Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 41 No.3 September 2010 ISSN 0046-5828

Piled Raft – A Cost-Effective Foundation Method for High- Rises

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ABSTRACT: During the last decades, the quick growth of cities all over the world has led to a rapid increase in the number and height of high-rise and super high-rise buildings. High-rises often rest on pile foundations, which are designed using the conventional method, where the piles take the full load from the superstructure. Recently it is increasingly recognised that the use of piles to reduce the foundation settlement and differential settlement can lead to considerable savings. Only a limited number of piles, called settlement-reducers, may improve the ultimate load capacity, the settlement performance, as well as the required thickness of the raft. In this article the result from the Author's experiment study, which strongly supports the concept of settlement-reducers in non-cohesive soil, are reviewed. Applications of FEM in design of piled-raft foundations for high-rises are also discussed.

1. FOUNDATION OPTIONS FOR HIGH-RISES

Foundation is the interface between the superstructure of the high-rise and the ground. Its task is to transfer safely the building loads into the ground and to keep settlement as small as possible. The foundation system must be designed to ensure sufficient external stability of the entire system and maintain the internal load-bearing capacity of the building components through appropriate design of the components. The serviceability of the building must be guaranteed for its entire lifecycle.

There are three principal foundation options to transfer the heavy loads from high-rises to the ground: 1) *Raft foundations*, where the loads are transferred to the ground via a foundation raft; 2) *Pile foundations*, where high-rise loads are transferred to a deeper load-bearing layers via piles or diaphragm wall elements; and 3) *Pile and raft foundations (PRF),* where the high-rise load is taken partly by the raft and partly by the piles or diaphragm wall.

1.1 Raft Foundation

In subsoil with good load-bearing capacity, as dense sand and gravel, un-piled raft foundation can be the most economic option for high-rises. The Trianon tower, which is almost 190m high and Main Plaza tower, 90m high, in Frankfurt are good examples, where the settlement remained under 100 mm and the tilting less than 1:800.

1.2. Pile Foundation

Pile foundations are necessary for cases, where the subsoil near the ground surface has low load-bearing capacity or heterogeneous conditions. The entire high-rise load is transferred to the firm layers only by piles or diaphragm wall. In such a foundation, or so-called *conventional pile foundation,* the raft is designed not to take any load from the superstructure. According to most standards, the piles must be designed with a safety factor of 2 to 3. This requirement results in a higher number and larger length of piles, and therefore the pile foundation is considerably expensive. Conversely, the settlement of the pile foundations is unnecessarily small. The pile foundation is the most common solution employed for high-rises worldwide, especially e.g. in the US, South East Asia, or Vietnam. Foundations are predominantly founded on large-diameter bored piles, barrettes or diaphragm wall, which are sometimes driven as deep as 80-100 m into the ground to reach load-bearing layers.

1.3. Piled-Raft Foundation

The traditional/conventional design practice for pile foundations is based on the assumption that the piles are free-standing, and that the entire external load is carried by the piles, with any contribution of the footing being ignored. This approach is over-conservative, since the raft or pile cap is actually in direct contact with the soil, and thus carries a significant fraction of the load. The philosophy of design is recently undergoing a gradual change. The concept of piled-raft foundations (PRF), in which the load from superstructure is partly taken by piles and the remaining taken by the raft is more and more accepted. The piles are designed to reduce the settlement, not to taken the total load. This idea of using piles as settlement-reducers was started in the seventies (Hansbo *et al*., 1973; Burland *et al.*, 1977). In the case of piled raft on clay, this philosophy has been developed into a refined design method in Sweden. According to the design method, the building load inducing stresses in excess of the clay pre-consolidation pressure is carried by the piles in a state of creep failure, while the remaining portion of the load is carried by the contact pressure at the raft-soil interface (Hansbo, 1984; Jendeby, 1986; Hansbo & Jendeby, 1998). A similar approach was introduced in the UK by Burland (1986). Enormous contributions to the development of the piled-raft foundation concept have been done in Germany during the 80's and 90's of the last century. Many piled raft foundations have been constructed in the Frankfurt Clay using settlement-reducing piled foundation for heavy high-rises (Sommer et al., 1985; Katzenbach et al., 2003). There are also applications in non-cohesive soil, like the Berlin Sand (El-Mossallamy et al., 2006). Recently, super high-rise buildings in the Gulf have often been constructed upon piled rafts. The load of the buildings is shared between the piles in shaft friction and the raft in direct bearing, with the pile system typically carrying about 80% of the total load directly into the deeper strata (Davids et al., 2008). For piled footings in non-cohesive soil, a systematic experimental study of the behaviour of the piled footings with the cap being in contact with the soil surface, has been carried out by the Author, Phung (1993). The study shows that the influences of the footing (cap) in contact with the soil on the bearing capacity of piles and on the load-settlement behaviour of a piled footing are considerable. The mechanism of load transfer in a piled footing involves a highly complex overall interaction between piles, pile cap and surrounding soil, which is considerably changed due to pile installation and to the contact pressure at the cap-soil interface.

2. CASE HISTORIES

During the last two decades, the quick growth of cities all over the world led to a rapid increase in the number and height of high-rise and super high-rise buildings, even in unfavourable subsoil conditions. Piled raft foundation concept has been successfully applied for many projects, some of which are summarised in Table 1.

Systematic measurements of the load transfer mechanism of piled raft foundations were performed to verify the design concept and to prove the serviceability requirements. The piled raft foundation has been widely applied as suitable foundation technique for high-rise buildings in Frankfurt to achieve economic solutions that fulfil both the stability and the serviceability requirements. The measured settlements of different case histories of piled rafts in comparison with traditional raft as well piled foundation are shown in Figure 1, in which factor α_L is a load factor representing the load taken by the piles relative to the total structural load.

Table 1: Piled Raft foundation-Case histories

No	Tower	Structure (height/storeys)	Load share (%)		Instrumen-tations	Settlement s_{max} (mm)
			Piles	Raft		
1	Messe-Torhaus, Frankfurt	130m, 30-storey	75	25	Yes	N.A.
2	Messeturn, Frankfurt	256m, 60-storey	57	43	Yes	144
3	Westend 1, Frankfurt	208m	49	51	Yes	120
4	Petronas, Kuala Lampur PF)	450m, 88-storey	85	15	Yes	40
5	QV1, Perth, West Australia	42-storey	70	30	N.A.	40
6	Treptower, Berlin	121m	55	45	Yes	73
7	Sony Center, Berlin	103	N.A.	N.A.	Yes	30
8	ICC, Hong Kong PF)	490m, 118-storey	70 D)	30 D)	N.A.	N.A.
9	Commerzbank, Frankfurt PF)	300m	96	4	Yes	19
10	Skyper, Frankfurt	153m	63	27	Yes	55

Note: PF) pile foundations; D) load share predicted by calculation design; N.A.= not available info

