fondés essentiellement sur cette approche, représentent une restriction plutôt qu'un stimulant et nécessitent une certaine révision.

1 INTRODUCTION

Piles have been used by mankind for foundation purposes since prehistoric times; their behaviour, however, is far from completely clear and a substantial volume of research is being carried out on the subject. The field is in evolution with continuous developments in the technologies, in the methods of analysis and in the design approaches. In fact the design of piles is a rather complex matter which, although based on the theoretical concepts of soil mechanics, heavily relies on empiricism. This is an inevitable consequence of the marked variability of behaviour of the piles, which is partly due to random factors but depends also on the effects of the installation techniques (De Beer, 1988; Van Weele, 1988; Van Impe, 1991; Viggiani, 1989, 1993).

According to Van Impe (2003), bored and CFA piles account for 50% of the world pile market, while the remaining is mainly covered by driven (42%) and screw (6%) piles. Summing up, the market is equally subdivided between displacement (driven, jacked, screwed, etc.) and non-displacement piles (bored, continuous flight augered, etc.).

Different proportions may be found locally: for instance displacement screw piles are about 60% of the total installed yearly in Belgium (ten times than in the world market) while bored and CFA piles reach more than 90% of the total in Italy (about two times than in the world). Again in Italy, in recent years CFA piles gained market against other bored piles, increasing from about 30% (Trevisani, 1992) to about 55% (Mandolini, 2004).

The regional practice in the different countries develops along different paths under the push of the local market. Such a situation brought the Belgian Geotechnical Society and the European Regional Technical Committee (ERTC3) of the ISSMGE to organize an International Seminar on the design of axially loaded piles (De Cock & Legrand, 1997) with the aim of reviewing the practice in the European countries. Irrespective of the most widespread type of pile in each country, the contributions to the Seminar confirmed that the common approach for the design of a single pile is still based on semi-empirical rules, sometimes calibrated against purposely performed load tests (Van Impe *et al.*, 1998).

A number of comprehensive and authoritative reports on pile foundations have been issued in recent years (Randolph, 1994, 2003; Poulos *et al.*, 2001; Mandolini, 2003; Poulos, 2003); accordingly, the present Report will not attempt a complete coverage of the matter, but rather address some selected topics believed to be timely and relevant.

2 SINGLE PILE

2.1 *Experimental evidence and investigations*

2.1.1 *Effects of the installation technique*

The installation effects are particularly significant for piles under vertical load, which is also the most common loading condition. In fact, the ultimate bearing capacity of a vertically loaded pile depends essentially on the characteristics of the soil immediately adjacent to the shaft and below the base of the pile; in these zones the installation produces significant variations of the state of stress and soil properties. Under horizontal load the effects are much less important, since the volume of soil influencing the behaviour of the pile is less affected by the installation. Accordingly, only vertical loads will be addressed here.

The problem is of particular concern and stimulated in recent years a number of initiatives by several countries and/or institutions. Among them:

- the prediction events planned by the Belgian group (Holeyman & Charue, 2003) and by Portuguese group (ISC2, 2004) of ISSMGE. Among other scopes, such events were aimed to compare the response of different piles installed in the same subsoil condition;
- the systematic collection of results of load test on piles installed with different procedures in different soils, to develop an extensive Deep Foundation Load Test Database (Federal Highway Administration, USA);
- the analysis of the experimental evidence on single pile for the assessment of the existing design methods and the development of new design methods for pile types not covered by the existing codes and regulations (Laboratoire Central des Ponts et Chaussées, France).

The installation technique affects: (i) the ultimate bearing capacity and (ii) the load-settlement response or axial stiffness of the pile. In recent years the focus is moving from the former to the latter topic, following the development of a settlement based design approach to replace the traditional capacity based one, allowing for a more rational design and substantial savings. Both the topics are discussed in the following.

Effect on the bearing capacity

The ultimate bearing capacity Q_S of a single pile with length L and diameter d may be written as:

$$Q_S = \frac{\pi d^2}{4} \cdot q_B + \pi d L \cdot q_S \tag{1}$$

where q_B and q_S represent the unit base resistance and the average skin friction respectively. The dimensionless ratio between the bearing capacity and the weight of the pile P is:

$$\frac{Q_S}{P} = \frac{1}{L\gamma_p} \cdot \left(q_B + 4\frac{L}{d} \cdot q_S \right) \tag{2}$$

where γ_p is the unit weight of the pile material. The ratio depends on q_B and q_S, and hence on soil properties, but also on L and L/d.

To demonstrate the influence of the installation technique, a data base of 20 load tests to failure on piles installed in the relatively uniform pyroclastic soils of the eastern Naples area will be employed. The 20 trial piles are all cast in situ concrete piles (bored, driven and CFA); the diameter d ranges between 0.35 m and 2 m; the length L between 9.5 m and 42 m; the ratio L/d between 16 and 61. The experimental values for Q_S were obtained at a displacement of the pile head equal to 10%d, either directly attained in the test or determined by hyperbolic extrapolation. The results are summarized in table 1 in terms of the ratio Q_S/P.

Bored piles give the smallest value (Q_S on average 12 times greater than the weight of the pile) and the larger scatter (COV = 26%); driven piles give the largest value (73 times the weight of the pile) and the smallest scatter (COV = 8%); CFA piles have an intermediate behaviour.

Table 1: Bearing capacity of piles in the soils of eastern Naples area

Pile type	$\left(\frac{Q_s}{P}\right)_{av}$	$COV\left(\frac{Q_s}{P}\right)$
Bored	12.1	0.26
CFA	37.5	0.25
Driven	73.1	0.08

Results of this type can be useful for assessing quantitatively the effects of different installation procedures in relatively uniform subsoil conditions like those prevailing in the eastern Naples area.

Effect on the load-settlement behaviour

Randolph (1994) modelled the installation effect on the axial stiffness of a pile by assuming:

- a linear radial variation of the shear modulus from a value G at the interface between pile and soil (r = d/2) to the “undisturbed value” G_0 (r = R),
- at a low load level the external load applied to a properly designed pile is transmitted to the surrounding soil primarily by skin friction along the shaft.

Calling K_0 the axial stiffness of the pile without installation effects and K the stiffness affected by the installation, the ratio K_0/K is reported in figure 1 as a function of $R^* = R/r$ and $G^* = G/G_0$. The diagrams refer to the set of values of the relevant parameters reported in the insert. The range of values $G^* > 1$ is representative of displacement piles, for which a higher soil stiffness in the zone immediately around the shaft may be expected; values of $G^* < 1$, on the contrary, represent non-displacement piles.

On the basis of the available experimental evidence (Van Weele, 1988; Peiffer & Van Impe, 1993; Viggiani, 1993) Mandolini (2003) found out that G^* and R^* may be expected to fall in the range 0.5 to 3 and 3 to 5, respectively. In this range, the effect on the pile stiffness is less than ± 20%.

These findings have been checked against the results of 125 pile load tests carried out in the soils of eastern Naples area, where the small strain stiffness had been determined by shear waves velocity measurements. All the piles are cast in situ concrete piles, but installed with different procedures:

- bored with temporary casing or bentonite mud
- bored CFA
- bored/screwed (Pressodrill)
- driven (Franki)

In order to process the data in an objective and repeatable way, the initial axial pile stiffness K was determined as the initial tangent of a hyperbola fitted to the first three points on the experimental load-settlement curve. The results obtained are shown in figure 2. The value of K has been normalised against the axial stiffness $K_C = \pi d^2 E_P/4 L_C$ of a column with a length equal to the critical length $L_C = 1.5\ d\ (E_P/G_L)^{1/2}$ (Fleming *et al.*, 1992), beyond which any increase of the pile length causes little or no increase of the pile stiffness. G_L is the value of the soil shear modulus at a depth L_C; it follows that some iterations are required in order to determine L_C.

The values of the ratio K/K_C falls in the range 0.94 to 1.90 for all the piles (average value ~ 1.4) with 16% < COV < 63%, average value ~ 35%). These findings convey essentially the same message of figure 1.

For a long time it has been claimed that the installation tech-

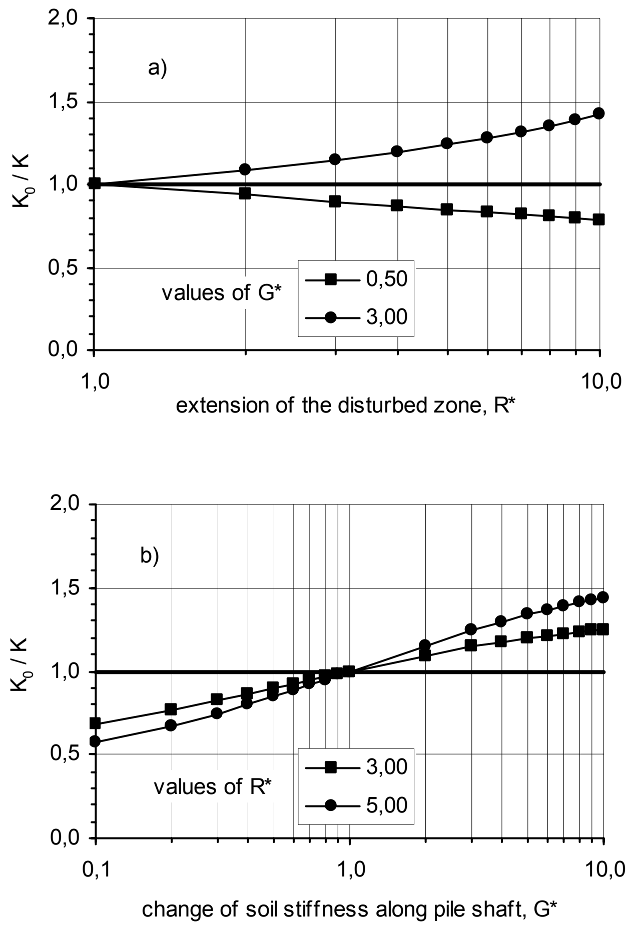

Figure 1. Influence of the extension R* of the disturbed zone (a) and of the change G* of the soil stiffness (b).

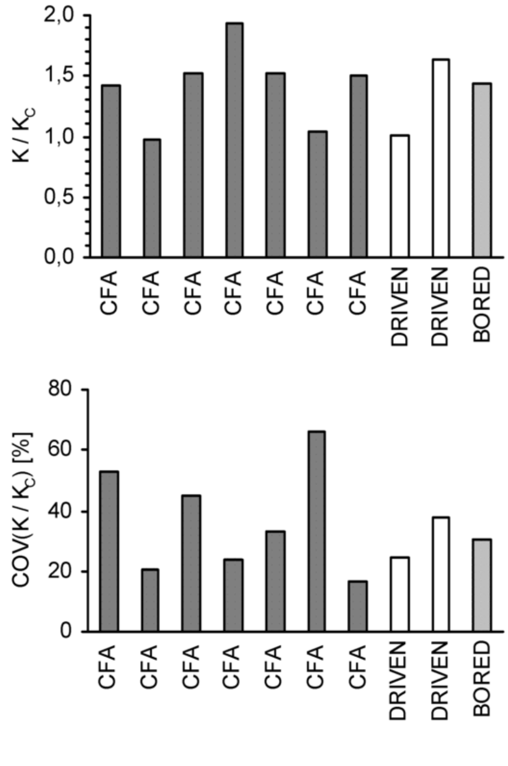

Figure 2. Variability among piles belonging to the same foundations in pyroclastic soils of eastern Naples area.

nique affects the axial stiffness of the piles much less than their bearing capacity (Poulos, 1989; Viggiani, 1989, 1993; Randolph, 1994; Van Impe, 1994). The data collected in table 1 and in figures 1 and 2 seem to support this view and confirm that the initial stiffness of the piles depends primarily on the small strain shear modulus of the soil (Mandolini, 1994; Randolph, 1994).

2.1.2 *Monitoring of the installation parameters*

Monitoring of the installation parameters is a common practice in some fields. An obvious example for driven piles is the use of set measurements in driving formulas, and its evolution in the dynamic analyses of pile driving.

Interesting developments have been recently recorded in the field of CFA piles. These are installed by means of an auger with an hollow stem, inserted into the soil by the combined action of an axial thrust and a torque. The stem is provided with a temporary closure plate at the bottom; once reached the desired depth, the plate is pushed out by pumping concrete or mortar through the stem, and the auger is lifted removing the soil within the screw. The sides of the hole are thus supported at all times by the soil filled auger or by the pumped concrete. The procedure allows a rapid and noiseless installation of piles with diameters up to 1 m and lengths up to some tens of metres, and is becoming increasingly popular and widespread all over the world.

During the insertion, the ratio between the rate of penetration V_P and the rate of revolution n is generally less than the pitch of the screw p. The penetration thus involves both a displacement and a removal of soil. If the volume of the soil removed during penetration is less than the displaced volume, the net effect is a compression of the soil surrounding the pile; the resulting stress state within the soil is somewhat intermediate between that of a bored pile and that of a driven one.

Viggiani (1989) defined a critical rate of penetration:

$$V_{Pcrit} = n\ p \cdot \left(1 - \frac{d_0^2}{d^2} \right) \tag{3}$$

where d is the overall diameter of the auger and d_0 the outer diameter of the central hollow stem. If V_P and n satisfy Eq. (3), during penetration the displaced volume equals the removed volume and the soil surrounding the pile is not decompressed. If $V_P > V_{Pcrit}$, the removed volume is less than the displaced one (net compression effect, similar to that of a driven pile); if $V_P < V_{Pcrit}$, the opposite is true (net decompression effect, similar to that of a bored pile).

Viggiani (1989) found that, in order to satisfy condition (3) whatever torque M_T is available, a substantial vertical thrust is needed up to a certain depth; at increasing depth the thrust needed to advance the auger tends to decrease and eventually vanishes. This finding agrees with the common experience of screwing a screw into the wood: at the beginning a substantial thrust on the screwdriver is needed, otherwise the wood is stripped, but once the screw has penetrated a sufficient depth, only a torque is needed to continue the penetration.

If the equipment lacks sufficient thrust capacity, then V_P falls below V_{Pcrit}. The auger acts partially as an Archimedean pump, the soil surrounding the auger loosens and the penetration becomes possible; the behaviour of the pile, however, approaches that of a non-displacement (bored) pile. Caputo & Viggiani (1988) reported examples of both satisfactory and unsatisfactory behaviour. Later on, Viggiani (1993) and Kenny *et al.* (2003) successfully interpreted those and other examples in the light of the above analysis.

During the extraction of the auger, concrete is pumped through the hollow stem at a prescribed rate V_C, while the auger is retrieved at a rate V_R. In a given time interval Δt a volume of concrete $Q_C = V_C \cdot \Delta t$ is installed, while raising the auger leaves a nominal volume $(\pi d_N^2 V_R \Delta t)/4$. The ratio between the volume of

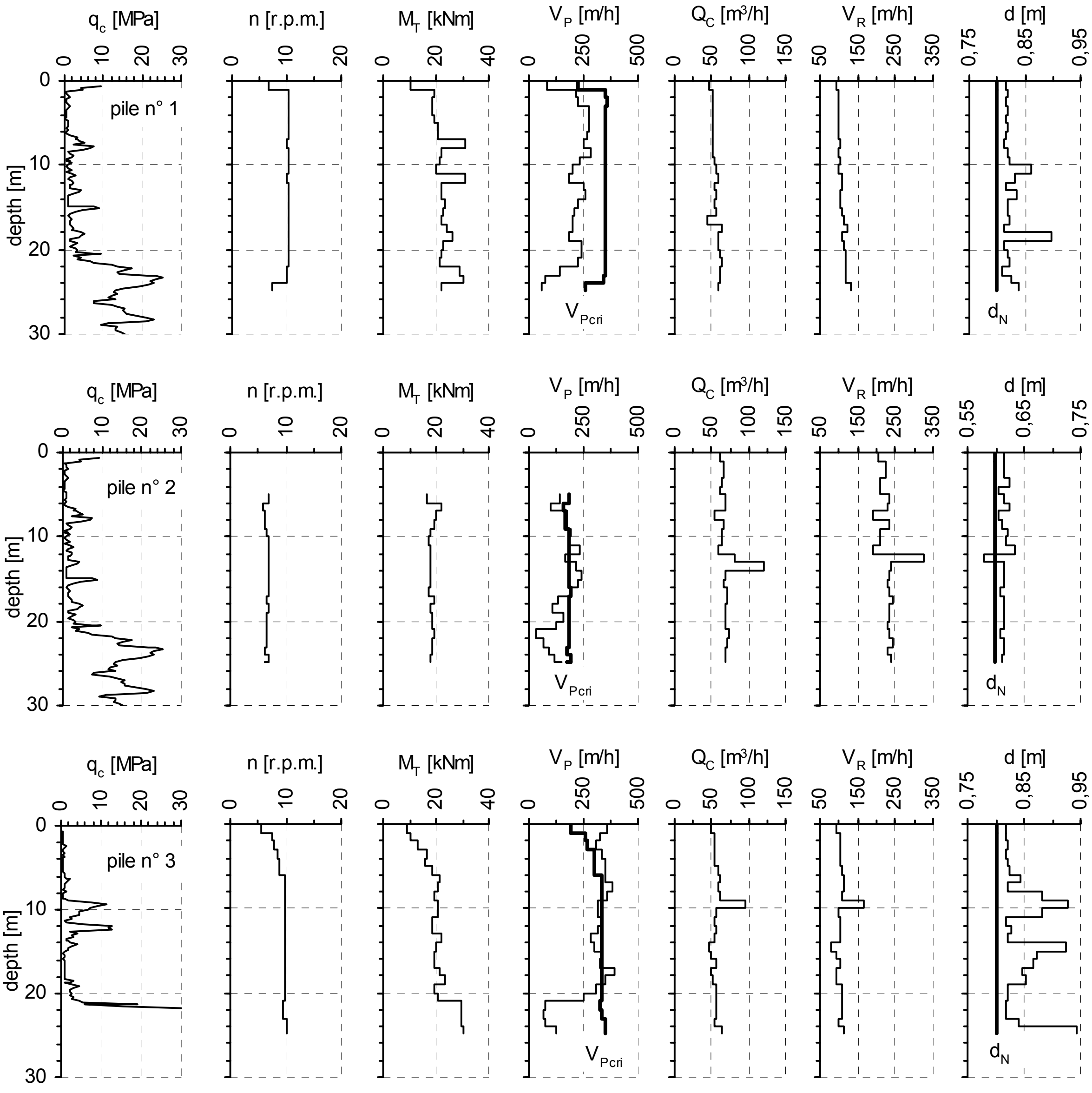

Figure 3. q_C-profiles and parameters measured during the installation of three piles: n° 1 (total length L_T = 25.5 m; embedded length L = 24 m; nominal diameter d_N = 0.8 m), n° 2 (L_T = 24 m; L = 22.5 m; d_N = 0.6 m) and n° 3 (L_T = 25.1 m; L = 24.1 m; d_N = 0.8 m).

concrete and the nominal volume is equal to $1.27 \cdot V_C / (d_N^2 V_R)$; if it is above unity, the effect is a lateral compression of the soil and hence a better behaviour of the pile, but also over-consumption of concrete and cost increase ($d > d_N$).

Three load tests to failure on trial instrumented CFA piles have been recently performed at a site were the subsoil conditions are relatively uniform in horizontal direction (Mandolini *et al.,* 2002). From the ground surface downwards the following soils are found: (a) topsoil, about 1 m thick; (b) alluvial soils of pyroclastic origin tightly interbedded with organic silt layers, about 20 m thick; (c) base formation of pozzolana to the maximum investigated depth (50 m). The groundwater table fluctuates between 1.2 m and 1.6 m below ground surface. Three CPT profiles are reported on the left side of figure 3.

The installation parameters of the test piles during the penetration (rate of revolution n, rate of penetration V_P and torque M_T) and during the extraction of the screw (concrete flow Q_C and retrieval rate V_R) are also reported in figure 3. Along most of the upper part of the pile shaft, crossing the alluvial soils (from the ground surface to a depth of about 20 m), the condition $V_P \geq V_{Pcrit}$ is satisfied for pile n° 2 and n° 3 but not for pile n° 1. Within the base formation of pozzolana, on the contrary, $V_P < V_{Pcrit}$; in that soil all the piles were thus installed essentially by boring.

The results of the load tests are reported in figure 4 as load-settlement curves (total load Q, shaft load, S and base load P) and load distributions along the pile shaft. Some relevant data are listed in table 2.

Table 2: Results of load tests

Pile n°	d_N (m)	L (m)	Q_{max} (MN)	w_{max} (mm)	P_{max} (MN)	S_{max} (MN)
1	0.8	24.0	4.08	75.6	1.55	2.81
2	0.6	22.5	3.26	81.9	0.89	2.59
3	0.8	24.1	5.30	22.8	1.36	3.94

The transfer curves of the shear resistance along the pile shaft and of the pressure at the pile base are reported in figure 5; the curves labelled "uncorrected" have been obtained referring to the nominal diameter d_N of the piles, those labelled "corrected" refer to the actual diameter $d = 1.13 \cdot (Q_C / V_R)^{0.5}$ obtained by the installation data.

Being the subsoil rather uniform, the differences in behaviour among the three piles are to be ascribed to differences in the installation details. The low unit shaft resistance of pile n° 1 is related to a penetration rate slower than the critical value (along the shaft V_P / V_{Pcrit} averages 0.66 < 1, table 3) determining an overall net decompression effect on the surrounding soil. On the contrary, during the installation of piles 2 and 3 the rate of penetration was on average larger than before (V_P / V_{Pcrit} = 0.96 to 1.05, table 3), with a slight compression effect on the surrounding soil giving rise to larger unit shaft resistances. The transfer curves of the base resistance for all the piles, once corrected for the actual base diameter, are practically coincident being equal the conditions of penetration of the auger. It may be noted that, in the absence of monitoring of the installation parameters and hence without a correction of the diameter, the higher base pressure for pile n° 3 would have been probably interpreted as due to random soil variability.

The unit shaft resistance q_S and unit base resistance q_B in granular soils can be related to the values of the cone penetration resistance q_C by the following expressions:

$$q_S = \alpha_S \cdot q_{c,S} \tag{4}$$

$$q_B = \alpha_B \cdot q_{c,B} \tag{5}$$

where: α_S , α_B are empirical coefficients; $q_{C,S}$ is the average value of q_C along the pile shaft down to a depth z = L – 4·d; $q_{C,B}$ is the average value of q_C between the depths (L – 4·d) and (L + d). The values of α_S and α_B are listed in table 3 and plotted in figure 6 against the corresponding ratios between the actual penetration rate and the critical one.

In table 3 the values of the ratio V_P / V_{Pcrit} averaged respectively along the pile shaft down to a depth z = L – 4·d and between the depths (L – 4·d) and (L + d) are also reported.