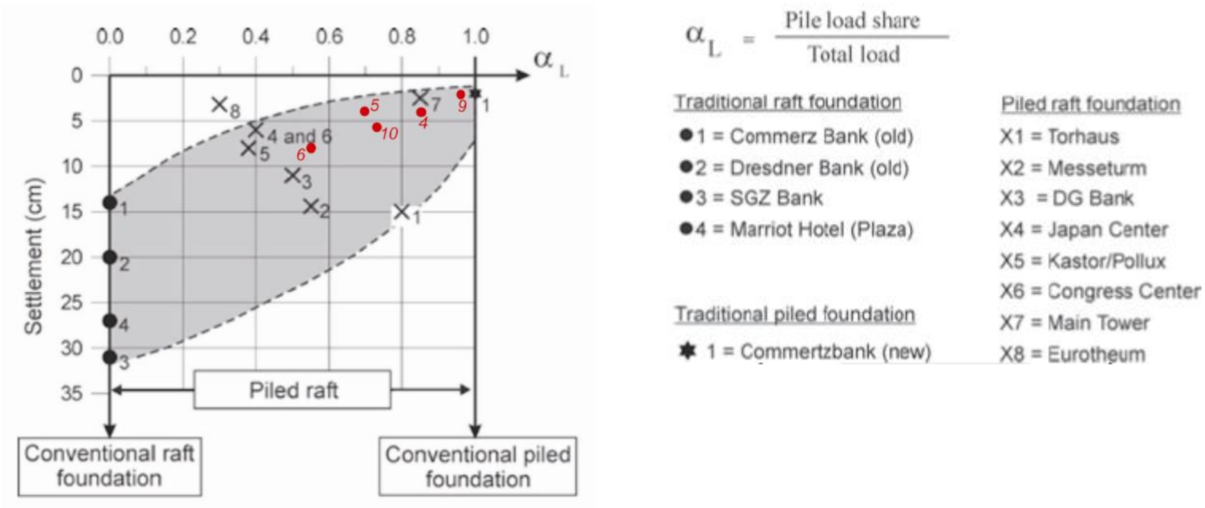

Figure 1: Raft and piled-raft foundation-Case histories (El-Mossallamy, 2008, modified by the Author by adding cases • 4,5,6,9 and 10 showed in Table 1)

It is noted that some foundations were designed as a pile foundation, but they acted as a combined piled-raft-foundation, i.e. the raft can take some part of building load. Petronas Tower in Kuala Lampur is a good example. The foundation was designed according to the conventional pile method. However, a certain part of the total load was still taken by the raft. According to the measurement, 15% of the dead load when the structure reached the height of 34 stories, or 40% of the total tower height. This percentage would have been smaller once the tower reached its full height. Low percentage of load carried by the raft seems to be due mainly to the presence of the soft soil near the ground surface. Commerzbank in Frankfurt is another example; in this case the piles take 96% of the total load.

From Table 1, we can see a clear connection between the settlement and the percentage of load carried by piles: the larger the load taken by piles, the smaller the settlement occurs. In fact the settlement (maximum value, differential settlement and its pattern) can be control by changing the number of piles, their length as well as their layout.

3. EXPERIMENTAL STUDY

The most well-known experimental study on pile groups in sand, which has been used as a major reference in most studies/researches is no doubt the one done by Vesic (1969). However, the experimental study was carried long time ago, and could not clarify some aspects of this complicated interaction problem. In order to clarify the overall cap-soil-pile interaction and the load-settlement behaviour of a piled footing in non-cohesive soil, three extensive series of large-scale field model tests were performed (Phung, 1993). Through the study, the Author has tried to create a better understanding of the load-transfer mechanism and of the load-settlement behaviour of a piled footing in non-cohesive soil, as well as the overall interaction between the piles, the cap and soil, especially the settlement-reducing effect of the piles.

Three different series of large-scale model tests (denoted as T1 T2 and T3) were performed. Each test series consisted of four separate tests on a shallow footing/cap (denoted as C), a single pile (S), a free-standing pile group (G), and a piled footing (F) under equal soil conditions and with equal geometry, see Table 2. The *overall pile-cap-soil interaction* of a piled footing in sand includes interaction between the piles, named as *pile-soil-pile interaction*, as well as between the piles and the pile cap (footing), which is in contact with the soil surface, named as *pile-soil-cap interaction*. Comparison of the results from the tests on free-standing pile groups with those on single pile shows the pile-soil-pile interaction, while comparison of the results on piled footings with those on free-standing pile groups and on un-piled footings (cap alone) shows the pile-soil-cap interaction.

A detailed description of the tests can be found elsewhere (Phung, 1993). All the pile groups were square, and consisted of five piles: one central and four corner piles. In these tests, the following measurements were made: individual pile loads, total applied load, lateral earth pressure against the pile shaft and displacement of the footing. Axial pile loads were measured by means of load cells at the base and the top of each pile. A load cell was placed in the middle a corner pile, to study the load distribution along the pile length. The lateral earth pressure against pile shaft was measured for the central pile, by twelve Glötzl total stress cells, installed symmetrically on all the four sides of the pile. Displacements were measured by electric resistance transducers. All the instruments were monitored by a data logger.

Comparison of the results from the separate tests in each test series is shown in Figures 3a to 5a. Look at Test series T1, Figure 3a, we can see that the load taken by cap in the piled footing, *T1F-Cap*, is very close to the load taken by cap alone, *T1C-Cap*. While the load taken by piles in the piled footing, *T1F-Piles*, is much larger than the load taken by piles in the free-standing pile groups, *T1G-Piles*. We can see a similar tendency in other test series, T2 and T3, see Figures 4a and 5a. Loads taken by the cap and the individual piles are shown against the total applied load in Figure 3b to 5b. In Figure 3b, the load share between cap and all single piles are drawn, while in Figures 4b and 5b, the load taken by cap is drawn together with the average load per pile.

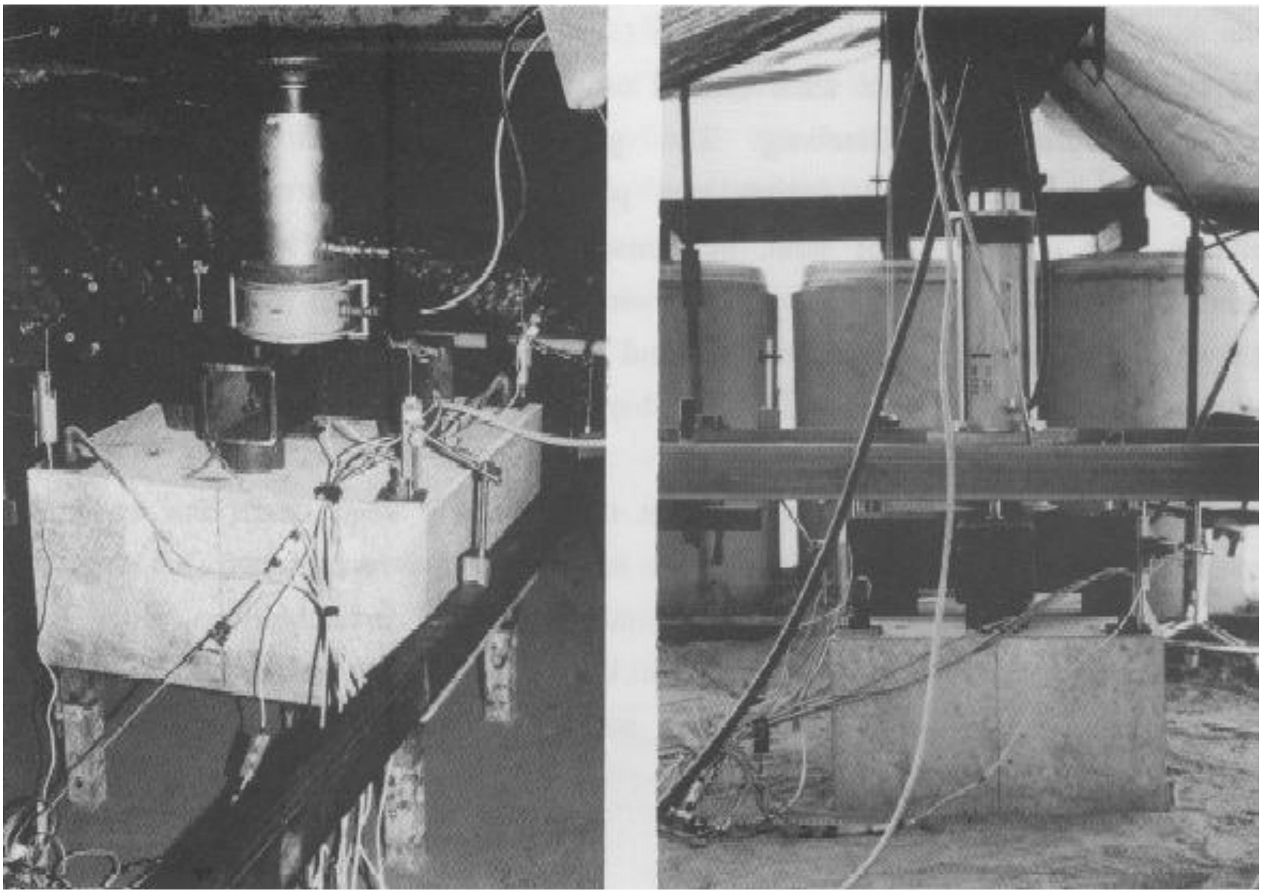

Figure 2. Field large-model tests set up: a) Test on a free-standing pile group; b) Test on a piled footing with the cap in contact with soil.

Table 2. Summary of the large-scale field model tests

Test Series	Pile Group and Cap (Footing)	Sand I_D, %	Separate tests in one test series	Pile length l_p (m)
T1	square group of five piles pile spacing *S*=4*b* cap: 46cmx46cmx30cm	I_D = 38%	T1C, shallow footing T1S, single pile T1G, pile group T1F, piled footing	- 2.0 2.1 2.3
T2	square group of five piles pile spacing *S*=6*b* cap: 63cmx63cmx35cm	I_D = 67%	T2C, shallow footing T2S, single pile T2G, pile group T2F, piled footing	- 2.0 2.1 2.3
T3	square group of five piles pile spacing *S*=8*b* cap: 80cmx80cmx60cm	I_D = 62%	T3C, shallow footing T3S, single pile T3G, pile group T3F, piled footing	- 2.0 2.1 2.3

From the test results, very important remarks are drawn:

- When the load is applied on the piled footing, the piles at first take a major portion of the load; not until pile failure a considerable portion of load is transferred to the cap, Figures 3bto 5b;
- The load-settlement curve of the cap in a piled footing is very similar to that of a cap alone, Figures 3a to 5a;
- The load carried by the piles in a piled footings is much larger than that the load carried by a free-standing pile group, Figures 3a to 5a.

3.1 Bearing capacity

From the test results, the Author suggested that the bearing capacity of a piled footing in non-cohesive soil P_{ft} can be estimated as follows:

$$P_{ft} = n \cdot (\eta_{1s} \cdot \eta_{4s} \cdot P_{ss} + \eta_{1b} \cdot \eta_{4b} \cdot P_{sb}) + \eta_6 \cdot P_c \quad (4)$$

where, *n* is the number of piles in the group; P_{ss} and P_{sb} are the shaft and base capacities of a reference single pile; P_c is the capacity of the cap; other symbols can be seen in Table 3 with indices "*s*" and "*b*" indicating (pile) shaft and base.

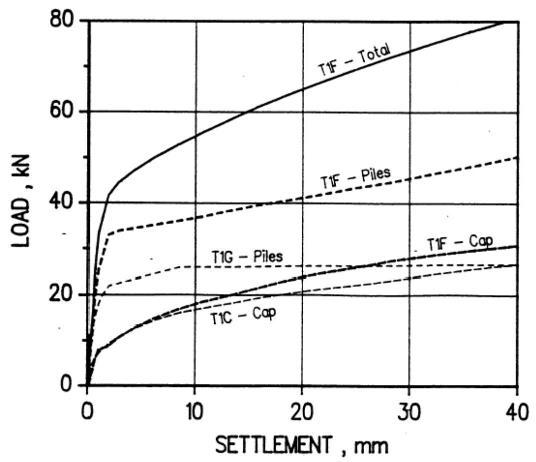

Figure 3a. Test series T1 – Comparison of separate tests

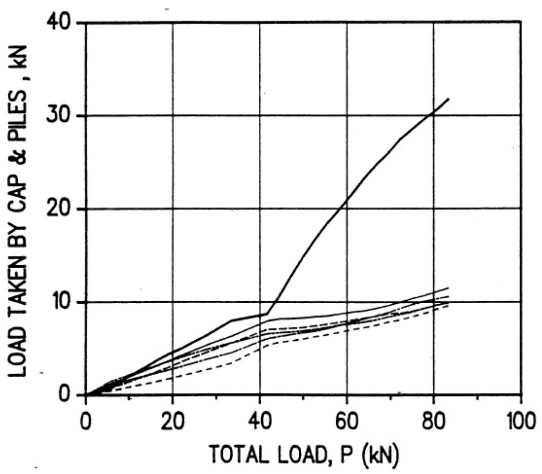

Figure 3b. Test T1F – Load share between cap and individual piles

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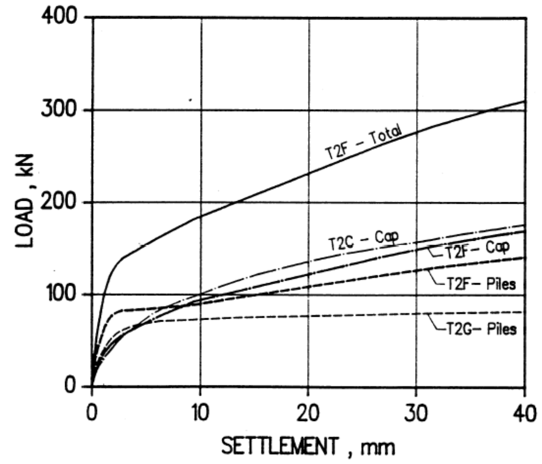