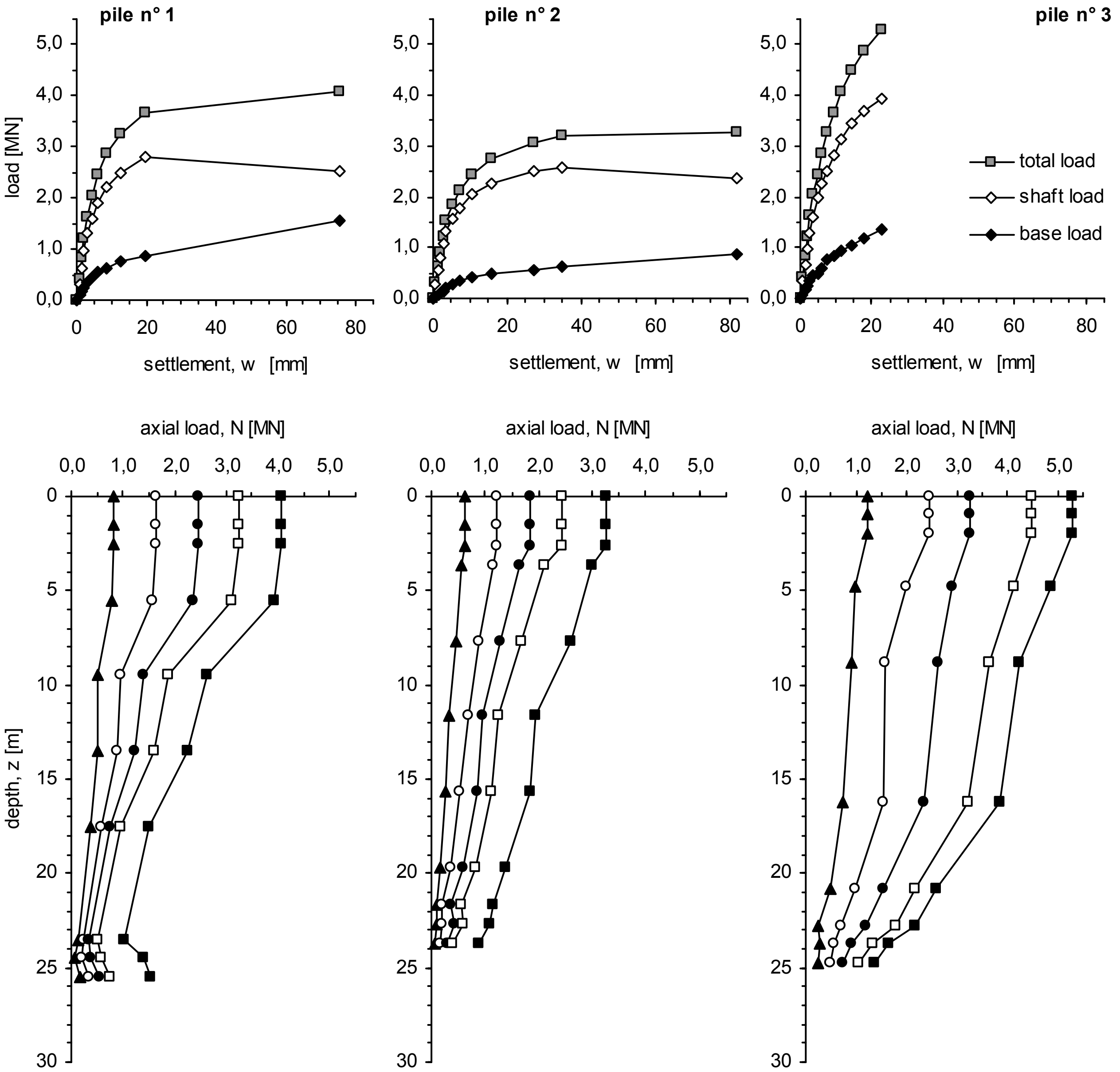

Figure 4. Load tests results: load-settlement curve (above) and axial load distribution along total pile length (below).

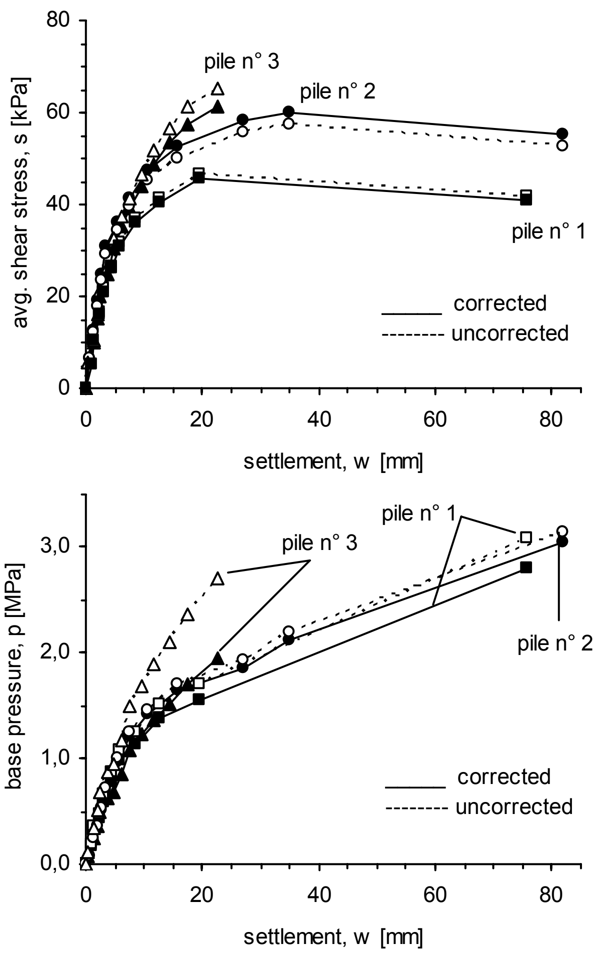

Figure 5. Corrected and uncorrected load transfer curves.

Table 3: Empirical pile design coefficients

Pile n°	V_P/V_{Pcrit} shaft	V_P/V_{Pcrit} base	$q_{S,max}$ [kPa]	$q_{B,max}$ [MPa]	α_S [-]	α_B [-]
1	0.66	0.22	45	2.8	0.021	0.180
2	0.96	0.81	60	3.0	0.029	0.246
3	1.05	0.29	68(1)	2.9(2)	0.031	0.184

(1) extrapolated value at w=4%d; (2) extrapolated value at w=10%d

As it was to be expected, the larger is the ratio V_P / V_{Pcrit} (either along the shaft or at the base), the larger is the corresponding coefficient α. These findings confirm that the behaviour of CFA piles is influenced by the installation procedures. It has been found that the volume of concrete supplied in the extraction stage plays a significant role too. A proper graduation of concrete pumping rates can compensate soil loosening occurred in the penetration stage and improve the performance of the piles, by increasing the pile diameter along the shaft and/or at the base and the horizontal soil pressure on the shaft.

All the above findings suggest the possibility of moving from monitoring to controlling the installation parameters.

2.1.3 *Static vertical load test*

It is widely accepted that a static load test to failure is the most reliable design method of a pile. As a matter of fact, most of the present insight into the behaviour of piles and the most significant advances in analysis and design have been obtained by collecting and interpreting load tests data.

Until a few years ago the aim of a load test was essentially the determination of the bearing capacity, to be employed in a capacity based design. Recently the attention is being switched to the settlement prediction, under the push of two main factors:

- the increasing use of large diameter bored piles, whose current design methods are settlement based (Jamiolkowski, 2004);
- the development of new design criteria for piled raft foundations, with piles as a mean to control the absolute and/or differential settlement.

The scope of pile load tests has thus broadened to include the determination of the whole load-settlement relationship.

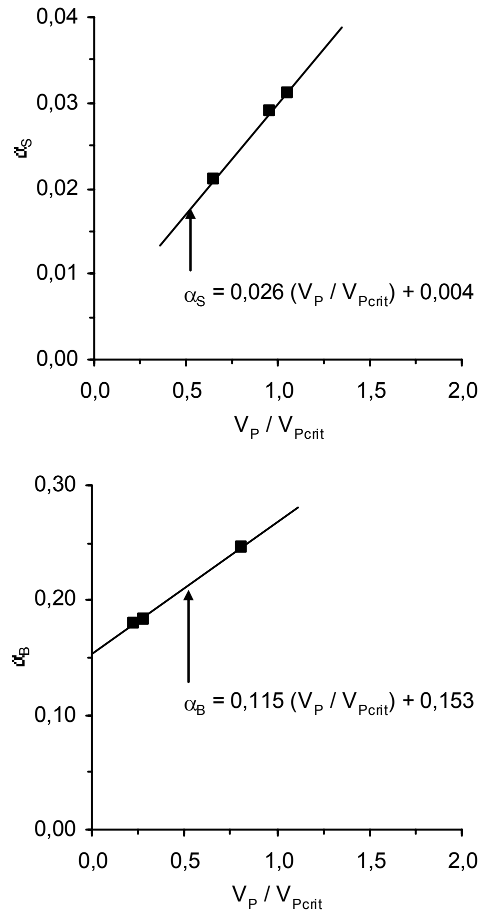

Figure 6. Relationships between α coefficients and the ratio V_P/V_{Pcrit}.

Load test practice

The static vertical load test is generally confused with the Ideal Load Test (ILT), figure 7a. In practice the load is applied to the pile by a hydraulic jack; the reaction system can be a kentledge resting on supports (figure 7b) or a beam anchored to the soil by tension piles or ground anchors (figure 7c).

Recently the so-called Osterberg cell (figure 7d), providing a "self-reaction", is becoming increasingly popular. The setups illustrated in figures 7b to 7d differ from the ILT because they apply to the ground a load system with zero resultant. The consequences on the load-settlement relationship and on the ultimate bearing capacity, as compared to that of ILT, will be discussed in the following.

Test with kentledge

In the case of a test with kentledge Poulos (2000a) claims that the stress arising in the subsoil from the weight of the kentledge tends to cause an increase of the shaft friction and end bearing pressure of the pile. As the load on the pile is increased by jacking against the kentledge, the stress will reduce and some upward displacements tend to develop in the soil, while the pile undergoes settlement. The pile head stiffness is thus overestimated, while the pile ultimate capacity may be relatively close to that of the ideal test.

Figure 8 reports the results of a parametric study on the load settlement curves of a pile subjected to an Ideal Load Test and to a test with kentledge. The curves have been obtained by nonlinear finite element elasto-plastic analyses assuming the soil to be a uniform sand with different values of the friction angle φ' and test piles with d = 1 m and L/d = 10, 20 and 50. The test with kentledge overestimates the initial stiffness of the pile, the more the higher the ratio L/d. On the contrary, at relatively large displacements (w = 10%d) the discrepancies decrease and eventually the value of the ultimate capacity is practically unaffected by the influence of the kentledge. Similar trends have been found for undrained clays.

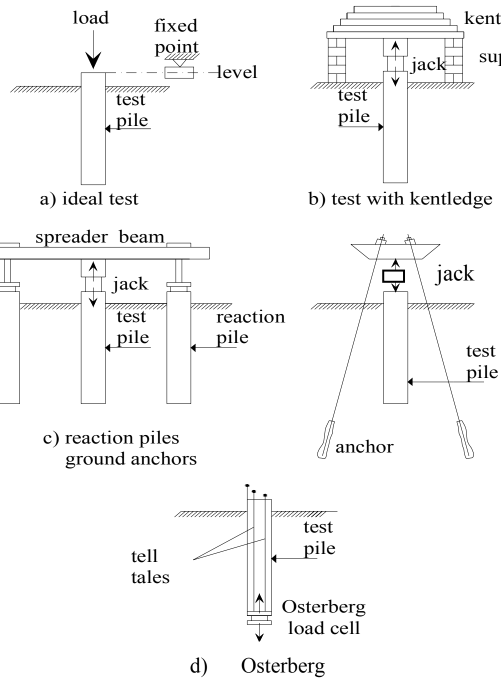

Figure 7. Various load tests setup and ideal test.

Reaction piles and ground anchors
The effect of interaction between reaction piles and the test pile is again an overestimation of the pile head stiffness. The overestimation may be very significant for slender piles and reaction piles close to the test pile (Poulos & Davis, 1980; Poulos, 2000a; Kitiyodom *et al.,* 2004). Some further results are reported in figure 9, which refers to a pile with d = 1 m, L/d = 20, two reaction piles identical to the test pile and different values of the spacing s between the test pile and the reaction piles (s/d = 4, 6, 10). Similar trends have been found for undrained clays.

In the case of a test pile jacked against ground anchors Poulos (2000a) has shown that the overestimation of the pile head stiffness is significantly less than when reaction piles are used, especially if the anchors are located well below the base of the test pile.

Osterberg Cell
The Osterberg Cell Test (OCT, figure 7d) has been developed commercially by Osterberg (1984). A special cell hosting one or more hydraulic jacks is cast at or near the pile base; by applying pressure, the base is pushed downward while the shaft is jacked upwards and provides the reaction. The test goes on until either the base or the shaft reach the ultimate resistance. This is a disadvantage of OCT, since only a lower bound of the total bearing capacity may be determined; the full value is approached only when the two resistances have nearly the same value. On the other hand in the OCT a reaction system is not needed; for this reason it may be a cheap alternative to tests with kentledge or reaction piles and anchors.

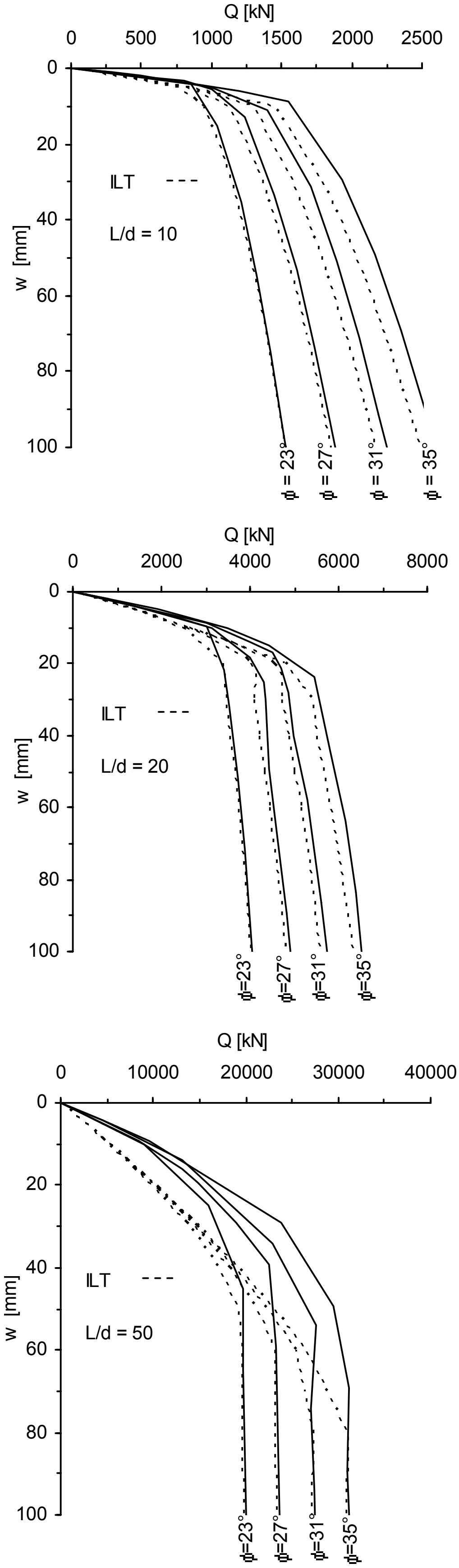

Figure 8. Load test with kentledge vs. Ideal Load Test.

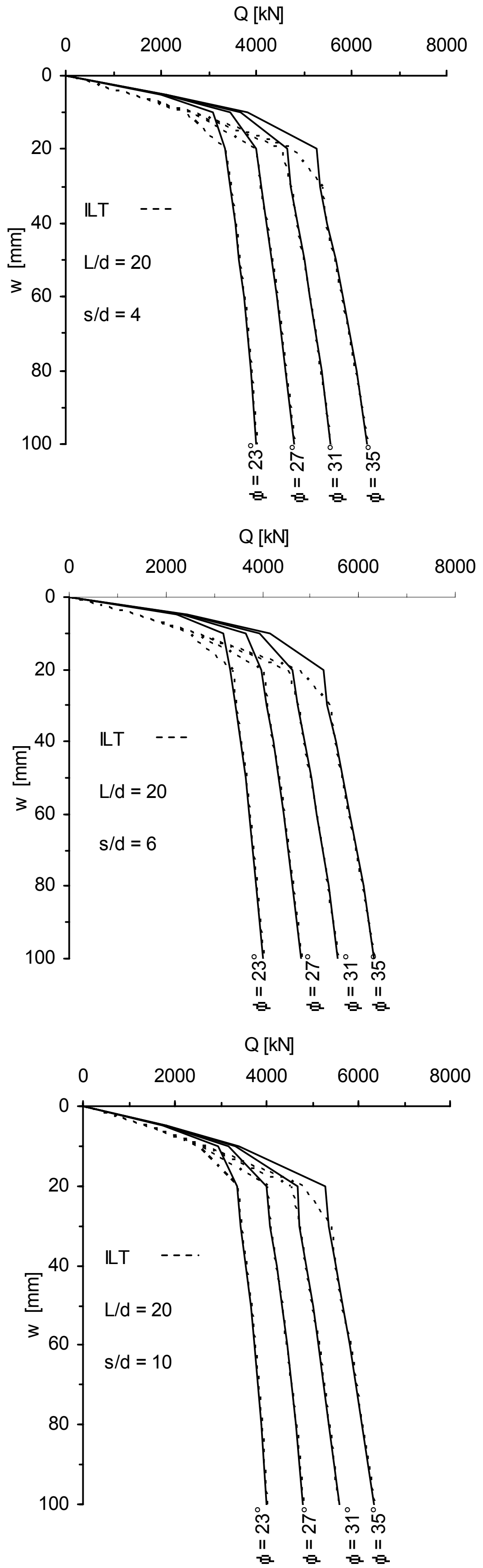

Figure 9. Load test with tension piles vs. Ideal Load Test.

Osterberg (1995) and Schmertmann & Hayes (1997) give suggestions to derive from the results of OCT a load-settlement curve of the pile head, equivalent to that obtained by the ILT. The suggested procedure relies upon two hypotheses:

- the pile is rigid;
- the load-displacement relationship for the shaft resistance is independent of the direction of the relative movement between the pile and the surrounding soil.

A third implicit assumption, which is often neglected, is that the stress and strain fields at the pile base and along the pile shaft are independent each other and the load-settlement relationship of the base and the shaft can be considered separately.

A parametric FEM analysis has been carried out on this topic (Recinto, 2004). The subsoil was assumed as a purely frictional or a purely cohesive, elastic perfectly plastic material. A comparison equivalent to that of figure 8 is reported in figure 10. A substantial overestimation of the pile head stiffness is again evident at low displacements, while a better agreement occurs in the late stage of the test. The shortcoming related to OCT is evident for piles with L/d = 50: the end bearing capacity is many times larger than the shaft capacity, preventing the OCT to explore the behaviour of the pile further than a settlement w = 1.5%d.

Figure 11 summarizes the results of comparisons between the ILT and other test setups in term of the ratio $k = (K - K_{ILT})/K_{ILT}$ for frictional and cohesive material.

Summing up, the test setups that have been examined are suitable for the determination of the bearing capacity; only the OCT may have significant limitation in this respect. As far as the load-settlement behaviour, and especially the initial stiffness, is the main purpose of the test, substantial corrections are needed in all cases. Without these corrections, any analysis based on the load test on single pile can be misleading and unconservative.

2.2 *Analysis*

Poulos (1989) classified the methods of analysis of a single vertically loaded pile in three main categories. The first one includes all the empirical procedures for predicting the bearing capacity and the head displacement. The second category is that of the methods based on some theoretical scheme, but characterized by significant simplifications. The third one is that of the advanced numerical methods, such as FEM and BEM. Another form of classification is that of separating methods to calculate the ultimate bearing capacity and methods to predict the settlement of the pile head. At the current state of the art theoretical contributions have greatly increased our insight of the mechanisms of pile failure, but practical predictions of the bearing capacity are still widely based on empirical or semi empirical approaches. On the other hand more sophisticated procedures have been developed and are actually used for settlement analysis.

2.2.1 *Bearing capacity*

Poulos *et al.* (2001) claim that, in principle, the effective stress approach to determine the bearing capacity of a pile, originally suggested by Burland (1973) and Meyerhof (1976), is the most acceptable one. Advances in this field include theoretical contributions (Randolph *et al.,* 1979; Viggiani, 1993); experimental investigations on carefully instrumented piles (Jardine & Chow, 1996); centrifuge tests (de Nicola & Randolph, 1993; Fioravante, 2002; Colombi, 2005). This work has produced a better insight of the mechanisms of development of side friction and base resistance; from a practical viewpoint, however, methods based on SPT (Meyerhof, 1956; Poulos, 1989; Decourt, 1995) and CPT (Poulos, 1989; MELT, 1993; De Cock *et al*., 1999) provide simple and adequate estimates of the bearing capacity.

Detailed scrutiny of most recent results on the topic of bearing capacity of a single pile have been provided, among others, by Poulos *et al.* (2001) and Jamiolkowski (2004).

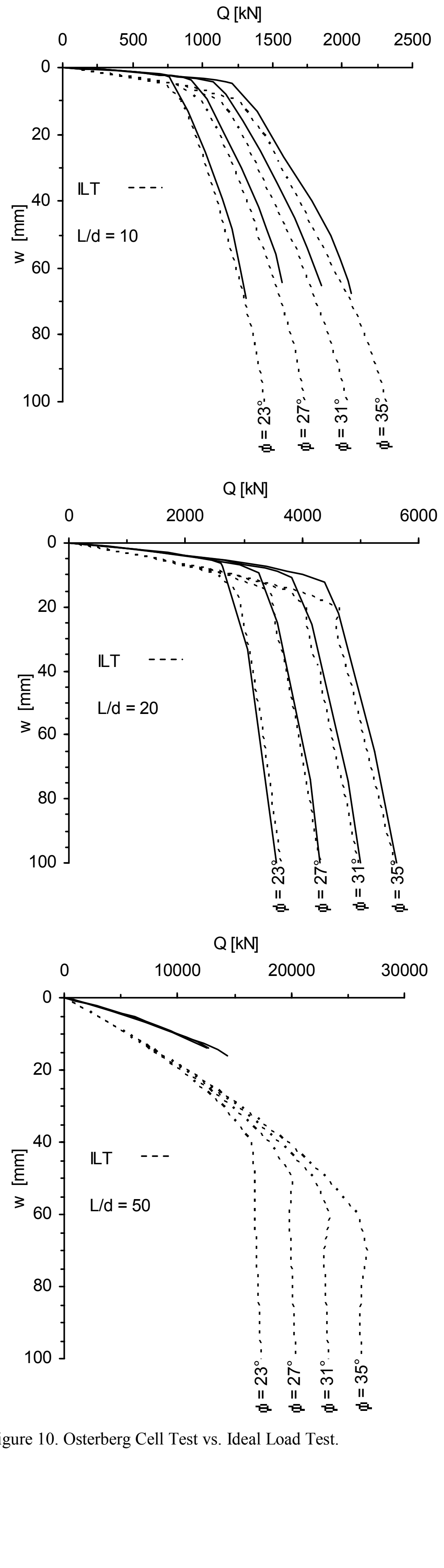

Figure 10. Osterberg Cell Test vs. Ideal Load Test.

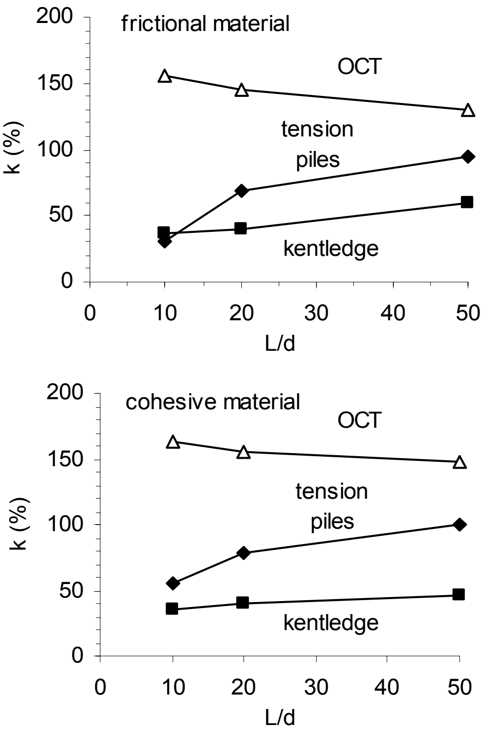

Figure 11. Ratio between the initial stiffness as deduced by different load test setup and by an Ideal Load Test.

2.2.2 *Load – settlement relation*

A variety of linear and non linear methods have been developed in the last decades for predicting the load – settlement response of a single vertically loaded pile. Poulos & Davis (1980) summarize the results obtained by the boundary element method using Mindlin (1936) solution as a Green function (D'Appolonia & Romualdi, 1963; Poulos & Davis, 1968; Mattes & Poulos, 1969; Butterfield & Banerjee, 1971). Randolph & Wroth (1978) produced simplified equations for the pile head settlement response.

Poulos & Davis (1968) proposed a cut-off procedure to account for local yielding along the pile shaft, thus developing a non linear boundary element technique. This procedure, however, typically predicts a load – settlement curve with a significant linear initial branch not corresponding to the actual pile behaviour. Van Impe *et al.* (1998) obtained a significant improvement just combining the cut-off procedure along the shaft with an appropriate t – z curve for the pile tip response.