Figure 4a. Test series T2 – Comparison of separate tests

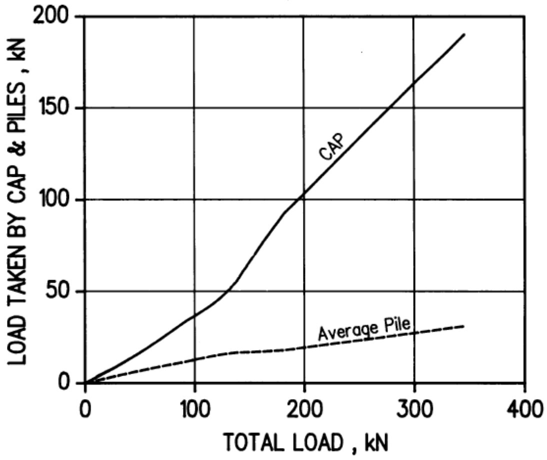

Figure 4b. Test T2F – Load share between cap and individual piles

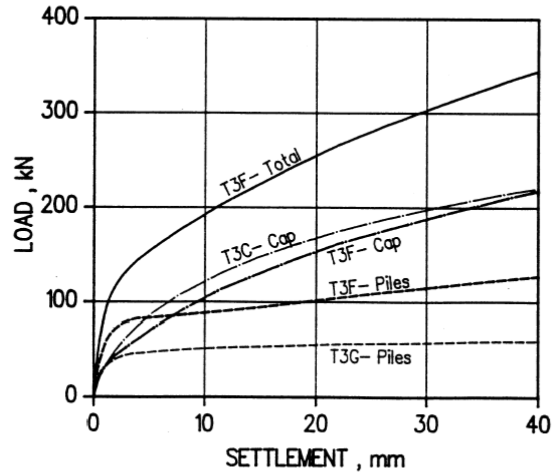

Figure 5a. Test series T3 – Comparison of separate tests

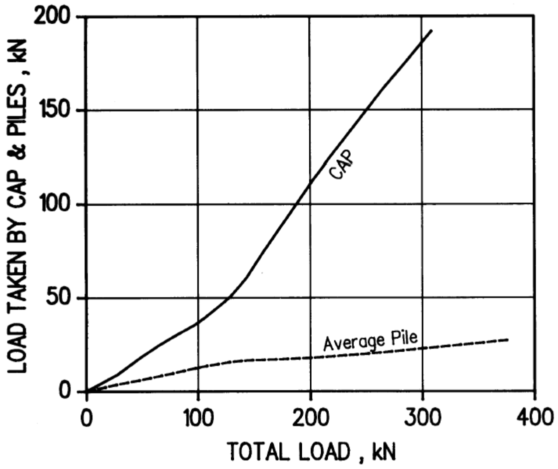

Figure 5b. Test T3F – Load share between cap and individual piles

Table 3. Definitions of load efficiency factors

Symbols	Definition	comparison between
η_1	P_{gr}/nP_s	free-standing pile group and single pile
η_4	P_{fp}/P_{gr}	piled footing and free-standing pile group
η_6	P_{fc}/P_c	piled footing and shallow footing

The efficiencies η_{1s} and η_{1b}, which show the influence of the pile-soil-pile interaction on the pile shaft and base capacities, can be estimated by comparing the load per pile in a free-standing pile group with that of a single pile at a certain settlement, e.g., s = 10 mm. The efficiency η_{1b} can be taken as unity for medium dense to dense sand, and higher than unity for loose sand. The efficiencies η_{4s} and η_{4b}, which show the influence of the pile-cap interaction on the pile shaft and base capacities, can be determined clearly by tests on piled footings performed according to the second test procedure. For piles long enough ($l_p>2.5B_C$, in which l_p is the pile length, and B_C is the cap width), we can take η_{4b} as unity. The efficiency η_6 shows influence of the pile-cap-soil contact on the cap capacity, and can be taken as 1.0 for loose sand and 0.9 for medium dense to dense sand.

3.2 Settlement Ratio

The traditional concept of settlement ratio ξ is used to compare the settlement of a free-standing pile group with that of a reference single pile. However, as discussed by the Author (Phung, 1992 and 1993), this ratio ξ depends very much on the choice of failure criterion and safety factor. For comparison of the settlement of a single pile, a free-standing pile group, a piled footing, and a shallow footing under equal conditions, different new settlement ratios were suggested by the Author in Table 4. In order to avoid the confusion caused by failure criterion the comparison is done at the same load level, i.e. at the same load per pile, or at the same applied load on footings.

Table 4. Definitions of settlement ratio factors

Symbols	Definition	comparison between
ξ_1	s_{gr} / s_s	free-standing pile group and single pile
ξ_3	s_f / s_s	piled footing and single pile
ξ_5	s_f / s_{gr}	piled footing and free-standing pile group
ξ_7	s_f / s_c	piled footing and shallow footing

In Table 4, s_S is the settlement of a single pile, and s_{gr}, s_C, and s_f are the average settlement of a free-standing pile group, a shallow footing and a piled footing under equal conditions. The ratios ξ_1 and ξ_3, estimated by comparing the settlement of a pile group or a piled footing with that of a single pile, are similar to the *conventional* settlement ratio ξ. These ratios have little practical meaning in estimating settlement of piled footings, and are not discussed here.

Comparison of settlement of a piled footing with that of a free-standing pile group leads to the ratio ξ_5. The test results show that this ratio at the same applied load is always much less than unity. This means that the increase in stiffness of the piles footing, as compared with the corresponding free-standing pile groups, is considerable. This conclusion is contrary to that drawn in most of the theoretical studies, based on the theory of elasticity (Butterfield & Banerjee, 1971; Poulos & Davis, 1980; and Randolph, 1983).

The ratio ξ_7, which is defined by comparing the settlement of a piled footing and that of a corresponding shallow footing at the same applied load, seems to be the most useful settlement ratio. This ratio means the reduction in settlement of a piled footing as compared with that of a shallow footing under equal conditions. As expected, the ξ_7-value, obtained from the tests is always lower than unity. The ratio is smaller in looser sand. This settlement ratio is further discussed later.

3.3 Influence of Cap Contact Pressure on Pile Skin Friction

As mentioned above, from the test results it can be found that the load taken by piles in a piled footing is much larger that that in a corresponding free-standing pile group. This can be explained by the increase in pile shaft friction, Δf_s, caused by the increasing lateral earth pressure due to the cap-soil contact pressure. The lateral earth pressure against the pile shaft was measured for the central pile in the groups by means of Glötzl total stress cells. The cells were read before and after each test series, before and after driving each pile, as well as before and after the tests on single piles and free-standing pile groups. The results are shown in the form of the increase in lateral pressure against the pile shaft as compared with the readings before the test. The intention is to separate the effect of the cap on the lateral pressure from other sources such as compaction effects due to pile driving, time effects, testing effects as the change of lateral pressure before and after the tests on the single pile and the free-standing pile group.

In all the three test series, the results show that before cap-soil contact, the lateral pressure increases only at the lower cells, while the readings from the upper cells are almost zero. This can be explained by the fact that a compacted zone develops around the pile tip at pile failure. The compacted zone causes the pressure to increase only at the lower cells, not at the upper cells near the cap bottom. Another possible reason is that the volume of sand increases due to dilatancy. This effect is larger with higher stress level.

The pressure increase due to the cap coming into contact with soil will add to the effect of the pile failure zone. The effect of the cap-soil contact is predominant for the upper cells near the cap bottom, while the effect of the pile failure zone is predominant for the lower cells near pile tip. Typical changes in lateral pressure against the pile shaft due only to the effect of the cap are plotted versus the load carried by the cap in the piled footings in Figure 6. More detailed results can be found elsewhere, see Phung (1993).

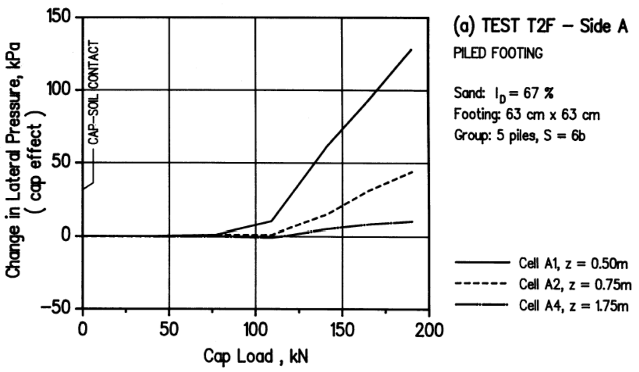

Figure 6a. Test T2F – Typical change in lateral pressure against pile shaft due to cap contact effect versus cap load in a piled footing

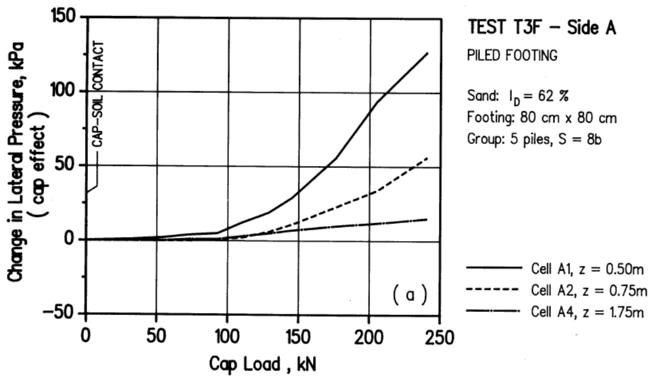

Figure 6b. Test T3F – Typical change in lateral pressure against pile shaft due to cap contact effect versus cap load in a piled footing

Generally said, in a piled footing, the pile skin friction consists of friction due to pile-soil-pile interaction, and friction due to the increase in lateral earth pressure caused by the cap-soil contact pressure and by the influence of the failure zone at the pile tip, as mentioned above. Only the skin friction due to the increase in horizontal stress is discussed here. The ultimate skin friction is generally expressed as:

$$f_{su}(z) = \sigma'_h(z) \cdot \tan\delta$$

where, $\sigma'_h(z)$ = horizontal effective stress, and δ = pile-soil friction angle.

The relative displacement between pile and soil $s_{ps}(z)$ should be large enough to mobilise full friction. In the general case, with a given value of the relative displacement $s_{ps}(z)$, the skin friction $f_s(z)$ can be calculated as:

$$f_s(z) = \sigma'_h(z) \cdot F(z) \cdot \tan\delta$$

where, $F(z)$ = level of mobilization of skin friction, $F(z) = s_{ps}(z) / s_{psu}$ when $s_{ps}(z) < s_{psu}$; otherwise $F(z) = 1$; and s_{psu} = relative displacement between pile and soil required to mobilise full skin friction, see Figures 7c and 7d.

The movement of the pile shaft relative to the surrounding soil, required to mobilise ultimate pile shaft resistance, is almost independent of the pile diameter and is in the order of 2 to 5 mm. When the pile cap comes into contact with the ground, it causes an increase in the horizontal pressure against the pile shaft, $\Delta\sigma'_h(z)$. At the same time it causes the soil under the cap to settle, called $s_c(z)$. As a result, the relative displacement between the pile shaft and the surrounding soil will be reduced in the region close to the cap. If the settlement of the pile top s_p and the pile compression $\delta_p(z)$ are known, and ignoring the settlement of soil due to the pile load, the relative pile-soil displacement becomes, see Figures 7b and 7c:

$$s_{ps}(z) = s_p - s_c(z) - \delta_p(z)$$

where, $s_{ps}(z)$ = relative displacement between pile and soil at depth z;
s_p = settlement of the pile top;
$s_c(z)$ = settlement of soil due to the cap;
$\delta_p(z)$ = pile compression.

At the pile head, depth z = 0, $s_{ps}(0) = 0$ because $s_p = s_c(0)$, and $\delta_p(0) = 0$. At a depth large enough $s_c(z) = 0$, and the above equation returns to the usual form: $s_{ps}(z) = s_p - \delta_p(z)$. The increase in skin friction Δf_s due to the cap in contact with soil will be zero at the cap-soil interface. It will then increase to a maximum value at a certain depth, where the relative soil-pile displacement is large enough. Thereafter it will decrease because $\Delta\sigma'_h$ reduces with depth. In Tests T2F and T3F, the depth, where the increase in skin friction Δf_s due to the cap reach the maximum value, is equal to or less than 0.5 m, because from this depth downwards $\Delta\sigma'_h$ always decreases. It should be noted that the reduction in relative displacement between the pile shaft and soil due to the cap being in contact with soil also makes the skin friction due to pile-soil-pile interaction reduced in vicinity of the bottom of cap. The effect of diminishing the relative displacement between piles and soil nearest below the cap can be seen just after cap-soil contact. Close to the pile tip, both $\Delta\sigma'_h$ and Δf_s increase due to the effect of pile failure, Figures 7e and 7f.