The load transfer method, widely known as the "t – z" method, was originally proposed by Seed & Reese (1957). In the following years many contributions made the method to develop into one of the most popular and widespread tool for the analysis of a vertically loaded pile (Coyle & Reese, 1966; Wright & Reese, 1977; Randolph & Wroth, 1978; Kraft *et al.*, 1981; Randolph, 1986).

Whatever method of analysis is used, the non linear behaviour exhibited by the single pile as well as its bearing capacity are strongly affected by the installation procedures and thus hard to predict reliably. The settlement of a single pile, however, is rarely a conditioning factor for the design. Much more important is the settlement of the pile group; as it will be clarified below (§ 4.1.3) the non linear component of the settlement of the single pile can be almost neglected for large pile groups while it keeps a role in the cases of small groups at relatively high load level.

2.2.3 *Horizontal loads*

Most foundations are subjected to some horizontal loads which are generally smaller than the vertical ones, earth retaining structures being one of the exceptions. In the case of significant horizontal loads, raking piles have been also installed, providing horizontal resistance by means of the horizontal component of the axial capacity. Nevertheless also vertical piles can support horizontal loading; the present report will be limited to this topic.

Starting from the simple and effective idea of considering the pile as an elastic beam restrained by springs (Matlock & Reese, 1960), widely known as p-y method, many tools for the analysis have been subsequently developed. The p-y method is still widespread in practice; its main advantage is the ability to easily incorporate variations of the soil stiffness with depth. Non linear p-y curves were later introduced by Matlock (1970) and Reese *et al.* (1975). The p-y curves, either linear or non linear, must be deduced by experiments and cannot be easily transferred to different situations; accordingly, many experiments have been carried out to define them, mainly as a function of the soil type. Some questions are still open, however, on the influence of the geometry of the pile and the installation technique (O'Neill & Dunnavant, 1984; Reese & Van Impe, 2001; Huang *et al.*, 2001); for a comprehensive coverage reference may be made to Reese & Van Impe (2001).

Duncan *et al.* (1994) produced a series of solutions with non linear p-y curves and derived simple equations to predict the load - displacement relationship at the ground line, the maximum bending moment along the shaft and its depth of occurrence. Poulos *et al.* (2001) present in detail the procedure, known as the characteristic load method.

On the other hand Poulos (1971a) proposed the application of the boundary element method to the analysis of a vertical pile under horizontal load, modelling the soil as an elastic continuum. Evangelista & Viggiani (1976) pointed out the importance of a proper discretisation on the accuracy of the solutions.

A similar approach was proposed by Banerjee & Davies (1978) with the pile embedded into a non homogeneous soil.

Non linearity was introduced by Davies & Budhu (1986) and Budhu & Davies (1987) using a cut-off procedure to limit the maximum value of the interaction force between the pile and the surrounding soil.

Randolph (1981) obtained solutions by FEM and summarized the results into analytical expressions of the deflections and rotations of the pile head as well as the maximum bending moment along the pile shaft. Observing that the displacement and the bending moment along the pile shaft are usually confined to an upper portion of the pile, he defined a critical length L_C. If $L > L_C$, it is the critical length instead of the true length to govern the behaviour of the pile. The ratio L_C/d depends mainly on the relative pile-soil stiffness; typically $L_C/d < 10$. An interesting consequence is that only a limited upper portion of the soil profile must be adequately characterized.

Poulos (1982) reviewed some suggestions for the determination of the soil properties relevant to the prediction of the response of piles under horizontal load both in clays and in sands; a significant scatter can be revealed. Suggestion for the evaluation of the undrained shear modulus and its degradation are given by Poulos *et al*. (2001).

In the continuum based approaches non linear analyses usually requires a limiting value of the pile-soil interaction. The results by Broms (1964a, 1964b) are still widely used; some later assessments (Kulhawy & Chen, 1993) have confirmed their validity.

The maximum bending moment along the pile shaft, rather than the head displacement, is probably the critical design issue. Randolph (1981) and Duncan *et al*. (1994) developed simple but reliable analytical procedures for its evaluation, and showed that the maximum bending moment is much less sensitive than the deflection to the exact values of the stiffness parameters of the soil.

3 PILE GROUPS AND PILED RAFTS, EXPERIMENTAL EVIDENCE

3.1 *Monitoring of full scale structures*

In the early 1970's several buildings supported on piled foundations were monitored in UK (Hooper, 1979). In the 90's the investigations carried out during the construction of several tall buildings in Germany, mainly in the Frankfurt area (Katzenbach *et al.,* 2000), provided new stimulating data. These and similar observations led to a deeper insight into the mechanisms which govern the behaviour of piled foundations.

The behaviour of full scale structures contrasts with the simplicity of research oriented experiments, either at laboratory scale or in the field. The history of construction, the complexity of subsoil conditions and the interaction between the superstructure and the foundation make the back analysis of the observed behaviour far from straightforward. On the other hand, such a complexity constitutes a richness, and some times unexpected phenomena have been detected and highlighted.

The case history of the main pier of the cable stayed bridge over the river Garigliano (Southern Italy) will be reported in some detail as an example, adding some new results to those already published elsewhere.

The subsoil conditions at the site, reported in figure 12 (Mandolini & Viggiani, 1992a), are characterized by a deep, rather compressible silty clay deposit. The foundation of the main pier, resting on driven tubular steel piles, is represented in figure 13. Load tests to failure on instrumented piles and proof load tests on production piles were carried out. The foundation was monitored during the construction and afterwards, measuring settlement, load sharing between piles and raft and load distribution among the piles.

The construction of the bridge started in October 1991 and the latest set of data has been recorded in October 2004, thirteen years later. The settlement is measured by means of precision levelling; 35 out of the 144 piles were equipped with load cells at the top to measure the load transmitted by the cap to the pile; furthermore, 8 pressure cells were installed at the interface between the cap and the soil. The load cells and pressure cells were constructed on site using three sensing units for each of them; a total of 129 vibrating wire load sensing units were used. Further details on the instruments and the installation technique are reported by Mandolini *et al*. (1992) and Russo & Viggiani (1995).

In figure 14 the load history and the measured average settlement are reported; differential settlement was negligible due to the very stiff pile cap. The net load is the total applied load minus the buoyancy, as deduced by piezometer readings. An accurate evaluation of the total pile load can be obtained by the measurements on 35 piles, with only minor extrapolations. The total raft load as measured via 8 pressure cells was almost negligible at all stages. It is possible, however, that the pressure cells did not work properly since their installation.

The first load increments were due to the casting of the raft (October to November 1991) and of the pier (March to July 1992); with the construction of the bridge deck the applied load increased rapidly to its maximum value. In the early stage, when the raft was concreted, apparently almost the entire net load was measured on piles. About four months later, under constant applied load, the measured load on piles had vanished. The weight of the raft was actually supported by the soil, even if not measured by the pressure cells; the apparent pile load was an effect of the hydration heat of the concrete on the vibrating wire load sensors (Russo, 1996). Since the start of the installation of the bridge deck, in February 1993, the increments of the applied load match almost exactly the corresponding increments of the observed total load on piles. Except the weight of the raft, almost the entire weight of the bridge was transmitted to the piles, sometimes with a minor delay. At the end of construction (March 1995) the settlement was about 42 mm; in the following ten years it has progressively increased to reach 52 mm in October 2004.

At the time being, 13 years after their installation, 127 of the 129 vibrating wire load sensors are still properly working; the last set of measurements confirms that the dead weight of the bridge is still resting almost entirely on the piles. This finding is to be related to the subsoil properties and to the design of the foundation, based on a conventional bearing capacity approach in which no reliance is given to the load transmitted by the raft to the soil.

In figure 15 a plan view of the foundation with the location of the 35 instrumented piles is reported. The behaviour of the various piles can be grouped into four distinct categories, corresponding to four zones underneath the pile cap. In table 4 the average values of the pile load for each of the selected areas are reported, as a ratio to the mean value of all the piles. The values reported refer to three different stages: end of construction, three years later and ten years later.

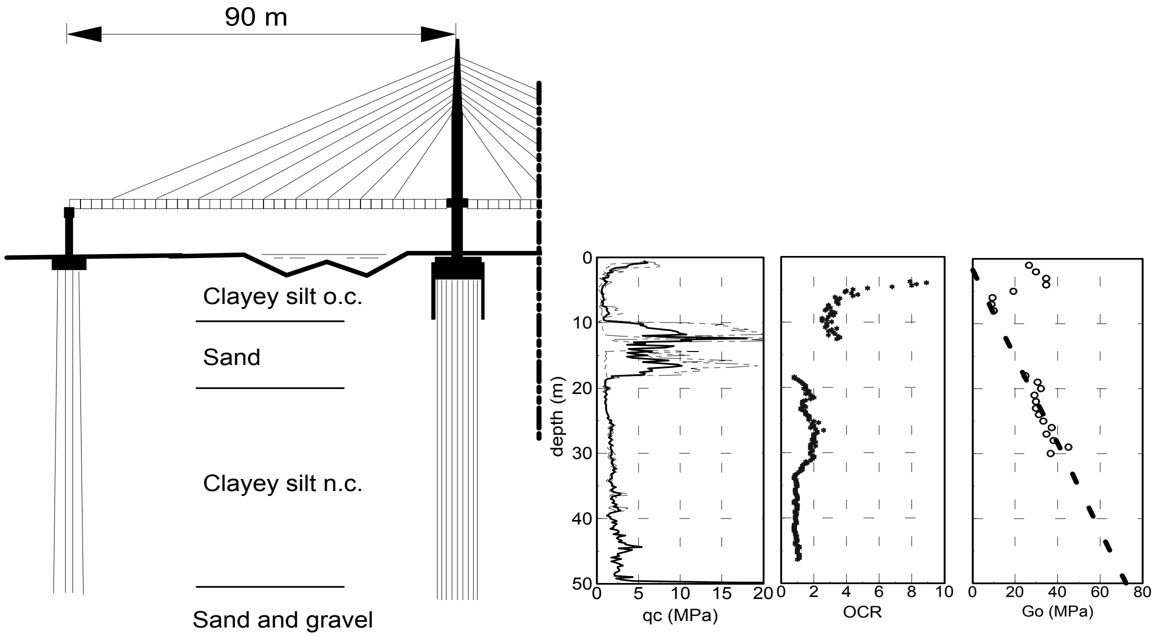

Figure 12. Subsoil profile at the location of the main pier of the Garigliano bridge.

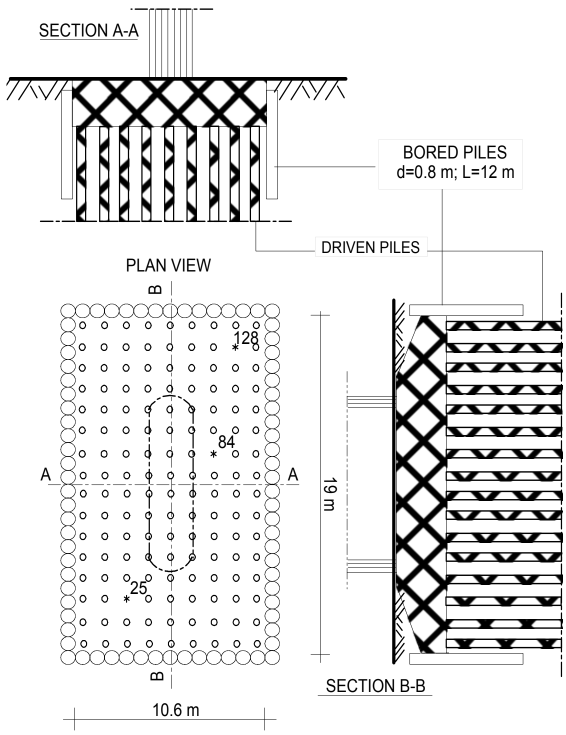

Figure 13. Layout of the foundation of the main pier of the bridge.

At the end of construction the measurements show a significant edge effect, as it was to be expected under a stiff cap, and some load concentration below the pier. Three years later the load distribution was undergoing significant variations: the load on the peripheral piles was decreasing, while that on the piles below the pier was slightly increasing. Ten years later this trend is still confirmed. To the writers' knowledge, such a phenomenon had not been observed before; the observed trend of variation suggests that the main factor is creep of the reinforced concrete raft.

Figure 16 reports the values of the load on some typical piles as a function of time, starting from the construction of the bridge deck in February 1993. While the total pile load keeps almost a constant value for the ten years after the end of construction (figure 14) the loads on the single piles undergo a cyclic variation, with a period of 1 year. The values reported in table 4 have been taken always in the same month of the year, in order to minimize the influence of the observed cyclic behaviour.

Table 4: Garigliano; load distribution among the piles vs. the time

	Corner piles	Edge piles	Internal piles	Piles under the pier
End of construction	1.30	1.00	0.80	0.90
3 years later	1.16	0.96	0.90	0.98
10 years later	1.10	0.93	0.94	1.03

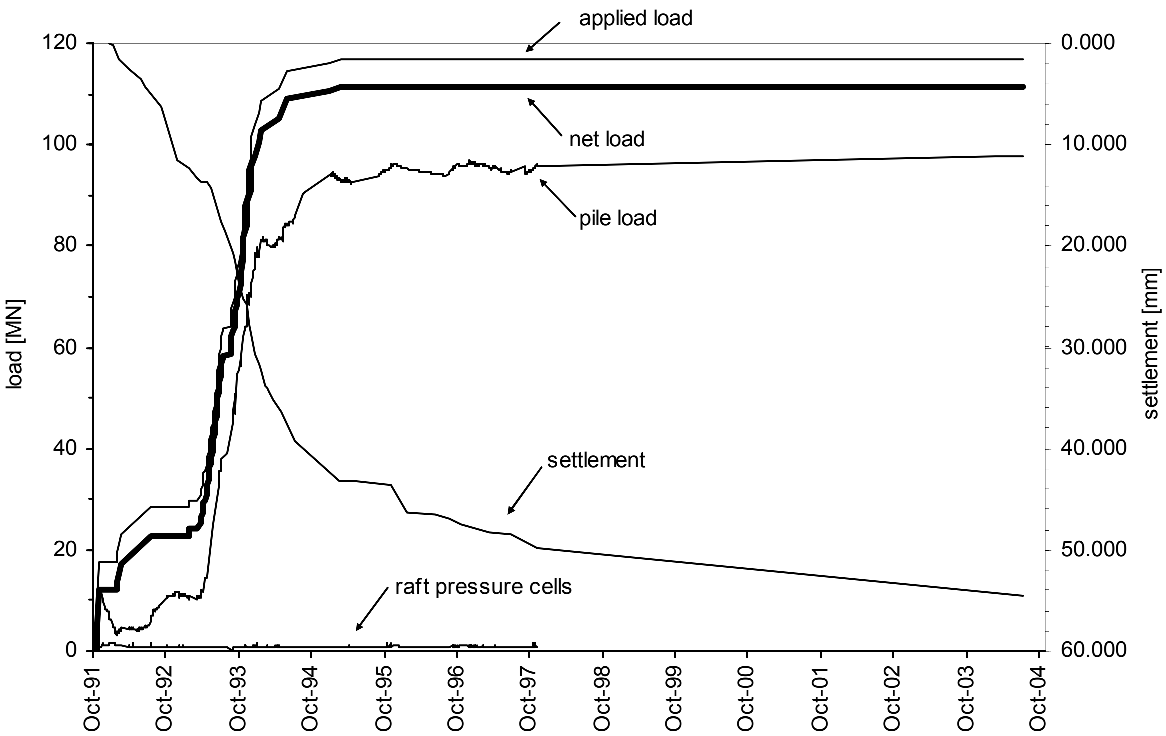

Figure 14. Total applied load compared to observed load sharing and measured settlement for the foundation of the main pier of the Garigliano Bridge.

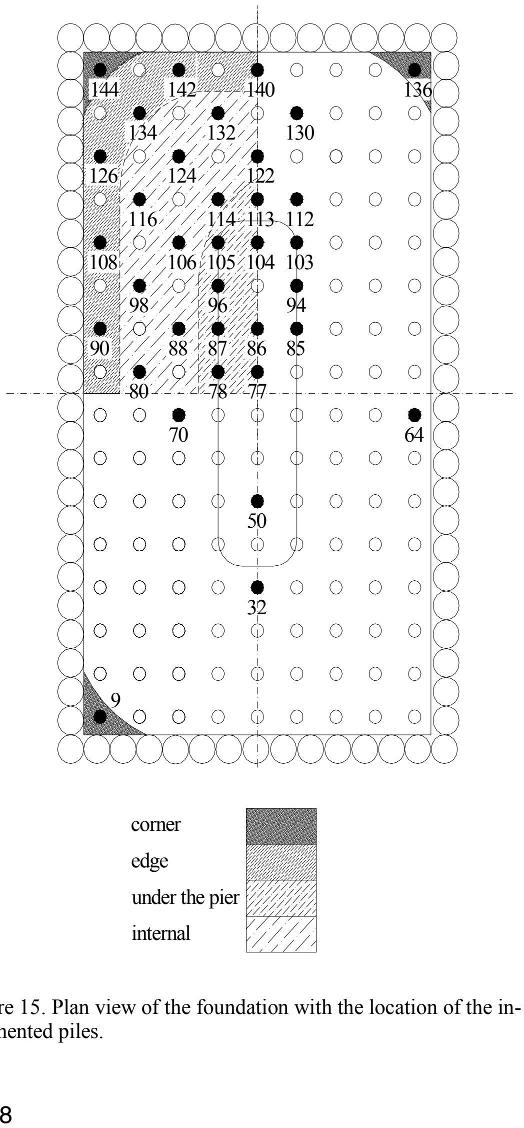

Figure 15. Plan view of the foundation with the location of the instrumented piles.

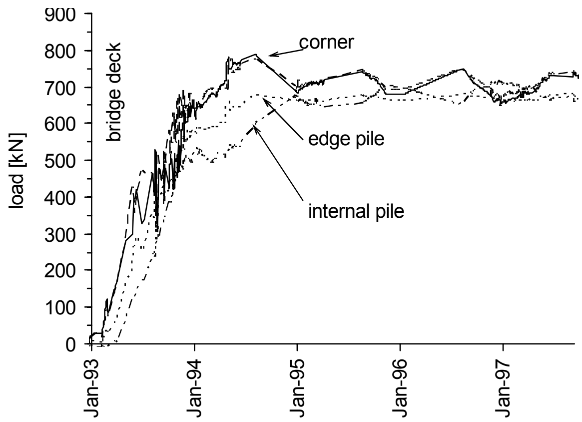

Figure 16. Load sharing among typical piles vs. time.

3.2 *Vertical loads*

3.2.1 *Settlement*

Mandolini *et al.* (1997) and Mandolini & Viggiani (1997) collected 22 well documented case histories of the settlement of piled foundations. The data base has been increased by Viggiani (1998) to 42 cases. The collection of further evidence brings now the total number of cases examined to 63; for all of them, besides the settlement records, load test on single piles and documentation on the subsoil and the construction are available. The main features of the case histories collected are listed in table 5. A wide range of pile types (driven, bored, CFA) assembled in a variety of geometrical configurations ($4 \leq n \leq 6500$; $2 \leq s/d \leq 8$; $13 \leq L/d \leq 126$) and regarding very different soils (clayey to sandy soils, stratified, saturated or not, etc.) are included.

The available measured settlement may be used as the basis for an entirely empirical evaluation of the expected absolute and differential settlement of a piled foundation.

The average settlement w of a piled foundation has been expressed as follows:

$$w = R_S \, w_S = n \, R_G \, w_S \tag{6}$$

where w_S is the settlement of a single pile under the average working load Q/n of the group (Q = total load applied to the foundation; n = number of piles), R_S is an amplification factor named group settlement ratio, originally introduced by Skempton *et al.* (1953) and representing the effects of the interaction between piles, and $R_G = R_S/n$ is the group reduction factor. The settlement of the single pile w_S is obtained by load tests on single pile. The group settlement ratio R_S has been expressed by Skempton *et al.* (1953), Meyerhof (1959), Vesic (1969) as a function of geometrical factors as the number n, the spacing s and the slenderness L/d of the piles.

On this empirical basis the following expressions for the upper limit $R_{S,max}$ and the best estimate of R_S, as a function of the aspect ratio $R = (ns/L)^{0.5}$ introduced by Randolph & Clancy (1993), have been found:

$$R_{S,max} = \frac{w_{max}}{w_S} = \frac{0{,}50}{R} \cdot \left(1 + \frac{1}{3R} \right) \cdot n \tag{7}$$

$$R_S = \frac{w}{w_S} = 0{,}29 \cdot n \cdot R^{-1{,}35} \tag{8}$$

Some of the case histories include information on the maximum differential settlement Δw_{max}; from these data the following relationship has been deduced:

$$R_{Dmax} = \frac{\Delta w_{max}}{w} = 0{,}35 \cdot R^{0{,}35} \tag{9}$$

Eqs. (7), (8) and (9), reported in figures 17, 18 and 19 ($R_S = n \cdot R_G$) , allow a preliminary evaluation of the maximum expected and the most probable values of the settlement as well as the maximum expected differential settlement.

More specific relationships for either different pile types (driven, bored, CFA, vibrodriven) or subsoil conditions (clayey, sandy, stratified) have been attempted but, in some way surprisingly, no better correlations have been found. Interaction among piles seems thus primarily controlled by pile group geometry (n, s, L, as expressed by the aspect ratio R). The properties of the subsoil and the influence of the pile installation enter the analysis via the value of w_S, obtained by a load test.

Some cases show a significant increase of the settlement after the end of the construction, due to primary consolidation in fine grained soils (Hooper, 1979; Katzenbach *et al.,* 2000) and creep in coarse grained soil (Mandolini & Viggiani, 1997). This aspect deserves some attention, being the long term settlement the most likely potential cause of damage to services, claddings and architectural finishes.

As pointed out by Poulos (1993) the relative amount of short term and long term settlement depends on the geometry of the foundation and the nature of the soil. Theoretical solutions show that immediate settlement accounts for about 93% of the final one for a single pile, decreasing to about 85% for a group of 25 piles.

Hooper & Wood (1977) compare a raft and a piled raft in London clay, in the same subsoil conditions. At the end of the construction the raft had settled about 50% of the final settlement while the settlement of the piled raft was very close to the final one. The data collected by Morton & Au (1974) for seven buildings on London clay show a ratio between the settlement at the end of construction and the final settlement ran ging between 0.4 and 0.7, irrespective of the foundation being piled or unpiled; in any case, the highest observed ratio is that of a piled foundation.

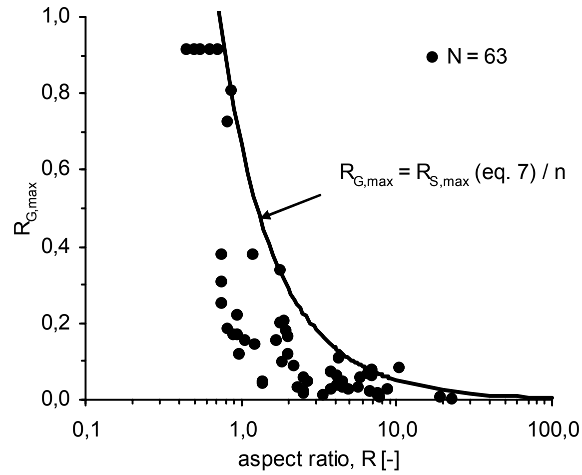

Figure 17. Relationship between $R_{G,max}$ and R.

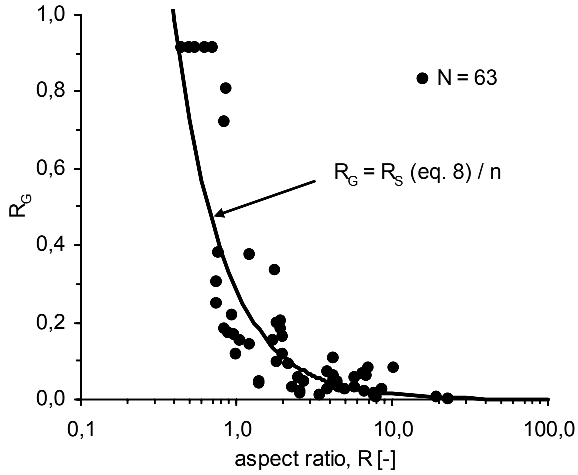

Figure 18. Relationship between R_G and R.

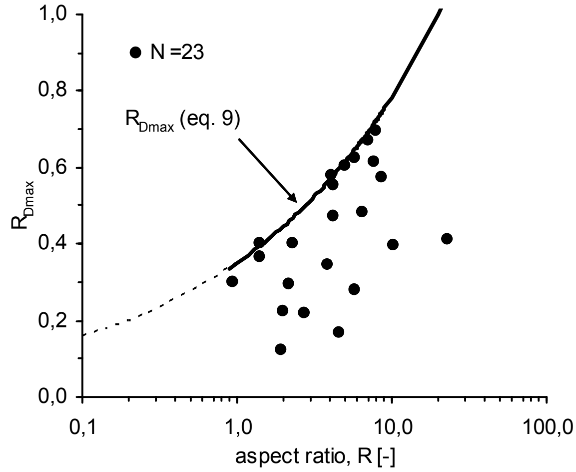

Figure 19. Relationship between R_{Dmax} and R.

Some case histories are summarized in table 6. It was decided to focus on two overconsolidated clays (London and Frankfurt) both for the sake of clarity and for the relatively large number of case histories available. In order to compare relatively homogenous data, the case histories are all referred to multi-storey framed buildings.

Table 5: Case histories of pile groups with settlement observations

Case	Reference	Pile type	n° of piles	d [m]	L [m]	s/d [-]	w_S [mm]	w [mm]	Δw_{max} [mm]
1	Vargas [1948]	D	317	0.50	11.6	3.5	0.8	16.0	-
2	Vargas [1948]	D	143	0.42	12.0	3.5	1.5	12.7	6.0
3	Feagin [1948]	D	239	0.34	11.7	2.9	2.7	28.7	-
4	Feagin [1948]	D	186	0.32	11.5	2.8	2.7	13.7	-
5	Vargas [1948]	D	205	0.42	12.0	3.5	2.2	11.6	7.0
6	Veder [1961]	B	104	0.53	25.0	3.0	11.4	24.0	-
7	Veder [1961]	B	104	0.53	25.0	3.0	11.4	19.0	-
8	Veder [1961]	D	24	0.53	25.5	3.9	9.8	11.0	4.0
9	Veder [1961]	D	24	0.53	25.5	3.9	9.8	10.0	4.0
10	Colombo & Failla [1966]	D	4	0.50	13.0	5.0	3.1	10.0	-
11	Koizumi & Ito [1967]	D	9	0.30	5.6	3.0	2.0	6.7	-
12	Calabresi [1968]	B	638	0.42	17.4	3.0	1.8	21.0	-
13	Komornik et al. [1972]	B	61	0.40	11.0	8.1	2.8	7.6	4.2
14	Koerner & Partos [1974]	B	132	0.41	7.6	6.9	6.2	64.0	43.0
15	Trofimenkov [1977]	D	7	0.34	4.5	6.0	2.0	4.7	-
16	Trofimenkov [1977]	D	6500	0.40	14.0	2.9	4.0	31.5	13.0
17	Trofimenkov [1977]	D	2016	0.34	5.5	2.9	3.2	31.0	-
18	Trofimenkov [1977]	D	9	0.40	12.0	3.0	2.6	5.0	-
19	O'Neill et al. [1977]	D	9	0.27	13.1	3.0	3.5	9.4	-
20	Clark [1978]	D	132	0.58	10.7	2.5	3.3	46.0	-
21	Brand et al. [1978]	D	4	0.15	6.0	5.0	1.0	3.8	-
22	Brand et al. [1978]	D	4	0.15	6.0	4.0	1.0	3.8	-
23	Brand et al. [1978]	D	4	0.15	6.0	3.0	1.0	3.8	-
24	Brand et al. [1978]	D	4	0.15	6.0	2.5	1.0	3.8	-
25	Brand et al. [1978]	D	4	0.15	6.0	2.0	1.0	3.8	-
26	Brand et al. [1978]	D	4	0.15	6.0	5.0	1.0	4.2	-
27	Brand et al. [1978]	D	4	0.15	6.0	4.0	1.0	4.2	-
28	Brand et al. [1978]	D	4	0.15	6.0	3.0	1.0	4.2	-
29	Brand et al. [1978]	D	4	0.15	6.0	2.5	1.0	4.2	-
30	Brand et al. [1978]	D	4	0.15	6.0	2.0	1.0	4.2	-
31	Bartolomey et al. [1981]	-	464	0.34	11.0	4.1	10.0	82.0	-
32	Bartolomey et al. [1981]	-	192	0.40	21.0	3.3	8.0	19.0	-
33	Bartolomey et al. [1981]	B	6	1.00	15.5	1.8	3.0	13.0	-
34	Cooke et al. [1981]	B	351	0.45	13.0	3.5	1.1	25.0	12.0
35	Bartolomey et al. [1981]	D	9	0.40	15.5	3.0	3.0	5.0	-
36	Thorburn et al. [1983]	D	55	0.28	27.0	7.0	4.6	29.5	6.6
37	Thorburn et al. [1983]	D	97	0.28	30.0	7.1	4.6	25.0	-
38	Kaino & Aoki [1985]	B	5	1.00	24.0	2.8	2.0	3.8	-
39	Viggiani [1989]	B	136	1.50	30.0	2.5	1.2	5.9	3.4
40	Marchetti [1989]	VD	54	0.35	18.0	2.8	0.6	4.9	-
41	Briaud et al. [1989]	D	5	0.27	9.2	3.9	2.0	2.5	-
42	Caputo et al. [1991]	B	241	2.00	42.0	2.9	3.7	28.1	17.5
43	Goossens & Van Impe [1991]	D	697	0.52	13.4	4.0	3.2	185.0	73.0
44	Mandolini & Viggiani [1992b]	CFA	637	0.60	20.0	4.0	1.7	26.4	15.1
45	Randolph & Clancy [1994]	B	27	0.80	20.0	3.5	5.0	24.5	3.0
46	Randolph & Clancy [1994]	B	38	0.80	20.0	3.5	19.4	22.5	9.0
47	Rampello [1994]	B	768	1.20	53.0	3.6	0.8	3.6	2.5
48	Russo [1994]	D	144	0.38	48.0	3.0	2.3	42.0	-
49	Mandolini [1994]	D	16	0.38	45.0	6.0	0.7	1.8	-
50	Mandolini [1994]	D	18	0.38	45.6	6.2	0.7	2.0	-
51	Mandolini [1994]	D	20	0.38	41.7	5.4	0.3	0.7	-
52	Mandolini [1994]	D	24	0.38	45.6	5.6	0.7	2.4	-
53	Randolph & Clancy [1994]	B	150	0.80	20.0	3.5	8.1	35.9	6.0
54	Rampello [1994]	B	74	1.20	56.8	3.1	0.8	5.4	1.6
55	Mandolini [1995]	B	16	0.80	23.0	2.4-3.0	0.8	1.8	1.1
56	Brignoli et al. [1997]	B	196	1.20	43.0	2.7	0.8	11.8	4.1
57	Mandolini & Ramondini [1998]	B	12	0.50	10.0	3.0	1.4	6.6	4.5
58	Tejchman et al. [2001]	D	264	0.50	13.5	3.5	1.05	15.9	4.4
59	Tejchman et al. [2001]	D	72	0.40	17.6	4.5	1.15	3.7	0.8
60	Tejchman et al. [2001]	B	292	1.00	26.5	5.4	2.4	14.6	9.0
61	Present report	CFA	13	0.60	11.3	5.8	9.0	19.0	-
62	Present report	CFA	13	0.60	11.3	4.8	4.8	12.5	-
63	Not published	CFA	13	0.60	11.3	5.3	5.7	15.0	-

Pile type: D = driven; B = bored; CFA = continuous flight auger; VD = vibrodriven

Table 6: Case histories with observation of the settlement vs. time

Case	Reference	Structure	Foundation Type	w_{eoc}[mm]	w_{fobs} [mm]
1	Morton & Au (1974)	Hurley House	Raft	50.0	104.6
2	Hooper & Levy (1981)	Island Block	Piled raft	15.0	27.0
3	Cooke et al. (1981)	Stonebridge park	Piled raft	11.0	18.0
4	Morton & Au (1974)	Cambridge road	Piled raft	17.0	23.1
5	Hooper (1979)	Hide Park Cavalry Barracks	Piled raft	16.0	21.0
6	Breth & Amann (1974)	Average of six cases	Rafts	-	-
7	Katzenbach *et al.* (2000)	Messe Torhaus	Piled raft	70.0	150.0
8	Poulos (2000b)	Messe Turm	Piled raft	85.0	115.0
9	Katzenbach *et al.* (2000)	Westend 1 – DG Bank	Piled raft	85.0	110.0

In figure 20 the ratio between the settlement measured at the end of construction, w_{eoc}, and the settlement measured at the end of the observation period, w_{fobs}, is plotted versus the ratio between the length of the piles L, and the width of the pile group B; the data reported for L/B = 0 refer to raft foundations.

In evaluating these data, it is to remind that the settlement at the end of construction probably includes some consolidation settlement, and conversely the settlement at the end of the observation period is probably smaller than the true final settlement. In any case, moving from raft to piled foundations the settlement ratio increases; for the same subsoil, the higher the ratio L/B the higher the settlement ratio. The only exception to this trend is the case of Torhaus; for this case the apparent anomaly could be explained by the very fast construction (figure 21), compared to the other case histories.

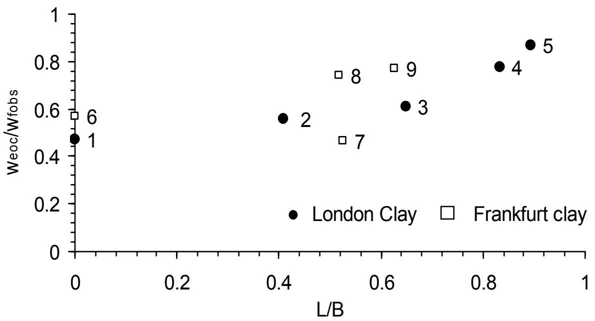

Figure 20. Ratio between settlement at the end of construction and settlement at the end of the observation vs. L/B.

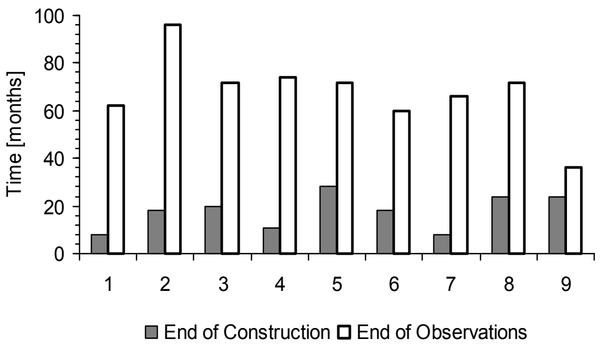

Figure 21. Duration of construction compared to duration of observation.

For the pier of the Garigliano bridge, resting on relatively soft clays, the immediate settlement of a raft should be in the range 10% to 20% of the final one. On the contrary the actual piled raft, with L/B = 4, exhibits a ratio w_{eoc}/w_{fobs} = 70%. For the Naples Law Court Building (Mandolini & Viggiani, 1997), founded on pyroclastic soils, w_{eoc}/w_{fobs} = 55% with a ratio L/B just below unity.

In this case both the construction time (6 years) and the observation interval (14 years) have been rather long.

Poulos (1993) claims that there are no theoretical solutions available for the rate of consolidation of pile groups. Numerical analyses of an impermeable block equivalent to the pile group indicate that the consolidation rate decreases with increasing the length to diameter ratio of the equivalent block. This result implies that the rate of consolidation of a shallow foundation is faster than that of a pile group. Available experimental evidence does not confirm this trend; on the contrary, the time to the final settlement seems independent of the type of foundation.

Further data with accurate long term settlement observations are needed to confirm the outlined trends.

3.2.2 *Load sharing and distribution*

A structure, its foundation and the surrounding ground interact with each other whether or not the designers allow for this interaction (Burland, 2004). The load sharing between the piles as a group and the raft is a fundamental quantity in the advanced design methods and in the new codes about piled raft foundations, in order to make the right use of the cooperation of the two elements. The load distribution among piles is a more complex issue, being markedly affected by the natural soil heterogeneity and the unavoidable pile variability (Evangelista *et al.,* 1977). Unfortunately, the bending moment and shear in the raft are strictly depending upon such distribution (Poulos *et al.*, 1997).

The experimental evidence on soil-structure interaction, either by small scale tests or monitoring of full scale structures, is much less than that available for settlement. Some data on load sharing between raft and piles and load distribution among piles, however, are gradually accumulating. In contrast to the 63 well documented case histories available on settlement (table 5), after a careful review of the literature only 22 sufficiently well documented case histories of soil-structure interaction have been found and are listed in table 7.

This experimental database will be used in the following to highlight some typical aspects of the observed behaviour.

About the load distribution among the piles, the available data reported in figure 22 come from cases with large differences in the type of subsoil but all characterised by a rather stiff foundation structure and/or superstructure.

An overall trend of increasing load on corner and edge piles with decreasing pile spacing can be recognised in figure 22. At the ordinary spacing of 3 diameters the ratio of the corner to centre pile load shows a large scatter but is definitely above unity, ranging from 1.5 to 3. This is an effect of the interaction among the piles; as the spacing increases the interaction decreases and the effect tends to vanish.

The pier of Garigliano Bridge, which is characterized by a relative stiff raft (figure 13) and no significant stiffening contribution by the superstructure, shows a long term smoothing effect (table 4 and figure 16). Any generalisation of this effect on experimental basis, however, is not yet possible because other long term observations of load distribution are not available.

Table 7: Case histories with observations of the load sharing

Case	Reference	Structure	s/d [-]	A_g/A [-]	raft load [%]	L/B [-]
1	Van Impe & De Clerq (1994)	Multispan bridge	3.8	0.70	27	1.00
2	Yamashita *et al*. (1993)	Building Urawa	8.0	0.90	51	0.64
3	Cooke *et al.* (1981)	Stonebridge park	3.6	0.90	23	0.65
4	Sommer *et al.* (1991)	Messe Turm	6.4	0.83	45	0.52
5	Joustra *et al.* (1977)	Apartament block	5.2	0.90	22	0.70
6	Hight & Green (1976)	Dashwood house	3.0	0.90	19	0.50
7	Jendeby (1986)	House 1	6.5	0.90	8	2.10
8	Jendeby (1986)	House 2	10.5	0.90	66	2.20
9	Jendeby (1986)	Uppsala house	11.2	0.90	64	2.20
10	Russo (1996)	Garigliano bridge	3.0	0.88	20	4.50
11	Katzenbach *et al.* (2000)	Messe Torhaus	3.5	0.80	20	1.14
12	Katzenbach *et al.* (2000)	Westend 1 – DG Bank	6.0	0.52	50	0.63
13	Katzenbach *et al.* (2000)	Japan Centre	5.5	0.45	60	0.60
14	Katzenbach *et al.* (2000)	Forum	6.0	0.55	62	0.70
15	Katzenbach *et al.* (2000)	Congress Centre	5.8	0.62	60	1.00
16	Katzenbach *et al.* (2000)	Main Tower	3.3	0.70	15	0.50
17	Katzenbach *et al.* (2000)	Eurotheum	5.2	0.55	70	0.80
18	Katzenbach *et al.* (2000)	Treptowers	6.5	0.86	52	0.38
19	Hooper (1979)	National Westimnster Bank	3.8	0.91	29	0.50
20	Hooper (1979)	Hide Park Cavalry Barracks	4.3	0.72	39	0.90
21	Present report	Tank 12 Harbour Napoli	5.8	0.82	50	0.92
22	Present report	Tank14 Harbour Napoli	5.0	0.82	46	1.10

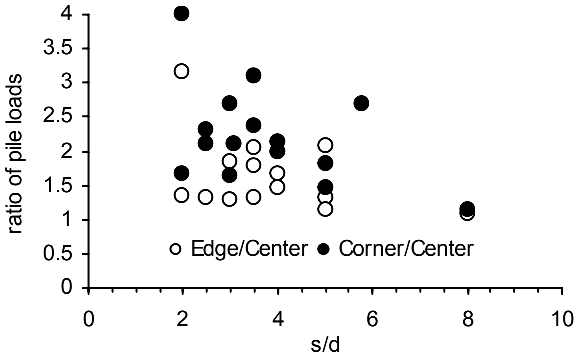

Figure 22. Load distribution among piles as a function of their location.

About the load sharing between the raft and the group of piles, the data reported in figure 23 come from only 11 out of 22 cases of table 7, and refer to foundations with piles more or less uniformly spread underneath the whole area of the raft ($A_g/A > 0.83$, where A is the area of the raft and A_g is the area of the pile group). The simple geometrical parameter s/d plays a major role in load sharing; the higher the spacing the higher the load taken by the raft.

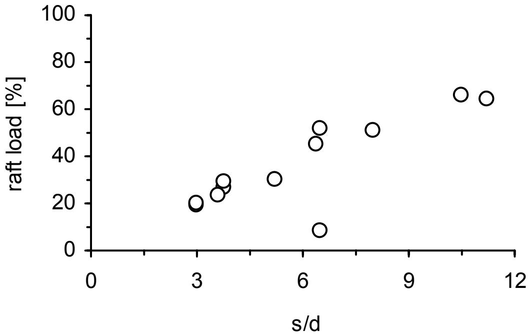

Figure 23. Load shared by the raft vs. spacing.

In figure 24 the plot is extended to all the 22 cases reported in table 7; the resulting relationship between the load sharing and s/d is not as close as it was in figure 23.

The added cases are generally characterized by piles concentrated in selected areas of the foundations. In figure 25 the load taken by the raft is plotted vs. the dimensionless parameter $(s/d)/(A_g/A)$; the load taken by the raft increases with increasing values of this parameter, becoming nearly constant for values below 4 or above 10.

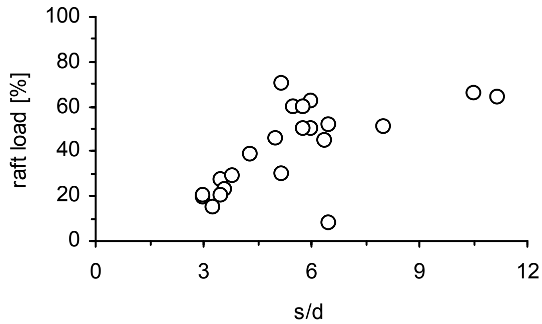

Figure 24. Load shared by the raft vs. spacing

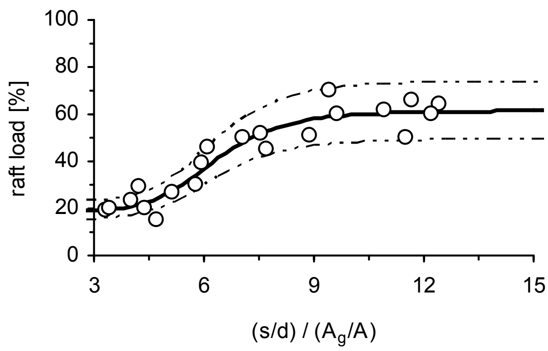

Figure 25. Load shared by the raft vs. spacing divided by the area ratio A_g/A.

3.2.3 *Bearing capacity*

A piled raft foundation consists of three elements: the raft, the piles and the subsoil. The load is equilibrated partly by the contact pressure between the raft and the soil and partly by the piles.

At failure, the bearing capacity of an unpiled raft Q_R may be evaluated by the conventional bearing capacity theory (Terzaghi, 1943; Brinch-Hansen, 1970; Vesic, 1973; Randolph *et al*., 2003). Collapse of the pile group may occur either by failure of the individual piles or as failure of the overall block of soil containing piles (Terzaghi & Peck, 1948). The axial capacity Q_P for individual pile failure is generally evaluated by:

$$Q_P = \eta \cdot \sum_{i=1}^{n} Q_{i,P} \tag{10}$$

where $Q_{i,P}$ is the bearing capacity of the i-th pile and η is a group efficiency factor depending on pile layout and type and soil type (Kezdi, 1957). Values for the efficiency η have been suggested by Whitaker (1957), Vesic (1969), De Mello (1969), Brand *et al.* (1972), O'Neill (1982), Briaud *et al.* (1989).

When considering the bearing capacity Q_{BF} by failure of the overall block of soil, it is generally assumed that the full shear strength of the soil is mobilised on the vertical surfaces of the block defined by the perimeter of the piles, as well as the bearing pressure at the base of the block. A suitable factor of safety FS should be provided against both modes of failure, taking into account that the settlement needed to mobilize the base capacity of the block is of the order of 5% to 10% of its width (Cooke, 1986). Since the end-bearing pressure q_B in granular soils is much greater than the average skin friction q_S (typically q_B/q_S ranges between 50 and 200), Fleming *et al.* (1992) claim that block failure may occur only when the base area is many times smaller than the side area. Groups consisting of closely spaced long piles are thus more likely to fail as a block than groups consisting of short piles at the same spacing. Such conclusion is consistent with the experimental data collected by a number of researchers (e.g. Vesic, 1969; Liu *et al.*, 1985; Ekstrom, 1989; Phung, 1993).

De Mello (1969) summarized data for pile groups up to 9^2 in clay soils; the block mode of failure occurs for spacing smaller than 2 to 3 pile diameters. Similar results have been reported by Cooke (1986).

Taking into account all the above evidence Poulos (2000b) suggested to estimate the vertical bearing capacity Q_{PR} of piled rafts as the smaller of the following values:

- the ultimate capacity Q_{BF} of the block containing the piles, plus that of the portion of the raft outside the periphery of the pile group;
- the sum of the ultimate loads of the raft Q_R and of all the piles Q_P in the system:

$$Q_{PR} = Q_R + Q_P \tag{11}$$

the latter having been proposed by Liu *et al.* (1985).

The installation of the piles, however, may affect the soil properties and consequently modify the performance of the raft in comparison with that of the unpiled raft. Moreover, it is becoming more and more evident that the behaviour of the piles belonging to a piled raft is affected not only by the interaction among piles but also by the surcharge exerted by the raft. As a consequence Liu *et al.* (1994) and Borel (2001a) suggested to modify eq. (11) as follows:

$$Q_{PR} = \alpha_R \cdot Q_R + \alpha_P \cdot Q_P \tag{12}$$

where α_R and α_P are coefficients affecting the failure load of the raft and the pile group when combined in a piled raft.

The available experimental evidence is reviewed in the following, to assess the likely values of the coefficients α_R and α_P and elucidate the factors affecting them.

The curves reported in figure 26 represent the theoretical failure load of a pile group, assuming either "block" failure or "pile group" failure, as a function of the pile spacing. Below a critical value s_{crit}/d of the spacing ratio (s_{crit}/d increases from about 2.5 for 3^2 piles to about 3.5 for 9^2 piles, bold line in figure 26) the failure load Q_{BF} corresponding to block failure is the smallest one and block failure should thus occur; at $s/d > s_{crit}/d$, the failure load of pile group Q_P should apply.

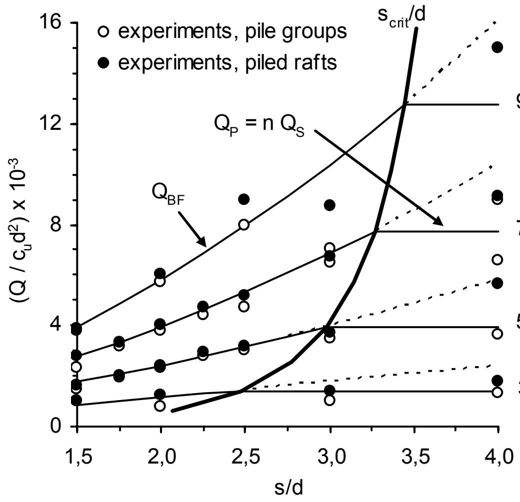

Figure 26. Experiments by Cooke (1986) on pile groups and piled rafts with L/d = 48.