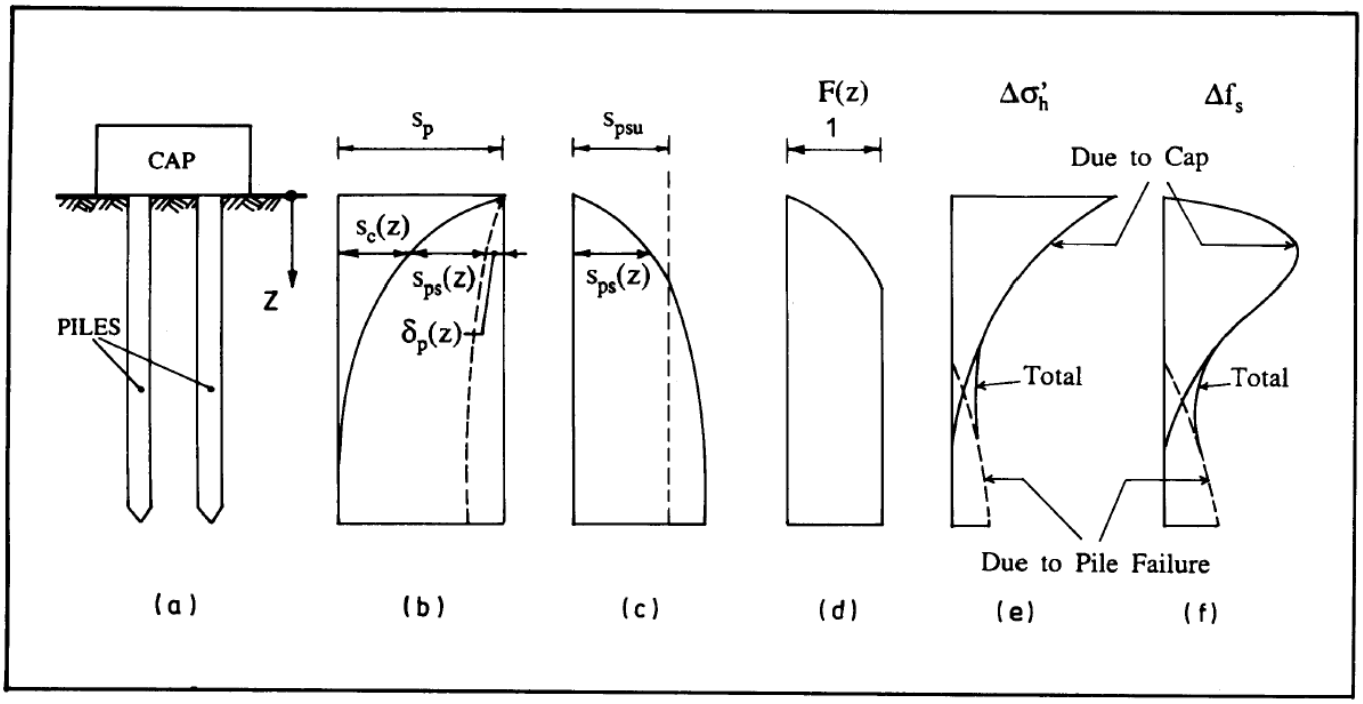

Figure 7. Increase in skin friction along a pile, due to effect of cap being in contact with soil surface and effect of failure zone at pile base, Phung (1993).

3.4 Simplified design method

From the test results we see that when the load is applied on a piled footing, the piles first take a major portion of the load, and only after pile failure, the load is considerably transferred to the cap. This means that the piles are close to failure (with a safety factor close to unity). We also see that the load taken by cap in the piled footing is very close to the load taken by cap alone. This means that the load-settlement relationship of the footing in a piled footing can then be estimated as that of the footing without piles under the same load. From these conclusions, a practical procedure of design of piled footing in sand can be carried out with the steps below:

1) *To estimate the load taken by the cap* (or unpiled raft) without causing excessive settlement. This load is equal to that can be taken in the cap in the piled footing P_{cap};
2) *To estimate the load taken by the piles* $P_{piles} = P_{total} - P_{cap}$, where P_{total} is the total applied load;
3) *To determine the number of piles*: As the piles are very close to failure state, the number of piles can be calculated as: $n = P_{piles} / P_s$, in which P_s is ultimate capacity of a single pile.

In Step 1, the load-settlement relationship is first estimated for the raft/footing without piles using any available method for shallow footings. The load taken by cap can be chosen at a chosen (allowable) settlement level. In Step 2, the remaining load will be taken by the piles. In Step 3, if we do not know about the pile-soil-pile interaction factor η_1 and the pile-cap interaction factor η_4, both the factors can be taken as unity. And the number of piles can be estimated by dividing the load taken by pile to the failure or creep load, of a single pile. This is on the safe side because under the cap-soil contact pressure the pile shaft resistance increase considerably.

The proposed method of settlement analysis was exemplified for all the three test series (Phung, 1993). The estimated settlements were quite comparative with the measured results. Poulos & Makarchian (1996) used this method to estimate the settlement of the model footing in their study and found a fair agreement with the test results.

*Example: To determine the number of piles to control the settlement for a square raft footing with a width B= 40m, in a soil with E_i = 30MPa, ν = 0.3 under an uniformly distributed load q = 50kPa, or a total load of P_{total} = 40m*40m*50kPa = 80000 kN = 80MN; and assuming the ultimate/failure load of a single pile P_S= 1500 kN. Settlement of a rigid square footing on a semi-infinite homogeneous elastic solid can be estimated as*

$$s = \frac{0.815 \cdot q \cdot B \cdot (1-\nu^2)}{E_i} = \frac{0.815 \cdot P \cdot (1-\nu^2)}{E_i \cdot B}$$

If the design settlement s = 40mm, P_{cap} = 64,720 kN = 64.72 MN. The load taken by piles will be $P_{piles} = P_{total} - P_{cap}$= 80 – 64.72 = 15.3 MN. The number of piles needed is n = P_{piles} / P_s = 15,300kN /1500kN = 10 piles. If the design settlement s = 20mm, P_{cap} = 32360 kN= 32.36 MN. The load taken by piles will be P_{piles}= 47.6 MN. The number of piles needed is n = P_{piles} / P_s= 47600 kN /1500 kN = 32 piles. If the conventional pile design approach is used, with a safety factor F_s=3, the number of piles needed is n = P_{total} / (P_s/F_s) = 80000/(1500/3) =160 piles.

This simple example indicates that: *if we know the load-settlement curve of a shallow footing and the failure load of a single pile we can predict the load-settlement curve of a piled footing*. Using the piled raft concept with settlement-reducing piles, the number of piles needed to control settlement is much smaller than that needed in the conventional pile footing design. Moreover with a bigger settlement allowed, the number of piles can be reduced considerably. This simplified calculation method is good enough for the concept design phase.

The Author also tried to make a relation between the so-called *relative cap capacity* α_c, which was defined as the ratio of the load applied on the shallow footing to that applied on the corresponding piled footing at a certain settlement, and the settlement ratio ξ_7, Phung (1993). The relative cap capacity shows the relative contribution of a cap to the total bearing capacity of a piled footing. With a chosen settlement of 5 mm, the α value is 0.27, 0.48 and 0.55 for Tests T1, T2 and T3, respectively. There are two extreme points: a) too many piles (pile footing); α_c = 0, and the settlement is close to zero; and b) no pile (shallow footing), α_c = 0, and ξ_7 = 1. The ratio ξ_7 can then be plotted versus the relative cap capacity α_c for different load levels between 60% and 120% of the failure load of the cap alone, P_{cf}, see Figure 8.