Cooke (1986) summarized the results of a broad laboratory investigation including load tests on model rafts, pile groups and piled rafts, founded on remoulded London clay with an undrained shear strength c_u ranging between 5 and 15 kPa. The tested piles had a ratio L/d = 24 and 48 and were arranged in 3^2, 5^2, 7^2 and 9^2 groups. Some results are shown in figure 26 for the case L/d = 48.

The experimental data for pile groups are in good agreement with the theoretical curves; the bearing capacity of the pile group Q_P is equal to Q_{BF} for $s < s_{crit}$, while $Q_P = n \cdot Q_S$ for $s > s_{crit}$. On the other hand the values of the bearing capacity of piled rafts Q_{PR}, at a measured settlement w = 10%B, fit the curves corresponding to block failure, for values of s/d both below and above s_{crit}/d. By introducing the coefficient:

$$\zeta_{PR} = \frac{Q_{PR}}{Q_P} \tag{13}$$

the experimental results can be summarized as follows:

- for $s/d < s_{crit}/d$, $Q_{PR} \sim Q_{BF} \sim Q_P \Rightarrow \zeta_{PR} \sim 1$
- for $s/d > s_{crit}/d$, $Q_{PR} \sim Q_{BF} > Q_P \Rightarrow \zeta_{PR} > 1$

The critical spacing ratio s_{crit}/d is generally defined as that value below which block failure occurs for pile groups. It may be actually better viewed as that value above which the raft either transfers part of the load directly to the soil or enforces block failure for the pile group. Therefore, ζ_{PR} may be assumed as a measure of the increase of bearing capacity due to raft-soil contact.

Figure 27 shows that ζ_{PR} increases with increasing spacing ratios and with decreasing number of piles; at s/d = 4, it is about 1.7 for 3^2 and 1.25 for 9^2 piles. For the case L/d = 24 at s/d = 4, Cooke's experiments give $\zeta_{PR} \sim 2.1$ for 3^2 and $\zeta_{PR} \sim 1.9$ for 9^2 piles.

All the experimental values of ζ_{PR} obtained by Cooke (1986) are reported in figure 28 as a function of the ratio s/s_{crit}, in the range $s/s_{crit} > 1$. The value of ζ_{PR} increases with increasing s/s_{crit}; with a conservative design approach, it could be assumed in clay soils that $\zeta_{PR} = s/s_{crit}$ (broken line in figure 28).

Sales (2000) reports a field investigation on the behaviour of piled foundations in the clay of Brasilia. Only the four load tests carried out on undisturbed soil (table 8) are considered here. The settlements attained during the tests range between ~ 20 mm for test II and ~ 45 mm for test IV. In any case, the piled raft (test VI) attained a settlement not larger than 3%B, significantly smaller than those reported by Cooke (1986).

Comparing the results of single pile (test II) and 2^2 pile group (test IV), an efficiency factor η=1 is derived; moreover, the sum of the ultimate loads for pile group (test IV) and raft (test I) is 450 kN, that means 12.5% greater than the observed value for the piled raft (test VI, 400 kN). In terms of coefficient ζ_{PR} the experiments yield a value of 400/300 = 1.33.

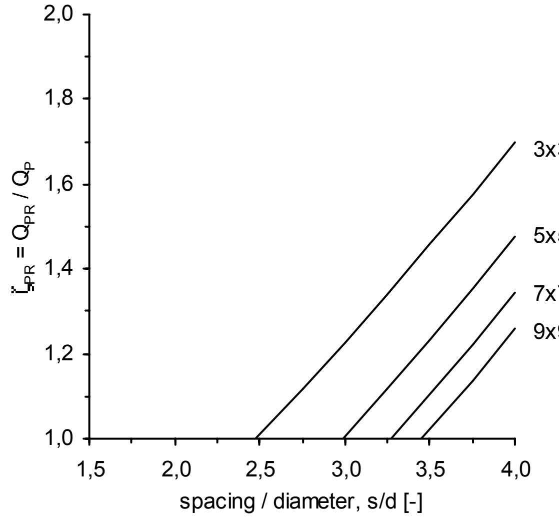

Figure 27. Relationship between the increase of bearing capacity and the increase of spacing ratio.

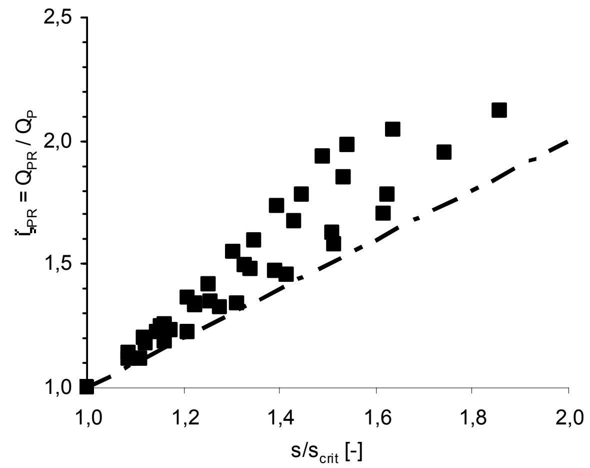

Figure 28. Relationship between ζ_{PR} and s/s_{crit}.

Table 8: Summary of the load tests reported in Sales (2000)

Test	Foundation	Q_{max} [kN]	w_{max} [mm]
I	Square raft, B_R = 1 m	150	28.6
II	Single pile, L=5m; d=0.15m	75	20.0
IV	2^2 Pile group, (s=5d)	300	45.4
VI	2^2 Piled raft, (s=5d)	400	27.7

At maximum settlement, the piles carried about 70% of the total load (280 kN), the raft the remaining 30% (120 kN). Assuming these values as the final ones, eq. (12) gives: α_R = 0.80 and α_P = 0.93.

Borel (2001a) reports a full scale load test on a capped pile in stiff clay. The cap is a concrete circular raft, 2 m diameter and 0.5 m thick; the closed-end steel displacement pile has d = 0.45 m, L = 12.2 m. Under the maximum applied load of 2.25 MN a settlement of about 225 mm, i.e. ~ 11% of the cap diameter, was attained. At this stage, Q_P/Q_{PR} = 59%; Q_R/Q_{PR} = 41%. By comparing the load carried by the pile (~ 1330 kN) with its failure load when isolated (Q_S ~ 1200 kN), a value α_P ~ 1.1 is found. In the same way, for the circular cap (Q_R = 1000 kN) a value α_R ~ 1.1 is derived. In terms of coefficient ζ_{PR} the experiment yields a value of 2250/1200 ~ 1.9.

Conte (2003) and Conte *et al.* (2003) report the results of an extensive series of centrifuge tests in fine grained soils on single piles (at prototype scale, L = 9 and 18 m; d = 0,63 m), unpiled square rafts (B = 9 and 18 m), pile groups (3^2 and 7^2 piles, s/d = 4) and piled rafts, obtained by a number of combinations of the above components. Five series of tests were carried out to measure the response of the components up to very large settlement (for single piles and pile groups, at least 80%d; for unpiled and piled rafts, not less than 10%B). Apart a few cases, a punching type failure has been systematically observed in the tests, i.e. the load continuously increases as the settlement increases. The analysis of the experimental results is thus strongly affected by the displacement level assumed to represent failure.

The values of the coefficient ζ_{PR} obtained for two series of tests at final settlement are reported in figure 29 as a function of the parameter R_M defined as:

$$R_M = \sqrt{\frac{n \cdot s}{L}} \cdot \left(\frac{A}{A_g}\right) = \frac{R}{\left(\frac{A_g}{A}\right)} \tag{14}$$

For piled rafts with piles spread below the whole raft, A_g/A is close to unity and R_M approaches the aspect ratio R as defined by Randolph & Clancy (1993); on the contrary, for piles concentrated in some region of the raft (for instance in the central zone), A > A_g and, consequently, R_M > R.

For piled rafts with A/A_g ~ 1, ζ_{PR} at large settlement attains values from ~2 to ~4. For A/A_g ~ 4, ζ_{PR} significantly increases, attaining a value of about 10. All these values of ζ_{PR} correspond to values of α_P and α_R close to 1.

Similar results have been obtained by centrifuge tests carried out on model piled rafts in granular soils (LCPC, 1998).

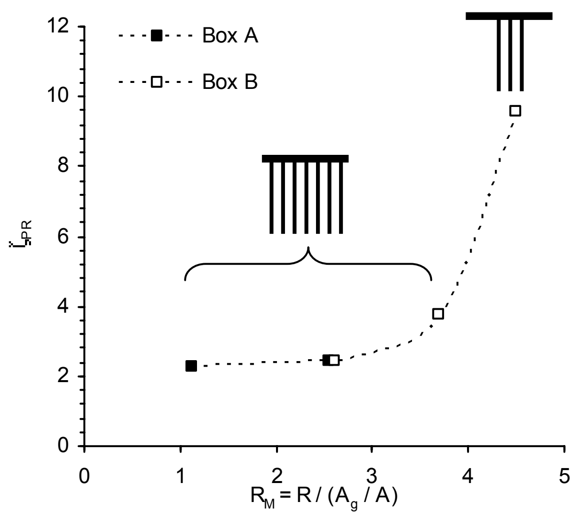

Figure 29. Relationship between ζ_{PR} and R_M.

Summing up the available experimental evidence, the assumption that the failure load of a piled raft with pile spacing above some critical value is equal to that of the pile group alone (α_R = 0 and α_P = 1 in eq. 12) appears overly conservative. The contribution of the raft to the ultimate capacity is always positive (ζ_{PR} > 1); a fraction α_R of the ultimate value for the unpiled raft Q_R can be definitely considered. For settlement of the order of some percent (say 5% to 10%) of B, the collected experimental evidence yields values for α_R approaching unity; being such a settlement too large for practical purposes, even for small rafts, later on (§ 4.1.5) the results of some numerical analyses will be reported allowing to explore which values of α_R and ζ_{PR} may be expected at smaller settlement.

3.3 *Horizontal loads*

Much alike piles under vertical loads, the response of a laterally loaded pile group with relatively closely spaced piles is quite different from that of a single pile, because of the interaction between piles through the surrounding soil, the rotational restraint exerted by the cap connecting the piles at the head, the additional resistance to lateral load provided by frictional resistance at the cap-soil interface, and passive resistance if the structure is totally or partially embedded.

The experimental evidence is rather scanty, compared to that available for vertical load, and is mainly related to the first two items. Relatively small groups have been tested, typically small scale models and full scale foundations with 2 to 16 piles. In recent years the use of centrifuge allowed the study of slightly larger groups (16 to 21 piles).

A comprehensive review of the experimental evidence is reported by Mokwa (1999). Valuable experimental investigations have been recently added (Remaud *et al.*,1998; Borel, 2001a, 2001b; Rollins & Sparks, 2002; Ilyas *et al.*, 2004; Rollins *et al.*, 2005).

Prakash & Saran (1967), Alizadeh & Davisson (1970), Matlock *et al.* (1980), Schmidt (1981, 1985) conducted a number of full scale and 1g model tests. In most cases only the load displacement relationship for single free head piles and for both free and fixed head pile groups was recorded; the experimental findings appeared generally compatible with the framework of the analytical tools available at that time.

Later centrifuge (Barton, 1984) and full scale tests (Ochoa & O'Neill, 1989) revealed that for a given horizontal load parallel to the columns of a group, calling trailing row the first row of the group while the last one is the leading row (figure 30), the leading piles carry more load than the trailing ones even at load levels far from failure.

Selby & Poulos (1984) measured shears and bending moments in the leading piles larger than that in the central and trailing piles in 1g model tests; they called this effect "shielding". Brown *et al.* (1988) observed the same effect in a full scale test on a 3^2 pile group and introduced the term "shadowing" to mean the phenomenon for which the soil resistance of a pile in a trailing row is reduced because of the presence of the leading pile ahead of it.

A rather large amount of experiments have been carried out in the last decade with the aim of deriving "general" rules to adapt p-y curves to account for group effects. Brown *et al.* (1988) introduced the concept of p-y multiplier, f_m, a multiplier of the p values capable of stretching the p-y curve for the single pile to account for the interaction among the piles in a group (figure 31). The multipliers have obviously values in the range 0 to 1. A major part of the latest experimental work has been devoted to the determination of the p-y multipliers. The latest experiments are at field scale or in centrifuge; the use of 1g small scale tests has been abandoned having recognized how misleading could be the obtained results in terms of stiffness.

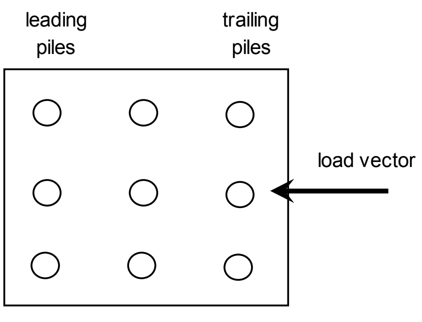

Figure 30. Leading and trailing piles for a given load vector.

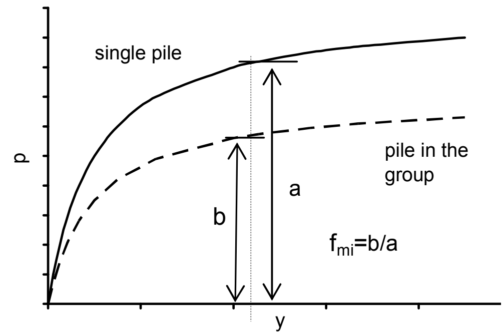

Figure 31. P-y multipliers for group effects.

The experiments usually attain rather large displacements, being oriented to extreme events. The p-y multipliers are obtained at the ultimate displacement reached during the test, and generally they are considered independent of the displacement or the load level. Field test displacements of the single pile and the pile group ranging between 10% and 15% of the pile diameter are rather usual. In centrifuge tests displacements as high as 25% to 50% of the pile diameter are typically attained.

The collected observations (Brown *et al.*, 1988; McVay *et al.*, 1998; Rollins *et al.*, 1998; Mokwa, 1999) seemed to suggest that:

- the multipliers can be defined for rows orthogonal to the direction of the load vector, being the differences among the piles in a row almost negligible;
- after the third leading row the same multiplier applies to the other rows, except the trailing one;
- the multipliers are independent of the soil type, pile type and load level, but depend essentially on the spacing;
- at a spacing above 6 to 8 diameters in the direction of the load vector, and 4 diameters in the orthogonal direction, the interaction among piles is negligible and the multipliers can be assumed equal to 1.

A parameter frequently used to compare the response of single pile and pile groups under horizontal load is the group efficiency:

$$G_e = \frac{\frac{H}{n}}{H_S} \tag{15}$$

where H is the total horizontal load applied to the group, n the number of piles in the group and H_S the horizontal load carried by a single pile at the same horizontal displacement. It is worth noting that, when dealing with piles under vertical load, the group effect is expressed through a multiplier R_S of the settlement at a given load; on the contrary for piles under horizontal load, a reduction of the load per pile at a given displacement is used. This reflects the fact that in the former case the emphasis is on displacements, in the latter the main design issue is the stress in piles.

The efficiency G_e can be easily expressed in terms of the p-y multipliers as follows:

$$G_e = \frac{\sum_{i=1}^{m} f_{mi}}{m} \tag{16}$$

where f_{mi} is the multiplier of the i-th row while m is the number of rows in the group. Assuming constant values for the p-y multipliers, irrespective of the load or the displacement level, the efficiency of the group G_e is also constant.

In figures 32 and 33 some experimental values of the efficiency G_e are plotted against the displacement normalised by the diameter of the pile. The data reported in figure 32 were obtained by field tests while those in figure 33 by centrifuge tests. In both cases the experiments were carried out under free head conditions for both the single piles and the pile groups.

The data reported in figure 34, on the contrary, were also obtained by centrifuge tests but the single pile was tested under free-head conditions while the pile group had a rotational restraint at the pile head. All the data reported in figure 32, 33 and 34 refer to pile groups with a constant spacing s = 3d.

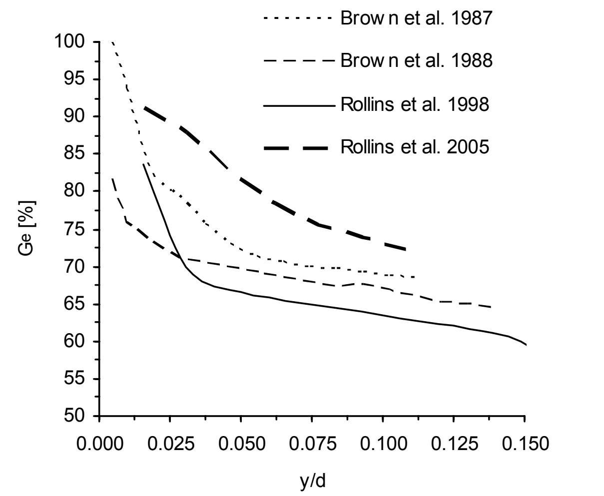

Figure 32. Efficiency G_e vs. displacement for field tests of small pile groups under horizontal load.

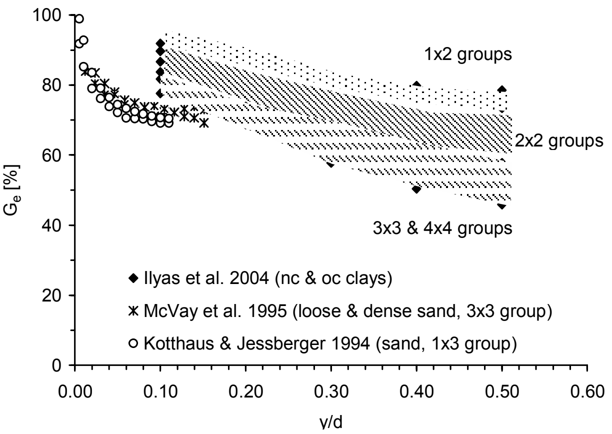

Figure 33. Efficiency Ge vs displacement for pile groups of different size under horizontal load (centrifuge tests).

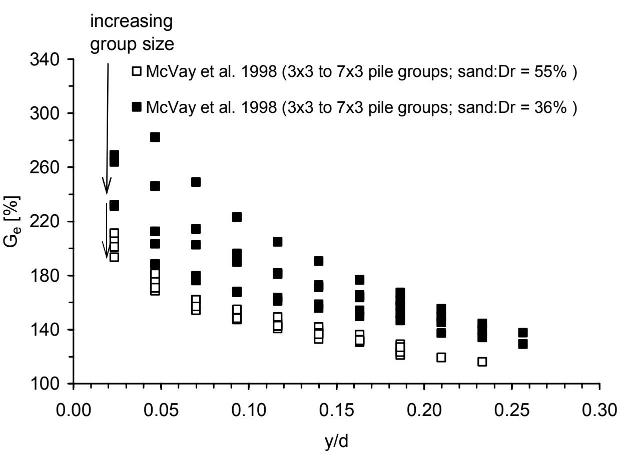

Figure 34. Efficiency Ge vs. displacement for pile groups of different size under horizontal load (centrifuge tests).

The efficiency G_e is always below unity and decreases with increasing displacement (figures 32 and 33). In figure 34 the efficiency is above unity, as expected, due to the rotational restraint at the head of the piles in group, but again G_e significantly decreases with increasing displacement. Figures 33 and 34 show a dependence of the efficiency G_e on the size of the group: the larger the group size, the lower the efficiency.

Being a widespread belief that the p-y multipliers can be assumed constant for each row and independent of the number of piles contained in the rows, the tests reported in figure 34 were carried out just increasing the number of rows and keeping constant the number of piles in each row.

To compare the behaviour of pile groups under horizontal and vertical load, the definition of efficiency G_e can be easily extended to pile groups under vertical load, just exchanging the shear H with the axial head force Q on top of the piles. In figure 35 the data provided by 3 field tests on small pile groups and 2 large pile groups under vertical loads are reported. As in the case of horizontal load, the efficiency under vertical load obviously decreases with increasing the size of the group. The effect of the displacement, on the contrary, is the opposite of that under horizontal load; in fact, the efficiency increases with increasing displacement. This is in substantial agreement with the widely accepted concept that the interaction among piles in a group under vertical load is essentially a linear phenomenon and is fully developed already at small displacement level, the non linearity of the single pile being concentrated at the pile soil interface and not amplified by group effects.

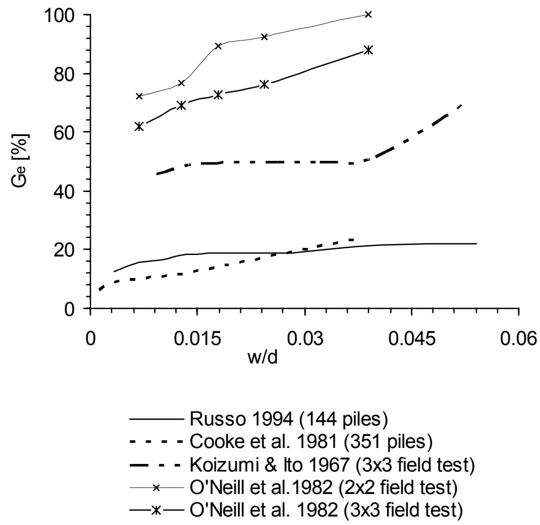

Figure 35. Efficiency G_e vs displacement for pile groups under vertical load.

On the contrary, under horizontal load the rather marked decrease of the efficiency with the increase of the displacement reveals a growing interactivity among the piles of the group. The interaction mechanisms under vertical and horizontal load are thus different.

The practice to fix a unique multiplier for each row is not so obvious; a summary of the available data on the load sharing among the piles in a group will be used to clarify the point. In all the experiments uniform settlement was imposed to the group and consequently the load is not uniformly distributed on piles.

Morrison & Reese (1986) carried out a field test on a 3^2 pile group in sand and reported a maximum difference between pairs of adjacent piles belonging to the same rows of about 33%, while for pairs belonging to the same column the difference was slightly above 100%.

The field test of a 4^2 pile group in sand carried out by Ruesta & Townsend (1997) revealed differences above 100% between piles in the same row and in the same column. McVay *et al*. (1998) performed centrifuge tests on groups of variable size and found differences between two adjacent piles in the same rows not always negligible and, sometimes, comparable to the differences between two adjacent rows. Similar results are reported by Ilyas *et al.* (2004).

Rollins *et al.* (2005) published the results of a field test on a 3^2 group at spacing of 3.3d. Substantial differences in the load sharing among piles in the same row were observed. The internal piles carried systematically the lowest load. The ratio between the centre and the outer pile loads in the same row is in the range 65% to 80%; the same range applies also for piles belonging to different rows. Even if the lower interactivity among piles placed orthogonal to the direction of the load vector compared to that among piles aligned in the direction of the vector is a widely accepted evidence, the above data show that the effects in a group are not at all negligible.

Being the bending moment in the piles more critical than the pile head deflection when designing piled foundations under horizontal load, it is also interesting to summarize the available experimental evidence on this item. The experimental results are affected by some scatter, probably due to the experimental difficulties and also to the detail of the rotational restraint imposed at the pile head. Some general trends can be however identified.

Data collected for the cases where both the single pile and the pile group were tested with free head conditions are reported in figure 36. The ratio between the maximum bending moment in different piles in a group and that in a single isolated pile is plotted against the displacement of the pile group. The ratio is evaluated at the same average load per pile. The data are rather scattered but in the majority of the cases the values of the moments in the piles belonging to the group are larger than the values in the single pile, the increase being a growing function of the displacement. Larger moments occur for the piles belonging to the Leading Row (LR) if compared either to the middle (MR) or the trailing piles (TR). This plot provides a valuable piece of information. It would be unsafe to approach the prediction of the moment in a pile within a group calculating the bending moment with the average load and the model of a single pile interacting with the soil.

This occurs for two reasons. The maximum bending moment in a pile subjected to a horizontal load at the head depends on: (i) the load and (ii) the depth needed for the pile to develop a sufficient reaction into the surrounding soil; the deeper the soil reaction the higher the bending moment with the same head load. In a pile group the applied load is not uniformly shared among the piles, being the load distribution a function of their position within the group. The higher moments of the leading piles in figure 36 are thus partially due to head loads higher than average. The interaction among the piles, furthermore, develops a deeper reaction in the surrounding soil, compared to what observed for a single isolated pile. This trend is more marked for the trailing piles than for the leading ones. This may well explain why in some cases differences in the observed maximum bending moments between leading and trailing piles are not very large, even being the head load on the leading piles much higher.

Kim & Sing (1979) tested both a free standing pile group and pile group with the cap in contact with the soil. The maximum bending moments observed in the piles were initially similar in the two experiments, probably due to a negligible mobilisation of the friction between the cap and the soil. At higher load level, however, the observed moments in the latter experiment were less than half those of the free standing group. Such an effect was likely due to the contribution of the friction between the cap and the soil, resulting in a decrease of the loads transmitted to the pile head. On the other hand Horikoshi *et al.* (2002), in a centrifuge test, observed that the cap – soil friction is fully mobilised at a displacement lower than that needed to mobilize the shear at the head of the piles. It is evident that further investigations on the effect of a cap in contact with the soil are badly needed.

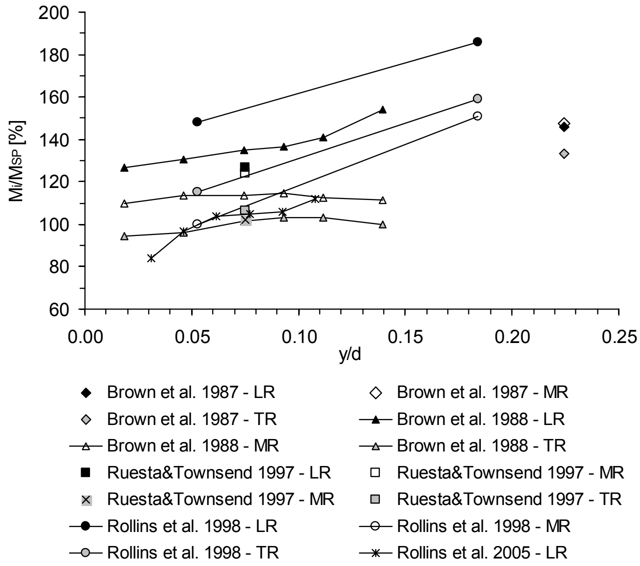

Figure 36. Ratio between bending moments of piles in a group and those in a single isolated pile vs. displacement.

4 PILE GROUPS AND PILED RAFTS; ANALYSIS

4.1 *Vertical loads*

4.1.1 *Model and reality*

Modern engineering is characterised by a design performed in the framework of scientific theories; it is tightly linked to the methodological structure of science and could not come into being without it. A scheme of the relationships between Science and Engineering is reported in figure 37 (Viggiani, 2001). A scientific theory, such as Euclidean geometry or thermodynamics or theory of elasticity, is characterized by two essential points:

- it does not deal with real objects, but with abstract entities specific to each theory: points, angles, segments; temperature and entropy; elastic half spaces;
- the structure of the theory is deductive. It consists of a small number of fundamental statements (axioms, or principles, or postulates) involving the above entities, and a universally accepted method to derive from them an endless number of consequences. All the problems that can be formulated within the framework of a theory can thus be solved by demonstration and calculus, and there is a general agreement on the solution among the scientists. In this sense, the truth of the scientific statements is warranted.

However, the application of a theory in engineering depends on correspondence relationships between the abstract entities of the theory and real objects. Unlike the statements which are internal to the theory, these relationships have no absolute validity; they have to be checked by experiments and in any case their validity is always limited.

Moving now from the heavens of science to the ground of foundation engineering, a similar scheme can be applied to the analysis and design of piled rafts. In recent years, a considerable research effort has been devoted to the procedures of analysis for the evaluation of the settlement of piled foundations and the study of soil-structure interaction under vertical loads. Generally soil, raft and piles have been modelled as elastic bodies, and their interaction analysed by numerical method, the most widely used being BEM (Poulos, 1968; Banerjee, 1970; Poulos & Davis, 1980; Banerjee & Butterfield, 1981; Basile, 1999). Different approximations have been introduced to curb the computational resources needed or to deal with non linearity, such as the interaction factors (Poulos, 1968) and so called hybrid methods (Chow, 1987). FEM are increasingly used with suitable constitutive models for the soil and the interfaces (Reul, 2000; Katzenbach *et al.*, 1997; de Sanctis, 2001; Potts & Zdravković, 2001), and it can be foreseen that, with increasing computational resources, the use of FEM will further spread out. As a result of this effort, entirely belonging to the realm of theory, quite a number of algorithms are by now available making the analysis relatively simple and straightforward.

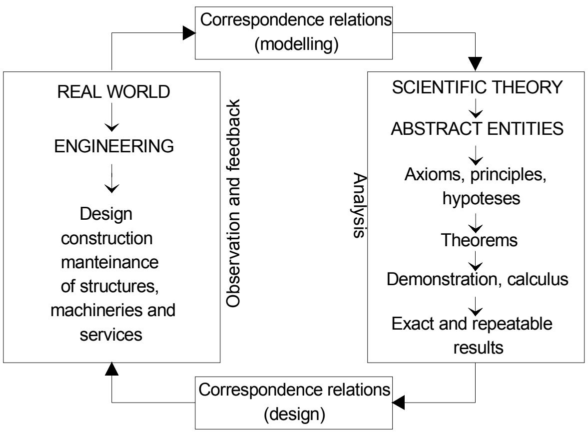

Figure 37. Relationships between Science and Engineering.

Less attention has been paid to the development and validation of suitable correspondence relations. In the more familiar terms of foundation engineering, this means: the subsoil model, the determination of parameters, the choice between a linear elastic (either tangent or secant) or a non linear analysis. Many Authors (among others: Poulos, 1972; Caputo & Viggiani, 1984; Randolph, 1994; El Mossallamy & Franke, 1997; Mandolini & Viggiani, 1992a, 1997; Viggiani, 2000) have addressed these topics, but their relevance seems to be not yet widely appreciated. In facts, some views about the reliability of the analysis and the need for further development of the procedures (Goossens & Van Impe, 1991; Poulos, 1993; Tejchman *et al.,* 2001) can probably be corrected by a proper consideration of these factors.

In the opinion of the Authors, the available procedures of analysis may be considered satisfactory for engineering purposes provided they are properly applied, paying due attention to the correspondence relations.

4.1.2 *Evaluation of soil properties and implementation of the analysis*

The elastic properties of the soil, to be used in the analysis, are difficult to evaluate because of the marked non linearity of the stress–strain relation and the influence of pile installation. Some Authors have suggested utilising to this aim the results of pile load tests (Poulos, 1972; Mandolini & Viggiani, 1997; Mandolini *et al.,* 1997). Such tests are often available for important projects; should not this be the case, the load – settlement behaviour of a single pile can be simulated for instance by the transfer curves approach.

In order to reduce the uncertainties connected to the choice of the parameters, a standard procedure has been developed, involving the traditional subsoil investigation as well as the results of load test on single pile. The procedure is described in detail by Mandolini & Viggiani (1997) and Viggiani (1998), and is summarised in the following. The results of all the available site and laboratory investigations are first used to develop a model of the subsoil, in which the geometry is adapted to a scheme of horizontal layering. The relative stiffness of the layers is also evaluated, such an evaluation being relatively easy on the basis of the results of laboratory tests, or site tests as CPT, SPT, DMT. The absolute values of the stiffness of the different layers are then fixed by fitting the load settlement curve of the single pile (preferably obtained by a load test, or simulated with a suitable procedure) to the results of an elastic analysis of the single pile based on the previously developed subsoil model. Once the subsoil model is fixed and the stiffness of each layer is established, the same model is used for the analysis of the piled foundation.

It is necessary to choose whether implementing a linear or non linear analysis. In the former case, the soil stiffness may be determined fitting the results of the elastic analysis either to the initial tangent of the load–settlement curve of the single pile (Linearly Elastic or LE analysis), or to a secant corresponding to the mean service load of the piles (Elastic analysis based on Secant modulus, or ES). This latter choice, apparently the most reasonable one, is at present the most widespread.

If non linearity is believed to be significant, then a stepwise linear incremental analysis (NL analysis) is performed, updating the stiffness matrix at each load step.

The three different procedures lead obviously to different results, and in some cases the difference is significant. A comparison with the experimental evidence clarifies the meaning of the different analyses and helps selecting the most suited one.

To this aim the code NAPRA (Russo, 1996, 1998a; Mandolini & Russo, 2005) has been employed in the back analysis of a number of case histories. Following Caputo & Viggiani (1984) the overall behaviour of each pile is modelled in NAPRA by a non linear relation (e.g., a hyperbolic relation between load and settlement), while the interactions among the pile and other elements are still assumed to be linear. In other words, the non linearity is concentrated at the pile–soil interface.

A substantially similar approach has been suggested by Randolph (1994) and El Mossallamy & Franke (1997). It may be shown that, in terms of settlement, this procedure is equivalent to adding the non-linear component of the settlement of the single pile to the settlement of the group, obtained as in the LE analysis. In any case, the main conclusions that will be presented are essentially independent of the particular code employed, and focused on the correspondence relationships.

4.1.3 *Settlement prediction*

A comparison between the observed average settlement of 48 out of the 63 case histories listed in table 5 and the predictions obtained by NAPRA is reported in figure 38.

The majority of the analysed foundations had been designed according to a conventional capacity based approach. As a consequence, their safety factor under the working load is rather high, and a simple linear analysis may be expected to be adequate for engineering purposes. Indeed the LE analysis, based on the moduli back figured by the initial stiffness of the load test on single piles, gives a rather satisfactory agreement with the observed values in all these cases (fig. 38a, open dots).

There are, however, some cases in table 5 and in figure 38a (full dots) referring to small pile groups constructed for research purposes and submitted to a load level close to failure (Brand *et al.,* 1972; Briaud *et al.,* 1989). For these cases non linearity plays obviously a major role and hence LE analysis is less satisfactory, resulting in a substantial underestimation of the settlement.

The NL analysis (figure 38b), which essentially consists in adding the non-linear component of the settlement of the single pile to the settlement of the group, obtained as in the LE analysis, slightly improves the prediction of the average settlement in all the cases where the LE analysis was already successful. In the cases where the non linearity plays a significant role, NL analysis significantly improves the prediction.

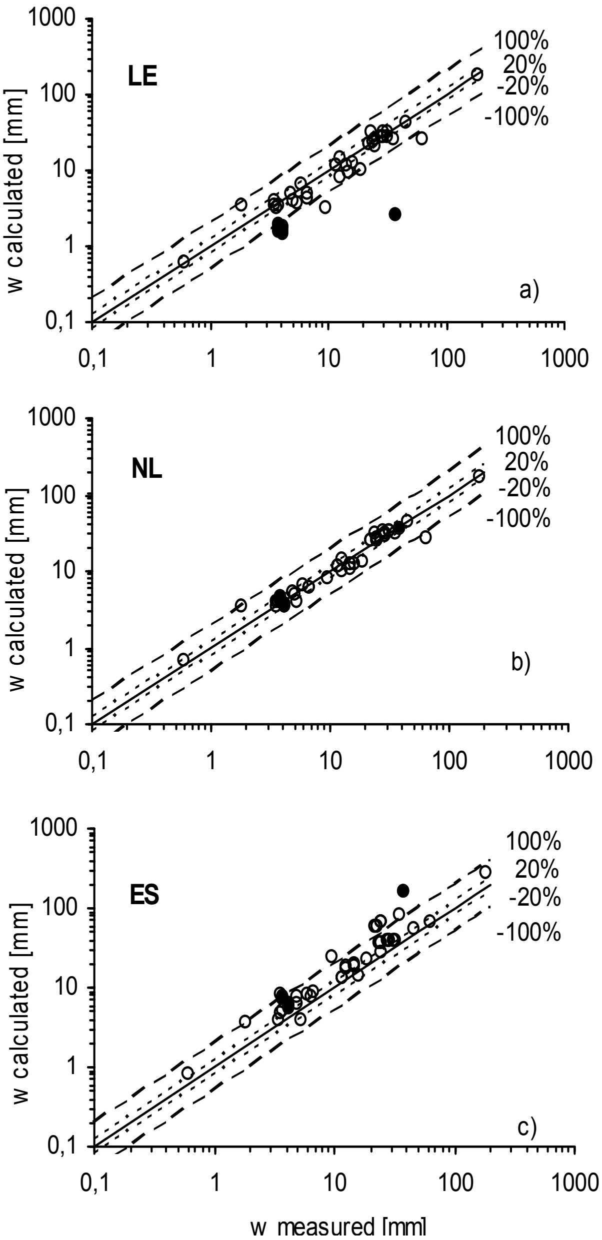

Figure 38. Comparison between predicted and measured settlement.

The ES analysis (figure 38c), on the contrary, incorrectly amplifies both the elastic and plastic components of the settlement of the single pile, and thus substantially overpredicts the observed settlement. It is clear that the choice of performing an elastic analysis on the basis of some secant modulus, the most widespread and apparently the most reasonable one, is in fact rather misleading.

The above comments also apply to the prediction of the maximum differential settlement (Mandolini & Viggiani, 1997). The available data, however, are slightly more scanty and scattered than those on the average settlement. This may be due, at least to some extent, to the unknown and variable influence of the stiffness of the foundation and superstructure.

4.1.4 *Prediction of load sharing and distribution*

Among the 22 case histories listed in table 7, only 4 provide the elements (including a load test on single pile) to carry out a prediction by NAPRA and to perform an assessment similar to that reported in figure 38 for settlement. Two of them (Stonebridge Park, Cooke *et al.* 1981; Garigliano Bridge, Russo, 1996 and present Report) are conventional piled foundations, designed following a capacity based approach assigning all the load to the piles. The sodium hydroxide tanks n° 12 and n° 14 in the Port of Napoli (Russo *et al.*, 2004 and present Report), on the contrary, have piles acting as settlement reducers.

The building at Stonebridge Park is founded on London Clay, with 351 bored piles, 0.45 m in diameter and 13 m long, connected by a raft in contact with the ground.

The load at the head of 8 piles and the pressure at 11 points beneath the raft have been measured; on this (rather limited) basis Cooke *et al.* (1981) backfigured the load sharing between piles and raft. In the early stage of construction the raft carried above 40% of the total applied load; this percentage decreased below 25% at the end of the observation.

The calculations carried out with NAPRA (Mandolini *et al.*, 1997; Russo, 1998b; Viggiani, 1998) in undrained conditions gave a load on the raft equal to about 20% of total; under drained conditions the load on the raft was almost negligible.

For the pier of the bridge across the Garigliano, at the end of the construction period the load taken by the piles was about 78% of the net applied load. After 10 years this percentage increased to 87%. Considering only the bridge deck weight and leaving apart the weight of the foundation raft these percentages increase to 86% and 98% respectively. The calculations were carried out considering the raft already in place acted upon by the bridge deck load (Mandolini *et al.*, 1997; Russo, 1998b; Viggiani, 1998). In undrained conditions 89% and in drained conditions 100% of the load was carried by the piles. The load distribution among the piles calculated in undrained conditions by NAPRA is compared with that measured at the end of construction in the contours plot reported in figure 39. The agreement is rather satisfactory.

Four steel tanks for the storage of sodium hydroxide have been recently built in the area of the Port of Napoli. The subsoil of the area consists essentially of cohesionless deposits, overlain by a cover of made ground; the water table is found at a depth of 2.5 m below the ground surface. A typical soil profile at the site of the tanks, including SPT blow counts and 2 CPT profiles is reported in figure 40.

The tank foundations are rather stiff reinforced concrete raft, with CFA piles designed to act as settlement reducers. The design issues, the type of piles used and the results of a load test on a single pile are reported by Russo *et al.* (2004) and will be briefly recalled in § 5 below. Settlement and load sharing among the raft and the piles were monitored during construction and first filling.

The loading program was controlled by the storage and supply needs of the sodium hydroxide, and thus neither a complete filling of a single tank, nor a contemporary filling of all the four tanks was performed. In figure 41 the time history of the applied load and the average settlement of three tanks are reported, together with a sketch of the four foundations. The only load considered is the weight of the liquid filling the tanks, because the measurements of the settlement started after the construction of the rafts and the weight of the steel tanks is almost negligible.

The load distribution among the piles on a diameter of the tanks 12 and 14 is plotted in figure 42 at six typical dates. At the beginning the piles carry part of the tank and raft weight, as it can be seen by the piles of the tank 14 which was still unloaded. At the partial filling of the tank 12, the piles show an edge effect, as it was to be expected under a stiff raft. It is interesting to point out that the load on pile 39, belonging to the tank 14, decreases when the adjacent tank 12 is loaded (date 3). At the maximum applied load (date 4) both the foundations show an edge effect with a significant asymmetry produced by the interaction between the two tanks. The subsequent unloading leaves in the piles of both tanks a residual load larger than that acting before the sequence of filling started.

In figure 43 the settlement along the same diameters at the selected dates shows how significant was the interaction in terms of rotations.

In figure 44 the total tanks load, including the weight of the rafts and of the steel, is plotted against the time together with the total load acting on piles expressed as a percentage of the tanks load. The data reported refer to the same dates selected for figures 42 and 43, plus a further one at the beginning, showing the load sharing recorded before filling started. For both the tanks, the pile supported initially 30% to 40% of the weight of the raft plus the steel tank. During filling the pile load increases up to about 50% of the total applied load.

The unloading of the tanks produces a substantial change in the load supported by piles, whose percentage approaches almost 100%.

For the tanks 12 and 14 two back-analyses have been carried out to simulate two stages of the complex load-time history reported in figure 41:

1. first filling of the tank 12 up to maximum load without taking into account the partial intermediate unloading;
2. first filling of the tank 14 up to the maximum load, taking into account the contemporary partial re-filling of tank 12.

Analysis 1) calculated that 45% of the applied load was supported by the piles of the tank 12, in substantial agreement with the observed percentage of 50% (figure 44). The pile load calculated by the analysis 2) for the tank 14 was about 55% of the total, again in reasonable agreement with the observed percentage of 45%. The calculated load distribution among the piles located along a diameter of the tank 12 is compared to the observed distribution in figure 45. The unexpected observed asymmetry is larger than that revealed by the calculations, but the agreement is not that bad. Furthermore the calculations reproduce the load decrease on the edge pile n° 39 of the tank 14 during the loading of the tank 12.

From the above comparisons, it seems that the limited available evidence is encouraging about our capacity of prediction; further comparisons, and hence further observations, are however badly needed.

4.1.5 *Bearing capacity*

On the basis of a broad numerical investigation, de Sanctis & Mandolini (2003) developed a simple criterion to obtain the ultimate vertical load Q_{PR} of piled raft from the separate capacities of the unpiled raft Q_R and of the uncapped pile group Q_P, as obtained by the conventional bearing capacity theory.

The main parameters adopted for this study and some of the results are listed in table 9. Once checked the validity of conventional theories for evaluating the bearing capacities of an unpiled raft (Q_R) and of an uncapped pile group (Q_P), they compared the sum of the ultimate load of single piles with those of pile groups, deriving from eq. 10 a value of the efficiency factor η equal to unity, in agreement with the findings of Cooke (1986) for $s/d \geq 4$.

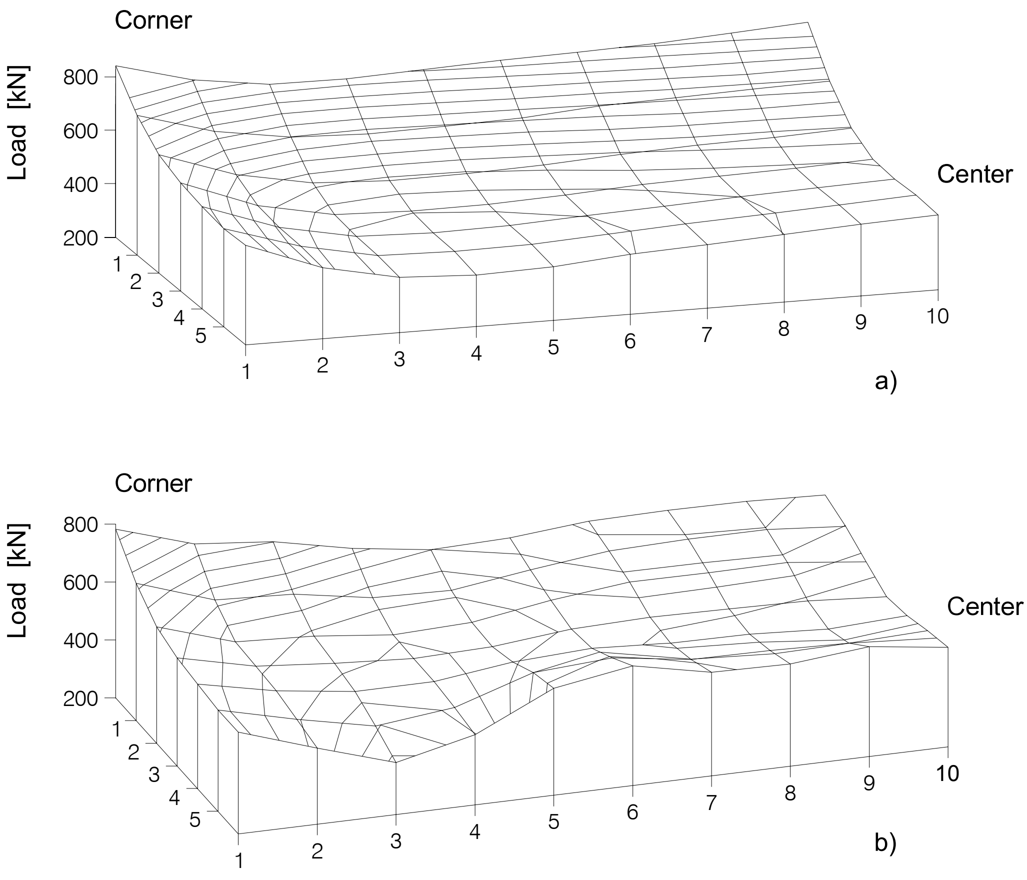

Figure 39. Load sharing among the piles of a quarter of the foundation of the main pier of the Garigliano bridge: a) calculated; b) measured.

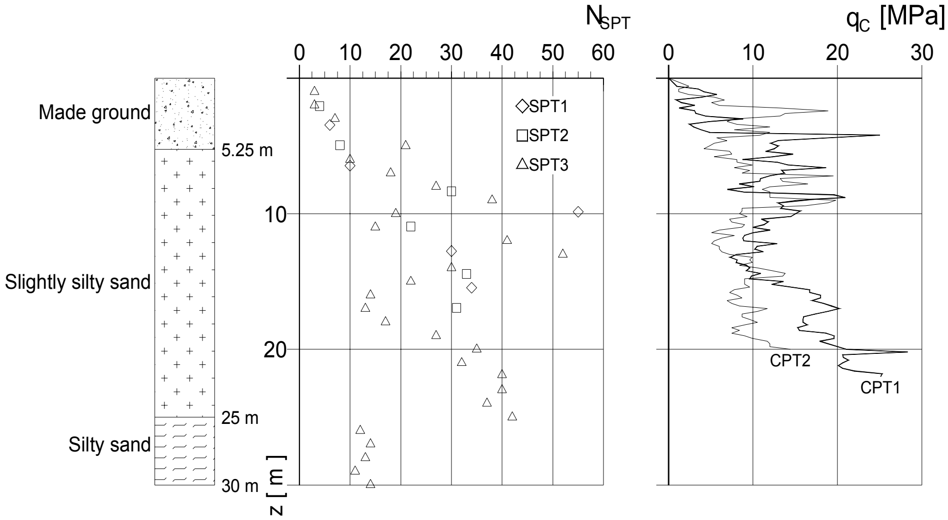

Figure 40. Soil profile with site investigations at the site of the tanks in the harbour of Napoli.