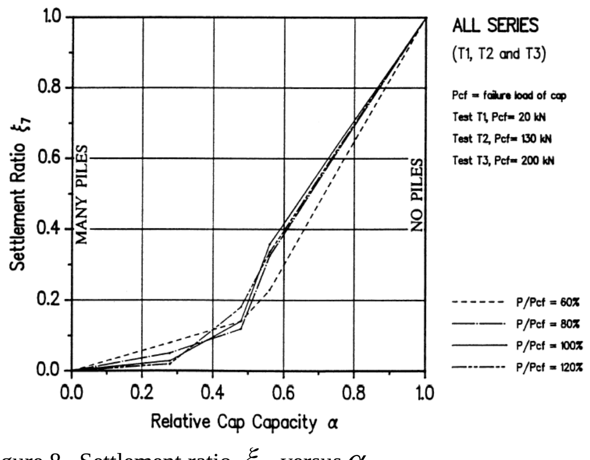

Figure 8. Settlement ratio ξ_7 versus α_c.

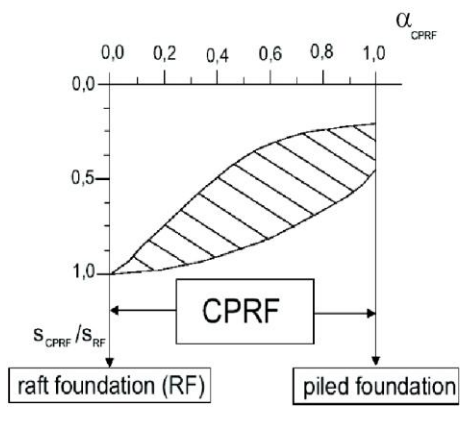

Figure 9. Settlement ratio ξ_7 versus α_{CPRF}.

This figure shows a clear tendency that when α_c is smaller than about 0.5, the settlement ratio ξ_7 decreases slowly with a decreasing α_c value. In other words, with α_c less than 0.5, a considerable increase in pile capacity (induced by increasing the number of piles or the pile length*) will not lead to* a significant further reduction in the settlement of the footing. However, with α_c higher than 0.5, i.e. when the cap contributes a major part to the capacity of a piled footing, the presence of piles has a clear effect in reducing the settlement of piled footings. This can be seen from the illustrated example above.

Figure 8 can also be used for a quick estimation of the settlement-reducing effect. As an example, let us assume that the cap has a capacity of 20MN, the settlement-reducing piles have a total capacity of 10MN. The relative cap capacity α_c is therefore 2/3. From Fig. 8, the settlement ratio ξ_7 is about 0.5, which means a settlement reduction of 50%.

It is very interesting that many years later a similar relationship was made from case histories in Germany (Katzenbach et al. 2003 and El-Mossallamy et al. 2006), see Figure 9. In the figure, the settlement ratio s_{CPRF} / s_{RF} was is the ratio between the settlement of a combined piled raft foundation (CPRF) and that of a raft foundation (RF), which is exactly the same definition of ξ_7 ; and α_{CPRF} is the ratio between the pile load share and that the total load on a piled footing. It can be easily seen the relation between α_{CPRF} and α_c , the relative cap capacity defined by the Author above: $\alpha_{CPRF} = 1 - \alpha_c$. It is easy to get the two graphs having the same co-ordinates by turning 180^o Figure 9. The two graphs are surprisingly in good agreement.

4. DESIGN APPROACHES

In the last decades, there has been considerable development of methods of calculating settlement for (free-standing) pile groups and piled footings, several of which are suggested to be used for footings with settlement reducing piles. However, most of the methods are based on the *theory of elasticity* and are therefore unsuitable for piled footings with settlement-reducing piles, especially in non-cohesive soil.

Piled raft foundation is a complicated soil-structure interaction problem. Many methods of analyzing piled rafts have been developed, and can be classified to four broad groups: 1) Simplified calculation methods; 2) Approximate computer-based methods; 3) More rigorous computer-based methods; and 4) Accurate numerical methods, as FEM. The methods were reviewed and discussed elsewhere (Phung, 1993; and Poulos, 2001).

For practical design, a problem should be first solved using simplified and less time-consuming methods, especially for feasible foundation option study. Detailed design of piled raft foundation for high-rises should be done by numerical analyses using FEM or explicit finite difference codes. This is a must in high-rise buildings especially when they become higher and heavier, and more complex in configurations. There are number of commercial codes available, both in 2D and 3D versions. The most common softwares are:

- PLAXIS 2D and 3D, Finite Element Code for Soil and Rock Analyses
- FLAC 2D and 3D, Fast Lagrangian Analysis of Continua
- ABAQUS 2D and 3D, general-purpose nonlinear finite element software
- DIANNA & Midas GTS

FEM is very effective tool for analysing any foundation and structure system. However, it is too complicated and time consuming to simulate a complicated soil-structure interaction problem as piled-raft foundation. There are a number of approaches that numerical analyses can be carried out:

- Full three-dimensional (3D) analysis,
- Equivalent two-dimensional (2D) plain strain model,
- Equivalent axi-symmetrical model.

Full 3D numerical analyses were almost impossible for complicated foundation configurations until this decade when the softwares could be developed due to faster computers. It is only recently that this technology has become a viable option to the engineers in the design office. This evolution may be explained by several factors. Pile groups and piled rafts are challenging design problems in the sense that they are 3D by nature and that soil-structure interaction is central to the behaviour of deep foundations. Although the background theory and the numerical tools necessary to model such deep foundation systems have been available for years, it is only in the last few years that available commercial softwares have reached a degree of maturity and user friendliness necessary to meet the needs of the design office.

4.1. Analysis of Piled Raft Foundation of ICC Tower

This is an example of simulating a piled-raft foundation using the approach of equivalent axi-symmetrical model, performed by the Author, (Phung, 2002). ICC Tower in Hong Kong is nowadays the forth tallest building in the world with a height of 484m and 118 stories, Figure 10. The foundation for the tower has a circular plan, and consists of 240 shaft-grouted barrettes (2.8m x 1.5m or 2.8m x 1.0m) within a circular perimeter shaft-grouted diaphragm wall (DW), see Figures 12 and 13. Below the raft, the soil profile consists of alluvium and CDG overlying rock. Within the basement area, rockhead level varies between -61mPD and -106mPD under ground surface. To minimise differential settlement the barrettes and DW panels are generally placed at a depth of about 2m above rockhead. The barrettes have thus a length varying between 35m and 70m. An 8m-thick base raft connects the barrettes and the DW. The excavation, 26m deep, is required for the construction of the 4-level basement and the pile cap.

The foundation was designed by the project engineers, as a *conventional pile foundation*, using the finite element program SAFE. This design is not discussed here. The Author, as the independent verifier, re-simulated the foundation using the FEM code PLAXIS Version 7.2, Phung (2002). The analysis is based on an axi-symmetric model with the barrettes and DW simulated as equivalent concentric rings. The objective of the analysis is to study the settlement behaviour of the foundation system, the load sharing between the foundation components, the barrettes, the DW panels and the raft. The 240 barrettes were modelled as 8 circular concentric rings representing the same surface areas of the barrettes. The barrette rings were modelled as a linear elastic material with an equivalent Young's modulus for bending E_1, and an equivalent Young's modulus for axial loading E_2. The DW was also included in the model as a ring. This allows the DW to carry part of the load as a component of the pile group. The DW and the raft were modelled as a linear elastic material with a long-term elastic modulus E for concrete. Soils were modelled as elasto-plastic materials with Mohr-Coulomb failure criterion, see Figure 11.