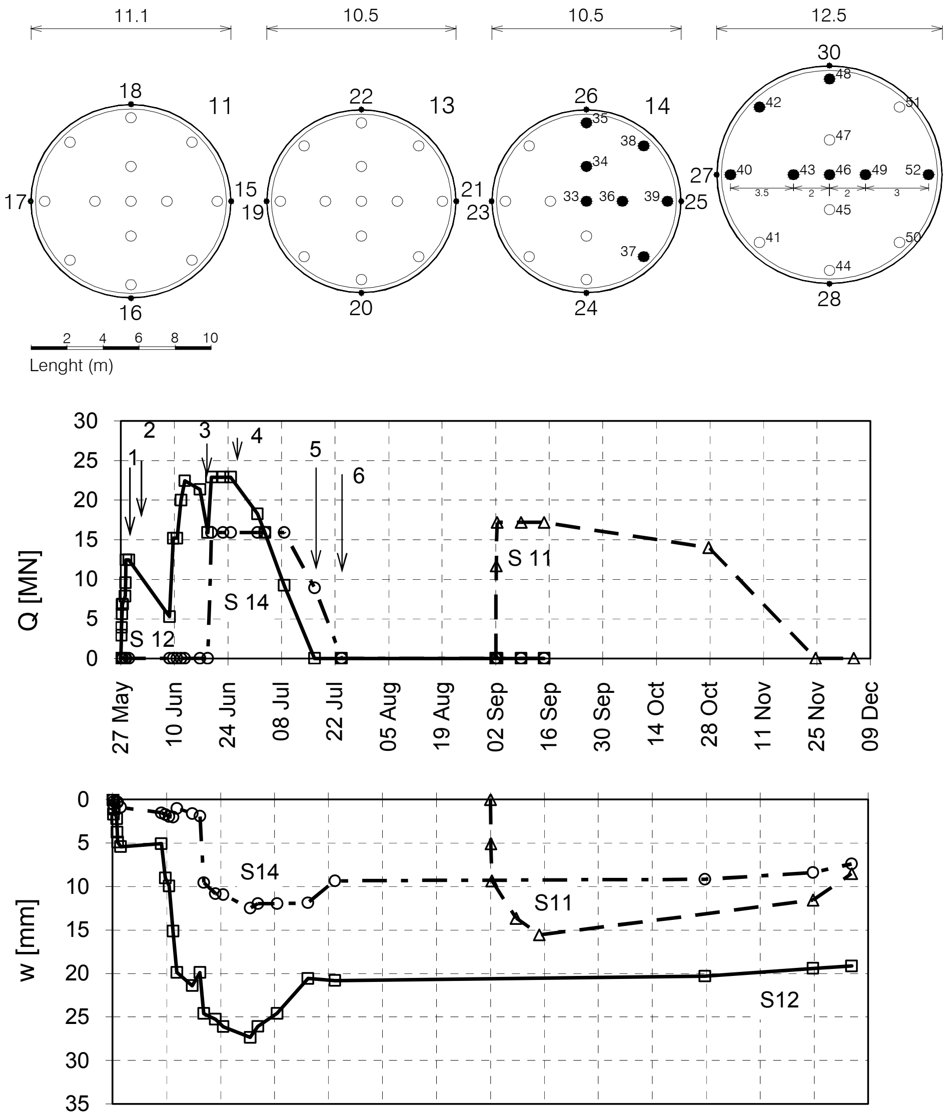

Figure 41. Applied load and average settlement for three tanks in the harbour of Napoli.

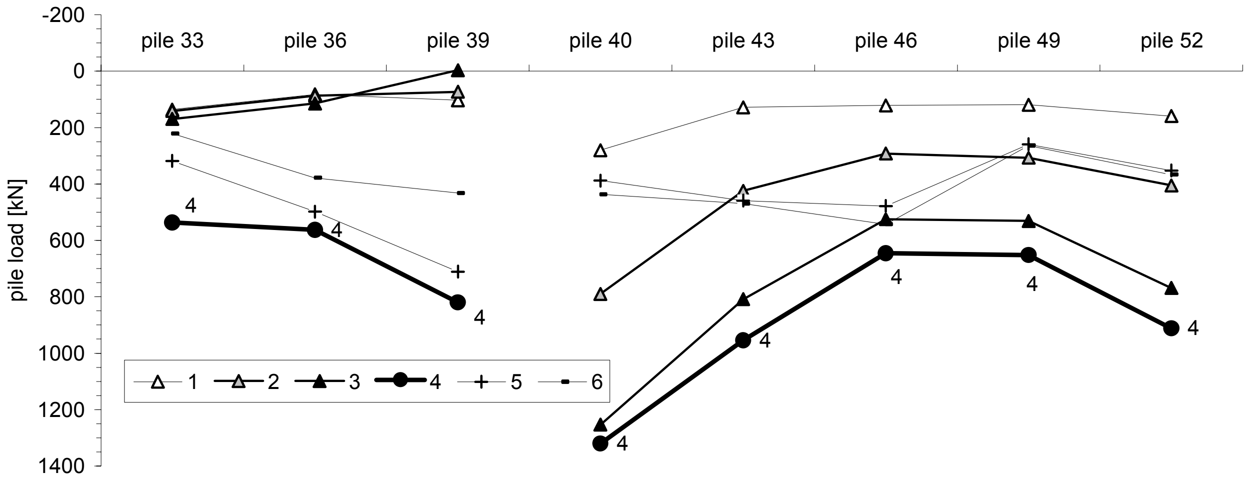

Figure 42. Load distribution among the piles on a diameter of the tanks 12 and 14.

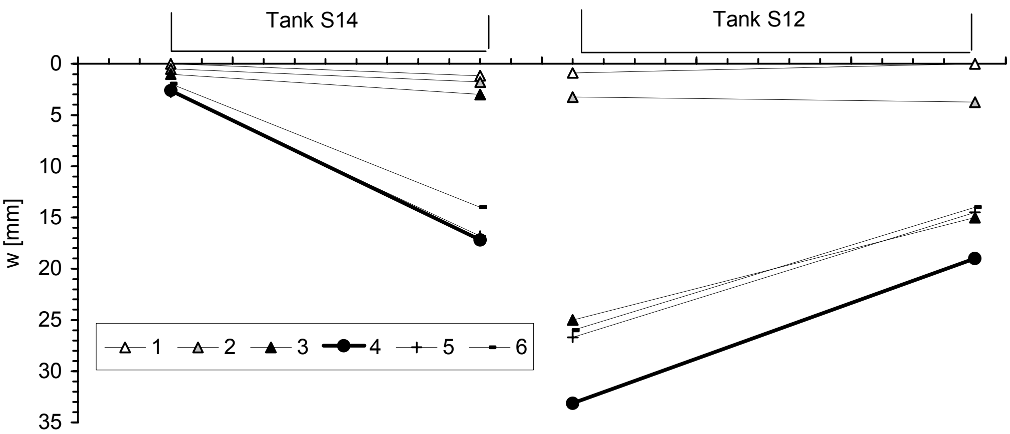

Figure 43. Settlement along the diameters of the two adjacent tanks 14 and 12.

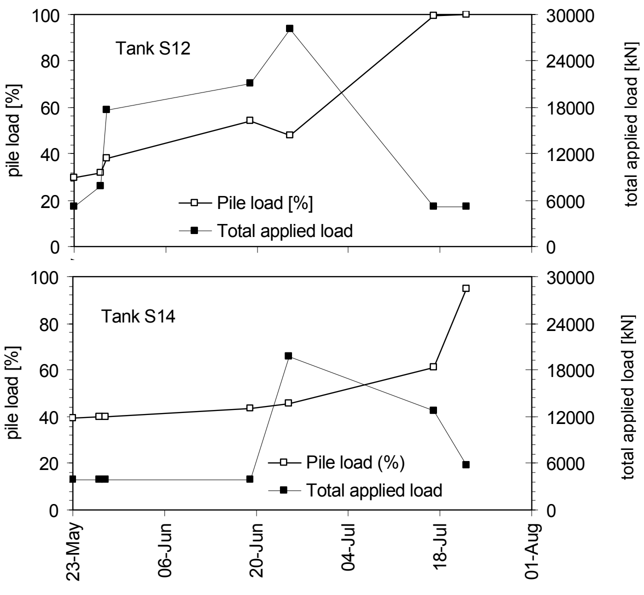

Figure 44. Total applied load and pile load vs. the time.

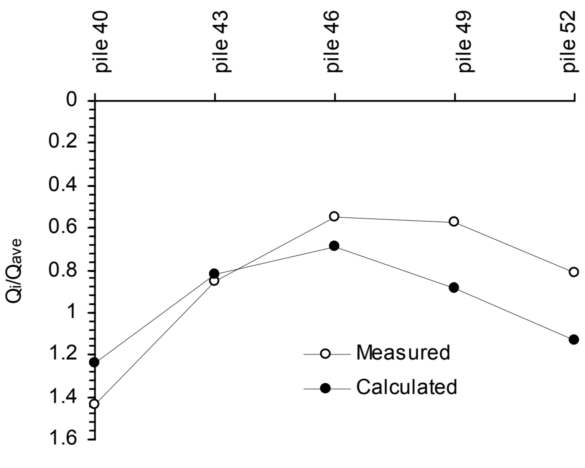

Figure 45. Load distribution among the piles along the diameter of the tank 12.

Table 9: Parametric study

Case	B/d	L/d	n	s/d	A_g/A	$\alpha_{R,10\%d}$ [%]	$\alpha_{R,25\%d}$ [%]	$\alpha_{R,10\%B}$ [%]
1	28	40	49	4	0,73	29	36	41
2	28	40	9	4	0,08	41	62	95
3	28	40	9	8	0,33	41	66	100
4	28	20	49	4	0,73	23	31	41
5	28	20	9	4	0,08	43	67	100
6	28	20	9	8	0,33	43	69	100
7	20	40	25	4	0,64	36	40	42
8	20	40	9	4	0,16	44	60	78
9	20	40	9	8	0,64	45	67	97
10	20	20	25	4	0,64	31	39	48
11	20	20	9	4	0,16	44	62	87
12	20	20	9	8	0,64	45	68	98
13	12	20	9	4	0,44	40	53	66
14	12	40	9	4	0,44	43	52	61

Moreover they found systematically $\alpha_P = 1$ (eq. 12); hence derived different values of α_R from the same equation at different displacements reached with the analysis (10%d and 25%d) or extrapolated by hyperbolic interpolation of the numerical data (up to 10%B). The three sets of values for α_R are listed in table 9 and plotted in figure 46 against the quantity (s/d) / (A_g/A) already introduced in § 3.2.2. As expected, α_R increases for increasing values of this quantity (see also Figure 25, working conditions) and for increasing displacement; moreover, it seems that some limiting value (s/d) / (A_g/A) ~ 10 exists, above which no significant increase of α_R occurs.

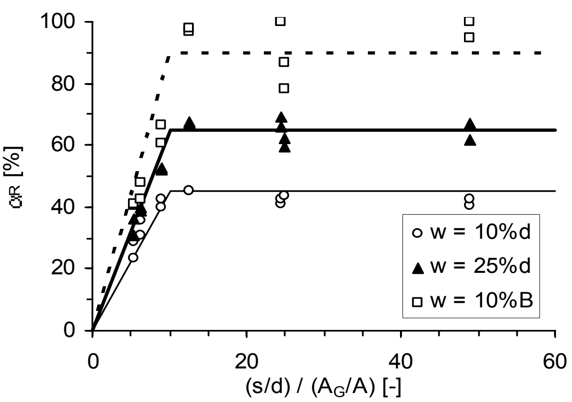

Figure 46. Relationship between α_R and (s/d) / (A_G/A).

de Sanctis & Mandolini (2005a, 2005b) defined a coefficient:

$$\xi_{PR} = \frac{Q_{PR}}{Q_R + Q_P} = \frac{FS_{PR}}{FS_R + FS_P} \tag{16}$$

representing the ratio between the ultimate load of a piled raft as derived form the numerical analysis and that of the unpiled raft and of the uncapped pile group evaluated by conventional theories. Under a given applied load Q on the piled raft, three different factors of safety may be defined: that for the unpiled raft ($FS_R = Q_R/Q$), for the pile group ($FS_P = Q_P/Q$) and for the piled raft ($FS_{PR} = Q_{PR}/Q$). Their ratio is always equal to ξ_{PR}, independently of the selected value for Q (or w).

In table 10 are reported the values of ξ_{PR} under a load Q corresponding to a settlement w = 3.5‰B for the unpiled raft, that is typical for a piled raft under working conditions when designed neglecting the contribution of the raft-soil contact (Cooke, 1986). As it can be seen, the sum of the factors of safety of unpiled raft and uncapped pile group equals to within ± 20% the computed value of FS_{PR} ($0,82 \leq \xi_{PR} \leq 1,00$). In other words, the safety factor of a piled raft is slightly lower than the sum of the two safety factors of the unpiled raft and the uncapped pile group. Such a result may be useful in design.

Table 10: Results of the parametric studies (p_{ref} = 100 kPa)

Case	$Q_R/p_{ref}d^2$ [-]	$Q_G/p_{ref}d^2$ [-]	$Q_{PR}/p_{ref}d^2$ [-]	$Q/p_{ref}d^2$ [-]	FS_R [-]	FS_G [-]	FS_{PR} [-]	ξ_{PR} [-]
1	1235	4090	4909	633	1.95	6.46	7.76	0.92
2	1235	751	1828	633	1.95	1.19	2.89	0.92
3	1235	751	1891	633	1.95	1.19	2.99	0.95
4	1235	1364	2120	633	1.95	2.15	3.35	0.82
5	1235	251	1446	633	1.95	0.40	2.28	0.97
6	1235	251	1465	633	1.95	0.40	2.32	0.99
7	630	2087	2644	299	2.11	6.98	8.84	0.97
8	630	751	1284	299	2.11	2.51	4.29	0.93
9	630	751	1348	299	2.11	2.51	4.51	0.98
10	630	696	1088	299	2.11	2.33	3.64	0.82
11	630	251	761	299	2.11	0.84	2.54	0.86
12	630	251	809	299	2.11	0.84	2.70	0.92
13	227	251	408	101	2.26	2.49	4.06	0.86
14	227	751	976	101	2.26	7.47	9.71	1.00

4.2 *Horizontal loads*

Poulos (1971b) first proposed the interaction factors method to analyze pile groups under horizontal load, using the theory of elasticity to derive the interaction factor α_{ij} between a loaded pile i and an unloaded adjacent pile j. The matrix of interaction factors is symmetric and independent of the load level. Compared to the case of vertical load where α_{ij} is only a function of the spacing of the piles i and j, computational complexity is slightly increased by the dependence of the interaction factor on the angle β between the load vector and the line connecting the pile i and j.

In figure 47 (Poulos, 1971b) the variation of α_{ij} with the angle β is reported; it attains a maximum at β = 0° and 180° and a minimum at β = 90°.

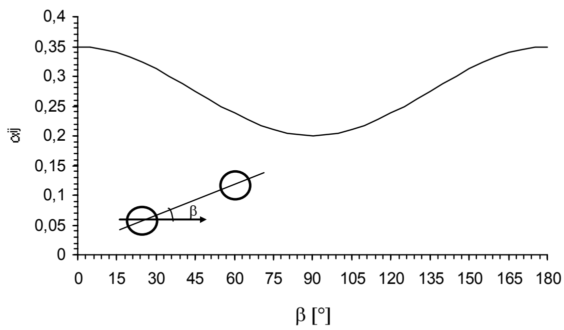

Figure 47. Typical variation of α_{ij} with the angle β.

Poulos & Davis (1980) presented a wide range of charts of interaction factors; Randolph & Poulos (1982) developed analytical formulae based on the critical length, L_C.

Banerjee & Davies (1978) and Davies & Budhu (1986) presented linear and non linear analyses based on boundary element method for piles embedded in non homogeneous soil profiles.

Along a different research path, the spring model introduced by Matlock & Reese (1960) has been extended to the analysis of pile groups. The p-y method cannot actually account for the interaction through a continuum both along the single pile and among piles in a group. In order to analyse a pile group, the shape of the curves must thus be adapted taking into account the type of soil and the geometry of the group.

The experimental findings presented in § 3.3 stimulate some comments.

- The widely diffused assumption that p-y multipliers f_m are independent of displacement appears a reasonable choice if a particular displacement level is of concern, i.e. an equivalent secant approach is adopted. On the contrary, if the objective of the analysis is an accurate prediction of the full load-displacement curve, the multipliers should be better selected as decreasing functions of the displacement level.

- The p-y curve method for the single pile and its extension to the pile group is an empirical procedure; both the shape of the curves and their multipliers can only be deduced by *ad hoc* experiments. Until observations on large size groups will not be available, the reliability of the method cannot be assessed. Bearing in mind the marked influence of the size of the group on the efficiency under vertical loads, and the lack of experimental data on large pile group subjected to horizontal load, the use of the multipliers obtained by experiments on small groups in the analysis of large pile groups under horizontal load is questionable and could be overly unconservative.

The experience shows that the asymmetry caused by the shadowing effect is confined to the very edge of the group (McVay *et al.* 1998). The rows typically affected are just the leading, the second and the trailing one. Modelling the soil as an elastic continuum and adopting boundary elements or interaction factors, such an asymmetry is not reproduced. Nevertheless these methods take into account the geometry and the size of the group. Is it preferable to push the use of empirical methods beyond the limits of the available evidence, or to use rational methods based on an analysis of the interaction through the continuum? In our opinion, the question is open.

5 DESIGN

Only the design under vertical loads is considered.

The first step in the design of a pile foundation is the selection of the pile type and installation method. The choice should depend on the subsoil properties, but it is often influenced by the local market and the regional practice.

Once the selection of the pile type has been made and the installation method specified, the next step is the evaluation of the bearing capacity of the single pile, that has been shown to be tightly connected to the installation procedures (§ 2.1.1).

The record of the installation parameter (as discussed, for instance, in § 2.1.2) provides a further insight into the single pile behaviour and partly explains the observed variability of pile response in apparently identical conditions.

At the time being, however, one cannot but agree with Poulos *et al.* (2001) that it is very difficult to recommend any single approach as being the more appropriate for estimating axial bearing capacity of a single pile. Given the very nature of the problem, the most reasonable approach seems to go on developing regional design methods combining the local experiences of both piling contractors and designers. The reliability of such methods depends on the quantity and quality of available evidence, particularly static load tests taking into account the influence of test setup (§ 2.1.3).

Moving from a single pile to a group, in general the effectiveness of a pile is reduced by the proximity to other piles. This is always true in terms of stiffness (the group reduction factor R_G, § 3.2.1, is always less than unity), but also in terms of the failure load for the usual values of the pile spacing (as for instance in the case of block failure, § 3.2.3). It follows that a rational design practice should minimise such a negative interaction using fewer and more widely spaced piles. A wider spacing, moreover, allows the structural element connecting the pile heads to transmit a portion of the external load directly to the foundation soil; such a sharing may be significant both under working conditions (§ 3.2.2) and at failure (§ 3.2.3).

Current design practice, however, is based on the assumption that a piled foundation behaves as a pile group with the cap clear of the ground; the design requisite is to ensure that the piles as a whole guarantee a proper factor of safety against a group failure. In some instances the same condition is required for each pile individually. Adopting such an approach leads to unnecessarily small settlement and to a significant cost increase. The requirement of satisfying some FS value for each pile individually is a further aggravation, because of the non uniform distribution of the load among piles due to the stiffness of the cap and possibly of the superstructure (§ 3.2.2), and leads to overly conservative design. When it was adopted by some UK Road Construction Units in the 1980's, the cost of the piled bridge foundations at least doubled (Burland, 2004); as a consequence, it was quickly dropped and designers reverted to applying a factor of safety to the bearing capacity of the group as a whole. In fact, if a corner pile approaches its full capacity its stiffness decreases and load redistribution takes place to the adjacent piles. Burland (2004) comments that the analogous situation for a rigid footing is that the high edge stress causes local yield with stress redistribution towards the middle of the footing; it has never been suggested that local factors of safety should be applied to such edge stress.

A good design should be aimed to satisfying some optimising criterion as, for instance, that "achieving maximum economy of the solution while keeping a satisfactory behaviour" (Russo & Viggiani, 1998). It is in the relation between these two aspects that an optimum has to be found; it is surprising that most of the papers dealing with optimum design do not deal at all with cost of the solution and/or definition of a satisfactory behaviour.

In order to clarify the interrelations between cost and performance, reference may be made to Figure 48 (de Sanctis *et al.*, 2002), where a quantity S defining the behaviour (absolute or differential settlement, stress, distortion) is plotted against the cost of the solution.

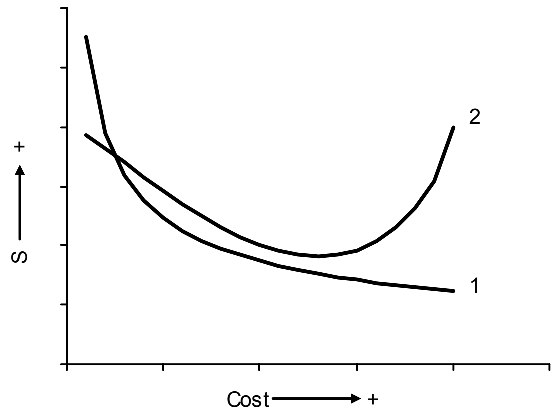

Figure 48. Interrelation between costs and performance.

In general, the performance of the foundation improves (e.g., the settlement decreases) as the cost increases. In some cases there is a steady improvement (1); in other cases there is a minimum S_{min} followed by an increase (2).

In the cases where curve 1 applies, the optimum solution is the one achieving the maximum admissible value S_{adm}, fixed by codes or local practice. Any further decrease of S results in a useless increase of the cost. Sometimes curve 1 approaches an asymptote S_∞; in this case if $S_{adm} < S_\infty$, a solution satisfying the performance criterion does not exist. Even if the admissible value is slightly larger than the asymptotic one, a solution may be too costly and a change in design may be needed.

In the cases where curve 2 applies, at a first glance the minimum could appear as an optimum, but this is not always the case. If $S_{adm} > S_{min}$ there are two solutions satisfying the performance criterion; the optimum one is obviously the left one. If $S_{adm} = S_{min}$, then the optimum solution is obviously defined. If $S_{adm} < S_{min}$, then a solution satisfying the performance criterion does not exist, irrespective of the cost, and it is necessary to change design or to renounce to the performance criterion. In the latter case, the most convenient solution is again that corresponding to S_{min}. Any further increase of the cost is useless, or even detrimental.

An example of steady improvement of the performance with increasing the number of piles (curve 1 in figure 48) is the redesign of the foundation of the main pier of the Garigliano bridge (Mandolini *et al.,* 1997). The total load acting on the foundation during construction was Q=113 MN (figure 14), approximately equal to the bearing capacity of the unpiled raft (Q_R = 112 MN). In a conventional capacity based design, 144 piles were added in order to increase the bearing capacity. With an ultimate capacity of the single pile Q_S = 3 MN, as deduced by load tests to failure, and a group efficiency η = 0.7, the Italian regulations (no contribution of the raft, FS $\geq$ 2,5) have been satisfied (Viggiani, 2001). As reported in § 3.1, such a design resulted in a measured settlement of 52 mm, while the actual load transmitted to the piles Q_P was about 87% Q.

Mandolini *et al.* (1997) back analysed this case history by the computer code NAPRA, obtaining satisfactory agreement both for settlement (figure 38) and for load distribution (figure 39). The same numerical model was then adopted to predict the behaviour of the foundation with a decreasing number of piles uniformly spread below the raft. The results obtained are reported in figure 49 in terms of settlement and load sharing, together with the prediction by Mandolini (2003), based on the simple PDR method (Poulos, 2000b).

Leaving apart the extreme solutions with a very small number of piles, for which the assumption of elastic raft-soil interaction is strongly questionable, figure 49 shows that a significant reduction of pile number (say from 144 to 72) is possible without noticeable increase of the settlement. Since the cost of the foundation is roughly proportional to the total length of piles, in this case a trend of performance vs. costs like curve 1 in figure 49 is occurring. There is a potential for substantial savings without significant reduction of performance, provided such a design was allowed by the existing regulations.

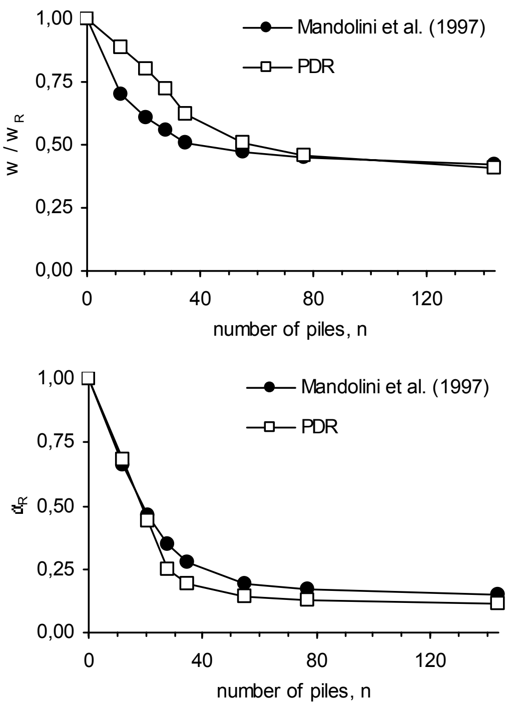

Figure 49. Garigliano bridge: settlement and load sharing calculated.

As an example of problems where the trend of curve 2 applies, Viggiani (2001) carried out a parametric study of the absolute and differential settlement of a large piled raft under uniform load (figure 50) and expressed the results in terms of the ratio $\Delta w/\Delta w_R$ between the differential settlement of a piled raft and that of the corresponding unpiled raft. The ratio is mainly affected by the parameter A_g/A, and attains a minimum at a spacing ratio s/d = 3 irrespective of the number of piles. In figure 51 the results for two different values of raft-soil relative stiffness K_{RS} (Fraser & Wardle, 1976) and three different ratios L/B $\leq$ 1 are reported. Larger values for K_{RS} and L/B are considered unrealistic.

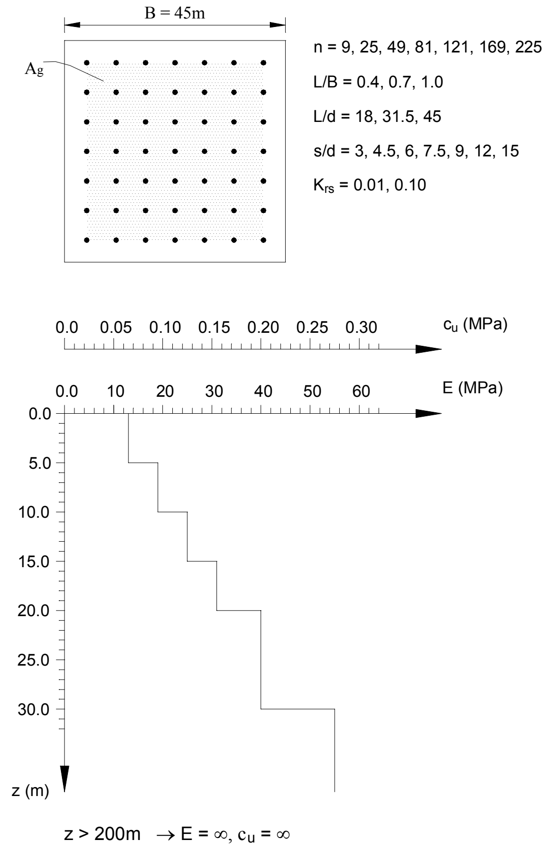

Figure 50. Scheme for parametric study of large piled rafts.

The same results are plotted in figure 52 against the total pile length nL, which may be considered roughly proportional to the cost of each solution. The curve is similar in shape to curve 2 in figure 48, and a minimum does actually occur. Furthermore, the longer the piles, the more economic is the solution; for a given total quantity of piles, a small number of long piles is the most convenient choice. For instance, adopting 81 piles with L=31.5 m (L/B = 0.7; nL = 2552 m), $\Delta w/\Delta w_R$ reduces to 15% with K_{RS} = 0.01, and to 2% with K_{RS} = 0.10. The same results may be obtained by using only 30 piles with L = 45 m (L/B = 1; nL = 1350 m). Similar charts have been recently proposed by Reul (2002), confirming that the addition of a small number of relatively long piles in the central zone of the foundation is very effective in reducing and even nullifying the differential settlement.

This indication does not apply when the applied load distribution is not uniform or the subsoil profile is markedly heterogeneous. In these cases an optimum solution has to be searched by a specific analysis for each specific problem. An example of this kind is reported by de Sanctis *et al*. (2002), with an exercise of optimisation of the foundation of two towers 90 m high in the eastern area of Napoli, founded on two adjacent piled rafts with 637 CFA piles, 0.6 m in diameter and 20 m in length, uniformly spread underneath the raft (figure 53). The weight of each tower was around 200 MN with the raft accounting for almost 100 MN. The superstructure consists of steel frames with reinforced concrete stiffening cores. About 66% of the total load is transmitted via the stiffening cores, while the remaining 34% is uniformly shared by the steel columns.

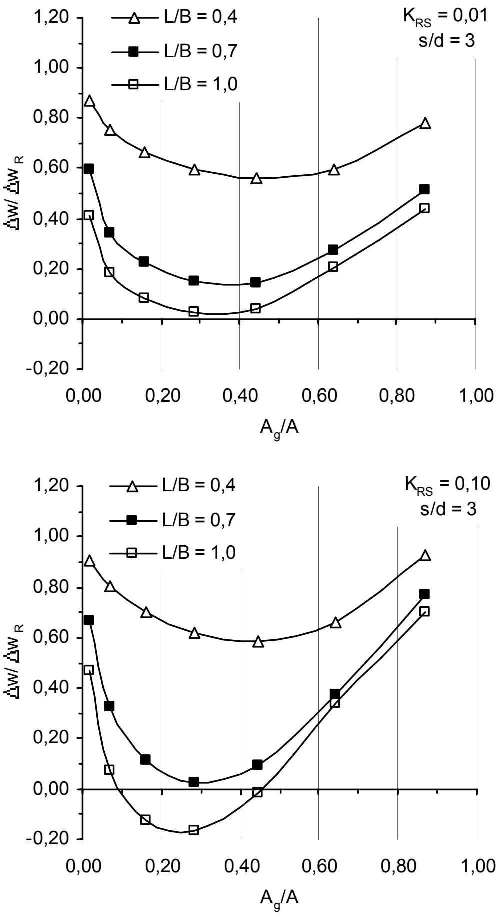

Figure 51. Influence of the ratio Ag/A on the maximum differential settlement of piled rafts (Δw) and unpiled rafts (Δw_R).

The code NAPRA was used again in order to fit the observed behaviour of the towers in terms of settlements. The plan and the cross-section of the (one quarter) foundation model used in the analysis are reported in figure 54, together with the location of the distributed and concentrated loads.

The measured and computed settlement for the actual piled raft and for the unpiled raft is plotted in figure 55. The provision of 637 piles uniformly spread below the raft, resulting from a capacity based design, reduces the average settlement by around 30% but is much less effective in reducing the differential settlement. Such a result can be considered typical of large piled rafts where the ratio between the raft width and the pile length B/L > 1; from a practical point of view, this value occurs when the foundation width B is of the order of some tens of metres. The agreement between computed and observed settlement is quite satisfactory, and thus the same computational model has been used to re-design the foundation with different criteria (small number of piles concentrated under the stiffening cores).

de Sanctis *et al*. (2002) demonstrated that the total pile length nL ≈ 12700 m (and hence, very nearly, the cost of the foundation) may be halved with a 25% reduction of the differential settlement and only a 10% increase of the maximum settlement.

According to Russo & Viggiani (1998) the design requirements are different for:

- small piled rafts, i.e. those in which the bearing capacity of the unpiled raft is not sufficient to carry the total load with a suitable factor of safety, and thus the primary reason to add piles is to achieve a sufficient factor of safety. This generally means that the width B of the raft amounts to a few metres, typically 5m < B < 15 m. In this range the flexural stiffness of the raft may be made, and usually is, rather high and the differential settlement does not represent a major problem; the requisite for an optimum design is the limitation of mean settlement and, subordinately, bending moments and shears in the raft. The width of the raft B is generally small in comparison with the length of the piles L (say B/L < 1);
- large piled rafts, i.e. those in which the bearing capacity is sufficient to carry the total load with a reasonable margin, so that the addition of piles is essentially intended to reduce settlement. In this case the flexural stiffness of the raft cannot be but rather small, and the requisite for an optimum design is the limitation of mean settlement and, above all, differential settlement. In general the width of the raft B is relatively large in comparison with the length of the piles (say B/L > 1).

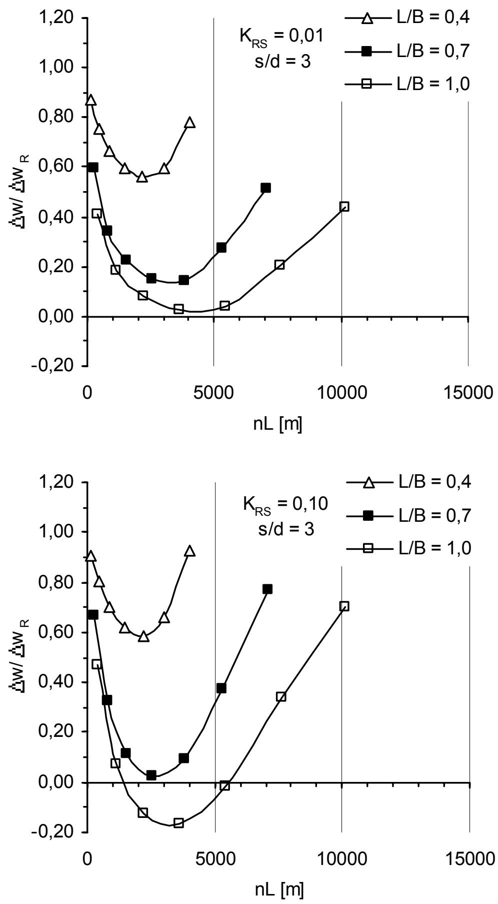

Figure 52. Influence of the total length of pile on the maximum differential settlement of piled rafts (Δw) and unpiled rafts (Δw_R).

Mandolini (2003) proposed a schematic chart for orienting the choice of the foundation type and the proper design approach. The chart is reproduced in figure 56 with minor modifications, and refers to a square unpiled raft resting on a deep deposit of clay.

In the figure, point A represents an ideal condition of optimum, for which under a certain applied load an unpiled raft experiences an overall settlement equal to some admissible value (= 100 mm in the figure) and in the mean time attains the minimum admissible value of the factor of safety FS (= 3 in the figure). Considering now another different raft foundation, three situations may occur:

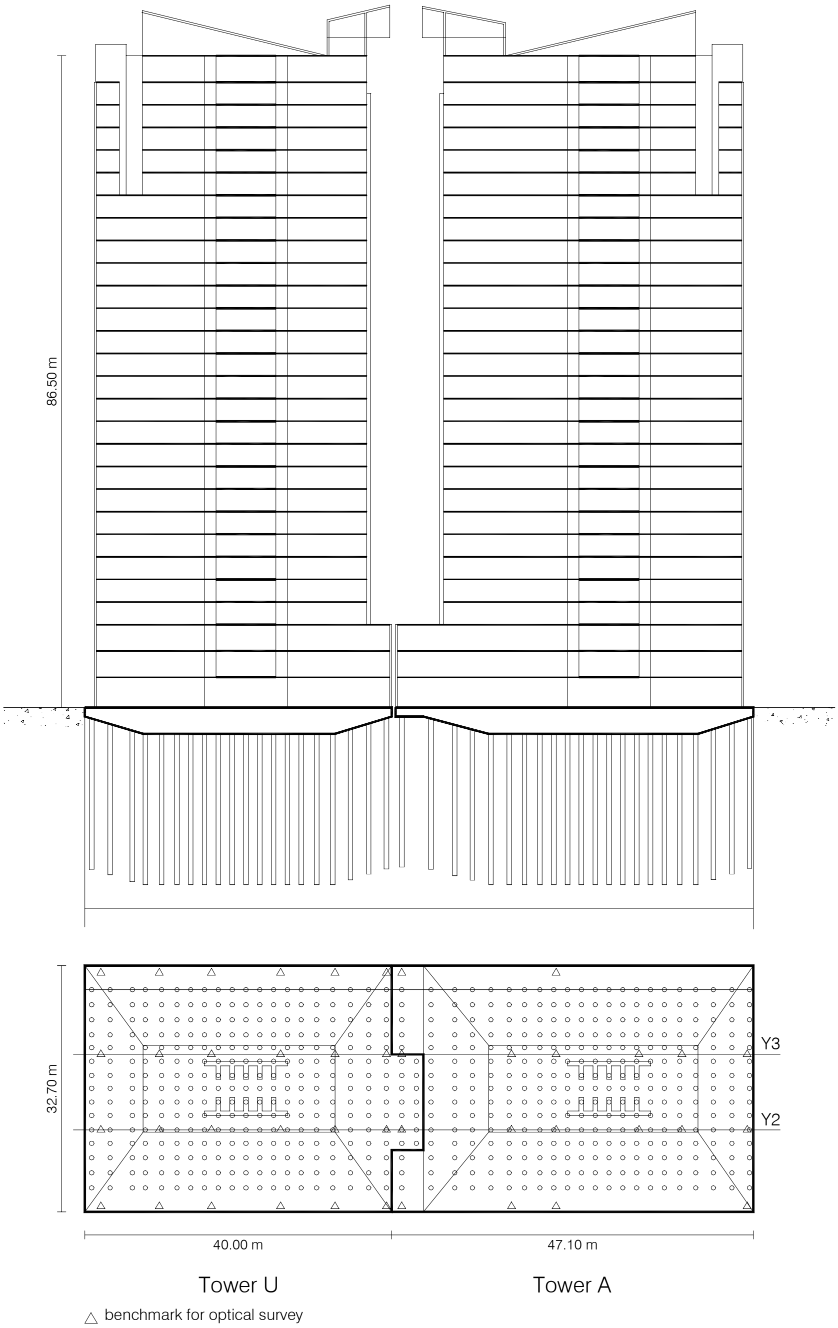

Figure 53. Plan of the foundation and cross section of the towers.

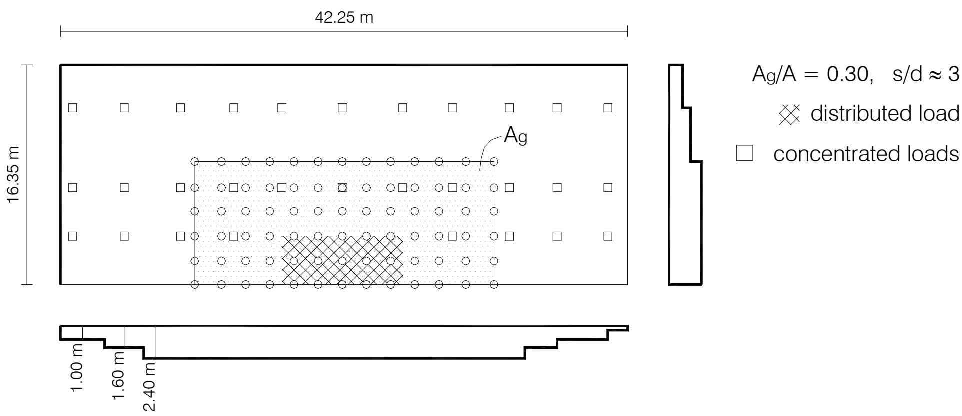

Figure 54. Foundation model adopted for the analysis by NAPRA.

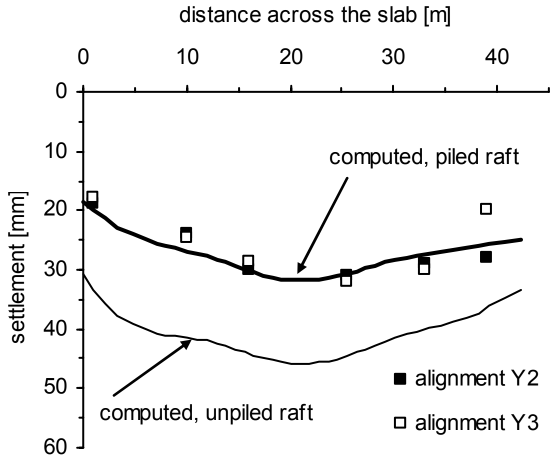

Figure 55. Measured and computed settlement for actual piled raft and the unpiled raft.

- both the estimated values of FS and w are acceptable (bottom right in the figure, point 1). The design requirement are satisfied; the adoption of an unpiled raft is possible;
- both the estimated values of FS and w are not acceptable (top left in the figure, points 2 and 3); piles have to be added in order to increase the value of FS and to reduce the overall settlement w;
- although the factor of safety is equal (point 4) or greater (point 5) than the minimum admissible value, the predicted settlement is above the admissible value. Piles have to be added again, but in this case with the aim of reducing settlement to an acceptable value.

The undrained settlement of the raft may be expressed:

$$w_o = qB \cdot \frac{\left(1-\nu_u^2\right)}{E_u} \cdot I_W \tag{17}$$

Assuming that the acting load $q = q_{ult}/FS = 6c_u/FS$, that the undrained modulus E_u of the clay is equal to $250c_u$, and being $\nu_u = 0.5$ and $I_w = 0.95$:

$$w_o = \frac{1.7\%B}{FS} \tag{18}$$

Eq. (18) indicates that the average immediate settlement increases linearly with increasing the foundation width; it is reported in figure 56 as a full line on the left of point A. Taking into account that non linearity will certainly play a major role as the factor of safety approaches unity, a more realistic trend is that depicted by the dotted line, attaining infinity for FS = 1.

With FS = 3 and w = 100 mm, eq. (18) leads to a value of B = 18 m (point A in figure 56). Under the same applied load, larger values of B will result in unacceptable settlement (point 4 in figure 56). For B > 18 m, even with FS > 3 unacceptable settlement can occur (point 5). If the applied load increases, smaller FS and larger w (points 2 and 3) will occur. A similar picture, although slightly more complicate, applies to drained conditions in clay and to granular soils (Mandolini, 2003). In all these cases (point 2, 3, 4 and 5) piles have to be added, but with different purposes, i.e. to fulfil different design requirements; consequently, their layout (number n, length L, diameter d, spacing s and distribution underneath the raft, A_g/A), should be obtained by different design approaches.

The case history of the sodium hydroxide tanks already referred in § 4.1.4 (Russo *et al.*, 2004) may be quoted as an example. The tank foundations are rather stiff reinforced concrete raft, with CFA piles designed to act as settlement reducers. Actually a foundation with unpiled raft would have been quite satisfactory from the viewpoint of safety against a bearing capacity failure (FS ≈ 5), but an average settlement of 160 to 180 mm was predicted. The computed settlement, and the expected associated differential settlement, was considered incompatible with a safe operation of the tanks. The next choice was a piled foundation. Following the Italian regulations, the conventional capacity based design ended in a total of 128 piles for the four tanks, with an estimated settlement of 12 – 13 mm.

The solution finally adopted includes 52 piles instead of 128, with an estimated settlement of 14 to 21 mm. Monitoring of the tanks at first filling showed a completely satisfactory behaviour (figure 41).

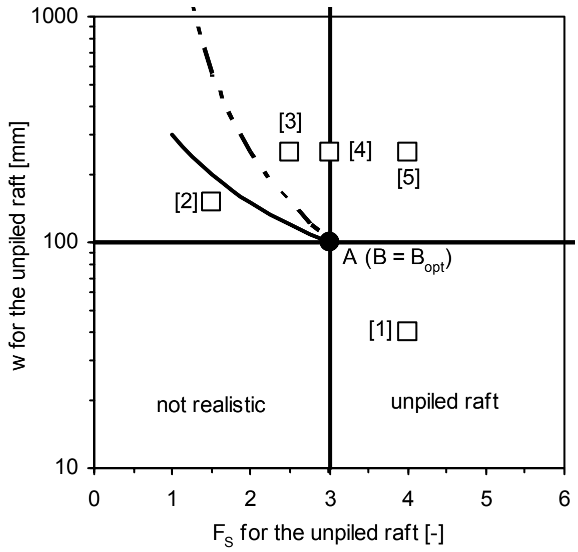

Figure 56. Chart for selection of the design approach for piled foundations (F_S=3; w_{adm}=100 mm).

The above case history is a typical example of the advantages that can be achieved using piles to control settlement. In practice, however, a capacity based approach imposing that the total structural load is carried by the pile with a nominal factor of safety is still dominant, as it is evident for instance in the current revisions of national and regional design codes. Such a situation may be attributed, at least partially, to a widespread belief that predicting deformations is more difficult and less reliable than predicting capacity. The evidence presented in this Report conclusively shows that this is not the case. Accordingly, codes and regulations compelling the designer to adopt a capacity based design are to be seen as an unjustified restraint.

There are cases, of course, in which the capacity based design approach can be considered appropriate, as for instance the cases when the unpiled raft exhibits a rather low factor of safety (point 2 in figure 56), and hence the possible contribution by the raft can be conservatively neglected.

When the factor of safety of the unpiled raft is not so low (say 2 ≤ FS < 3, points 3 and 4 in figure 56) the adoption of capacity based design results in overly conservative design with unnecessarily small settlement. In these cases a different design approach should be adopted, taking clearly into account the contribution of the raft in terms of bearing capacity and the positive effect of the piles in terms of decreasing the settlement. Looking at the factor of safety, results like those in table 10 allow for an assessment of the factor of safety of the piled raft starting from those of the individual components (unpiled raft and uncapped pile group) evaluated by conventional theories. Being the coefficient ξ_{PR} independent on the applied load and falling in a narrow range (0.8 to 1), FS_{PR} can be simply evaluated on the safe side by taking 80% of the sum of FS_R and FS_P.

Lack of a proper graduation in designing piled foundation becomes more and more evident for point 5, i.e. for large piled foundations. In such cases, piles are necessary only to reduce settlement (overall and differential) to some admissible value. Often the pile length L cannot exceed the raft width B, so that it is practically impossible to reduce significantly the absolute settlement, as shown above in figure 55 (de Sanctis *et al.,* 2002). In these cases piles should be located to prevent differential settlement (figures 51 and 52).

6 CONCLUDING REMARKS

It is a pity that restrictions by codes and regulations hinder a free selection of the most proper design approach, forcing to the adoption of a capacity based design whatever the actual design requirement is. This was not the case of the of Eurocode n° 7 (EN 1997-1: 1994), provisionally issued more than ten years ago. Quoting from § 7 - Pile foundations: "*When the piles are used to reduce the settlement of a raft, their resistance corresponding to the creep load may be used in analysing the serviceability state of the structure.*"

Inexplicably, the last version of Eurocode 7 (EN 1997-1: 2004), formally approved in March 2004, includes a different statement: *"The provisions of this Section should not be applied directly to the design of piles that are intended as settlement reducers, such as in some piled raft foundations."*

Piled rafts are thus not forbidden, and a somewhat stretched interpretation of this statement allows any design approach. Considering the rather bureaucratic aptitude of many public authorities, however, the Authors are rather sceptic on the practical possibilities of introducing significant innovations in design. Again, the new regulation acts as a restraint rather than a stimulus. It seems that a step backwards has been taken, just in a period when significant advances in the understanding of the mechanisms and in the capacity of analysis have been registered.

In the Authors' opinion, there are a number of points that can be considered "ready for use" in design:

- increasing the number of piles is generally beneficial but does not always produce the optimum solution. There is an upper limit to the useful number of piles, beyond which further increase is useless or even detrimental. Conventional design results generally in a number of piles beyond this limit;
- to control the average settlement, an optimum performance is achieved by the use of piles with a length L larger than the width B of the raft, uniformly spread below the whole raft area. This is practicable for small and medium rafts, but not for large ones. In this latter case, the average settlement is but slightly reduced by the addition of piles;
- to control the differential settlements, an optimum performance is achieved by a suitable location of a relatively small number of piles, rather than using a larger number of piles uniformly spread or increasing the raft thickness. The most suited location depends on the distribution of the load; for instance, in the case of uniform load over the raft, the piles are best concentrated in the central zone (A_g/A = 0.20 to 0.45). Again, the longer the piles, the most effective they are in reducing the differential settlement;
- the thickness of the raft affects the bending moments and the differential settlement, but has little effect on the load sharing between raft and piles and on the average settlement.

As a final comment, the Authors wish to insist that, in their opinion, the available procedures of analysis may be considered satisfactory for engineering purposes provided they are properly applied, paying due attention to the correspondence relations. The same opinion is expressed in a slightly different form by Poulos *et al.* (2001) when they write that: "the key to successful prediction is more the ability to choose appropriate geotechnical parameters rather than the details of the analysis employed".

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