The settlement at the raft bottom level is about 40mm at the centre and 9mm at the DW edge. This compares quite well with the project engineer's estimated settlements. The loads at the head of the pile rings were calculated and the results show that the central piles carry higher loads than the boundary piles. The foundation was designed as a *conventional pile foundation*, but the Author's analysis indicates a major part, up to 30% of the total load, is carried by the raft. It is very common that the foundation is designed as a pile foundation, but acting as a combined piled-raft-foundation.

4.2 3D Finite Element Modelling

3D FEM is nowadays used to design almost all of the tallest high-rises. 3D FEM analysis with appropriate soil constitutive laws is a powerful tool to model this complex piled-raft foundation problem. However, the main disadvantage with applying the 3D FE analyses is the need of a huge number of volume elements which can exceed the available computer capacities. To cover this problem, a new technique combined the so called *embedded pile model* with the 3D finite element model was developed by Plaxis. Figure 14 shows an example of FE model for a piled raft with more than 600 piles using Plaxis 3D Foundation Version 2 (Schweiger, 2008; Brikgreve, 2008).

Figure 10. ICC Tower in Hong Kong

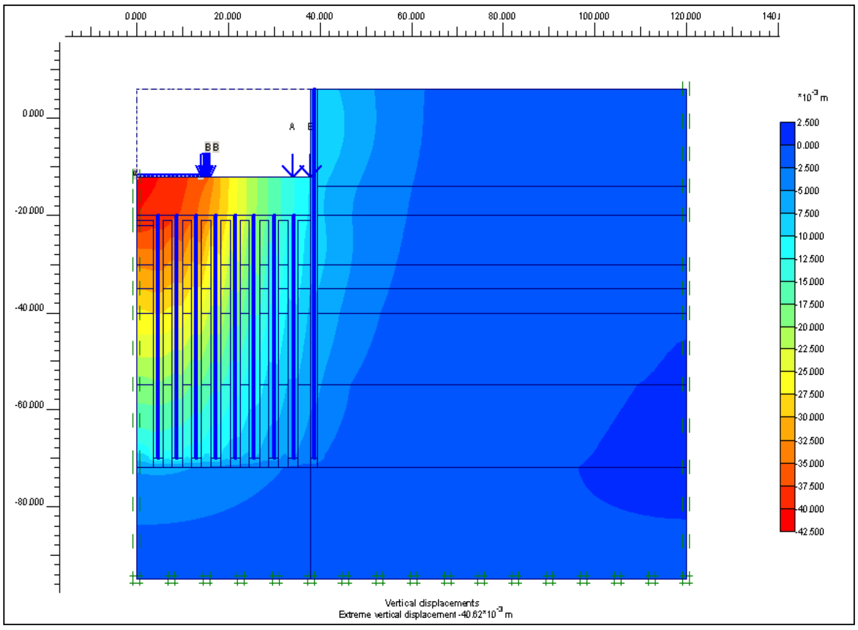

Figure 11. ICC Tower-Axisymetrical modelling foundation using Plaxis, Phung (2002)

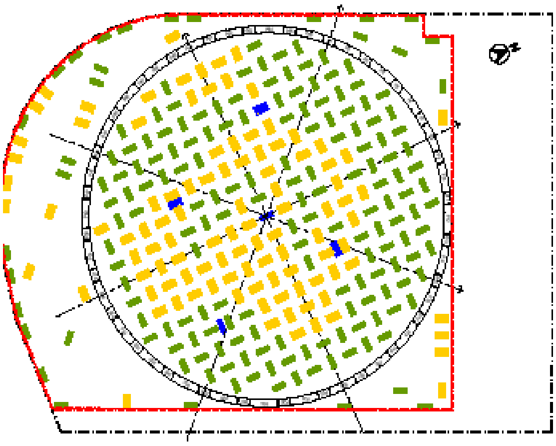

Figure 12. ICC Tower-Foundation plan

Figure 13. ICC-Foundation under construction

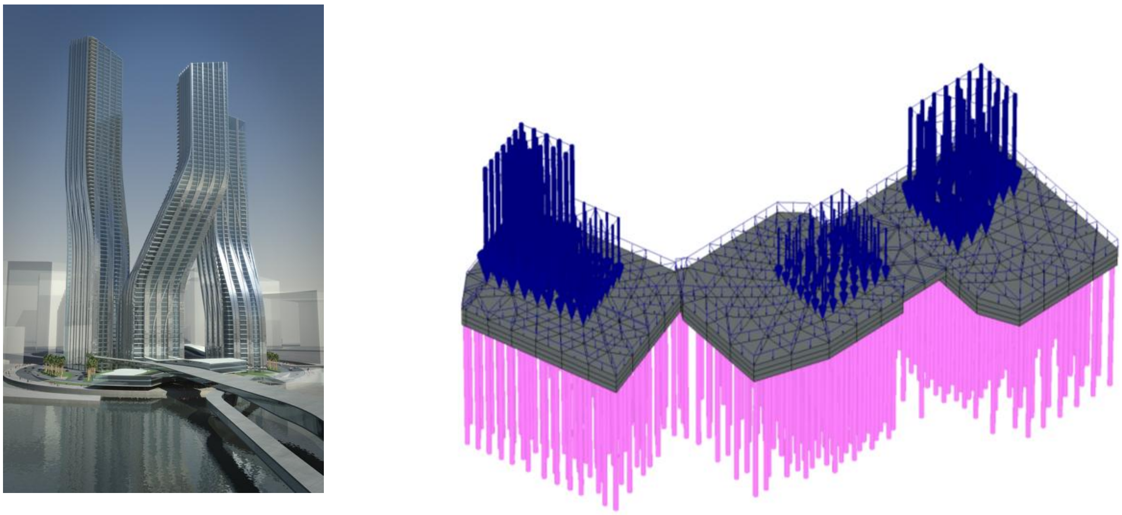

Figure 14. 3D-Modelling a piled-raft using Plaxis 3D Foundation –Vers 2. (Brikgreve, 2008)

5. CONCLUSIONS

Predicting the settlement of piled footings is a difficult task for geotechnical engineers due to the complex pile-cap-soil interaction. The available prediction methods, which are based on the theory of elasticity, are not suitable for piled footings with settlement-reducing piles, especially in non-cohesive soil. Results of the experimental study, performed by the Author, have created a better understanding about the load-transfer mechanism of piled footings in sand, as well as the load-settlement behaviour. The study strongly supports the idea of settlement-reducing piles. The simplified methods suggested in this paper can be used as a practical design procedure, especially in the foundation option study phase. Detailed foundation design for high-rises must include 3D FEM analysis, which can be realised by different commercially available computer codes.

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