Piled Raft Foundations for Tall Buildings

H.G. Poulos[1], J.C. Small [1] and H. Chow[1]
[1]*Coffey Geotechnics, Sydney, Australia*
E-mail: harry_poulos@coffey.com

ABSTRACT: Piled raft foundations are increasingly being recognised as an economical and effective foundation system for tall buildings. This paper sets out some principles of design for such foundations, including design for the geotechnical ultimate limit state, the structural ultimate limit state and the serviceability limit state. The advantages of using a piled raft will then be described with respect to two cases: a small pile group subjected to lateral loading, and then the design of the Incheon Tower in South Korea. Attention will be focussed on the improvement in the foundation performance due to the raft being in contact with, and embedded within, the soil.

KEY WORDS: Piled rafts, design, lateral load, case studies.

1. INTRODUCTION

High rise buildings are usually founded on some form of piled foundation which is subjected to a combination of vertical, lateral and overturning forces. Combined pile-raft foundations can be a particularly effective form of foundation system for tall buildings because the raft is able to provide a reasonable measure of both stiffness and load resistance.

This paper sets out a limit state design approach for tall building foundation systems, with attention being focused on piled raft foundation systems. Some of the advantages of piled rafts are outlined, and then the principles of the design approach are set out. A published case history involving a 9-pile group is then analysed to compare the performances of a pile group and a piled raft. An example of the application of the design approach is then described for a proposed tower on reclaimed land in Incheon, South Korea.

2. ADVANTAGES OF PILED RAFT FOUNDATIONS

Piled raft foundations utilize piled support for control of settlements with piles providing most of the stiffness at serviceability loads, and the raft element providing additional capacity at ultimate loading. Consequently, it is generally possible to reduce the required number of piles when the raft provides this additional capacity. In addition, the raft can provide redundancy to the piles, for example, if there are one or more defective or weaker piles, or if some of the piles encounter karstic conditions in the subsoil. Under such circumstances, the presence of the raft allows some measure of re-distribution of the load from the affected piles to those that are not affected, and thus reduces the potential influence of pile "weakness" on the foundation performance.

Another feature of piled rafts, and one that is rarely if ever allowed for, is that the pressure applied from the raft on to the soil can increase the lateral stress between the underlying piles and the soil, and thus can increase the ultimate load capacity of a pile as compared to free-standing piles (Katzenbach et al., 1998).

A geotechnical assessment for design of such a foundation system therefore needs to consider not only the capacity of the pile elements and the raft elements, but their combined capacity and interaction under serviceability loading.

The most effective application of piled rafts occurs when the raft can provide adequate load capacity, but the settlement and/or differential settlements of the raft alone exceed the allowable values. Poulos (2001) has examined a number of idealized soil profiles, and found that the following situations may be favourable:

- Soil profiles consisting of relatively stiff clays
- Soil profiles consisting of relatively dense sands.

In both circumstances, the raft can provide a significant proportion of the required load capacity and stiffness, with the piles acting to "boost" the performance of the foundation, rather than providing the major means of support.

3. DESIGN PRINCIPLES

3.1 Design Issues

The following issues usually need to be addressed in the design of foundations for high-rise buildings:

1. Ultimate capacity of the foundation under vertical, lateral and moment loading combinations.
2. The influence of the cyclic nature of wind, earthquakes and wave loadings (if appropriate) on foundation capacity and movements.
3. Overall settlements.
4. Differential settlements, both within the high-rise footprint, and between high-rise and low-rise areas.
5. Structural design of the foundation system; including the load-sharing among the various components of the system (for example, the piles and the supporting raft), and the distribution of loads within the piles. For this, and most other components of design, it is essential that there be close cooperation and interaction between the geotechnical designers and the structural designers.
6. Possible effects of externally-imposed ground movements on the foundation system, for example, movements arising from excavations for pile caps or adjacent facilities.
7. Earthquake effects, including the response of the structure-foundation system to earthquake excitation, and the possibility of liquefaction in the soil surrounding and/or supporting the foundation.
8. Dynamic response of the structure-foundation system to wind-induced (and, if appropriate, wave) forces.

In this paper, attention will be concentrated on the first five design issues.

3.2 Design Requirements

In limit state format (for example, as per the Australian Piling Code AS2159-1995), the design criteria for the ultimate limit state may be expressed as follows:

$$R_s^* \geq S^* \quad (1)$$

$$R_g^* \geq S^* \quad (2)$$

where R_s^* = design structural strength = $\phi_s . R_{us}$
R_g^* = design geotechnical strength = $\phi_g . R_{ug}$
R_{us} = ultimate structural strength
R_{ug} = ultimate geotechnical strength (capacity)
ϕ_s = structural reduction factor
ϕ_g = geotechnical reduction factor
S^* = design action effect (factored load combination)

The above criteria in equations 1 and 2 are applied to the entire foundation system, while the structural strength criterion (equation 1) is also applied to each individual pile. However, it is not considered to be good practice to apply the geotechnical criterion (equation 2) to each individual pile within the group, as this can lead to considerable over-design (Poulos, 1999).

R_s^* and R_g^* can be obtained from the estimated ultimate structural and geotechnical capacities, multiplied by appropriate reduction factors. Values of the structural and geotechnical reduction factors are often specified in national codes or standards. The selection of suitable values of ϕ_g requires judgment and takes into account a number of factors that may influence the foundation performance.

In addition to the normal design criteria, as expressed by equations 1 and 2, additional criterion can be imposed for the whole foundation of a tall building to cope with the effects of repetitive loading from wind and/or wave action, as follows:

$$\eta \, R_{gs}^* \geq S_c^* \tag{3}$$

where R_{gs}^* = design geotechnical shaft capacity
S_c^* = maximum amplitude of wind loading force
η = a factor assessed from geotechnical laboratory testing.

This criterion attempts to avoid the full mobilization of shaft friction along the piles, thus reducing the risk that cyclic loading will lead to a degradation of shaft capacity. A value of η of 0.5 is suggested in the absence of other information. S_c^* can be obtained from computer analyses which gave the cyclic component of load on each pile, for various wind loading cases.

3.3 Design Implementation

3.3.1 Overall Stability

For consideration of the overall stability of the foundation system, an analysis is carried out in which the geotechnical and structural resistances of the foundation components are reduced by the appropriate geotechnical reduction factor and the ultimate limit state (ULS) load combinations are applied. The design requirements in equations 1 and 2 will be satisfied if the foundation system does not collapse under any of the sets of ULS loadings. In addition, a check can be made of the cyclic actions generated in the foundation elements to assess whether the cyclic loading requirement (equation 3) is satisfied.

If any of the above requirements are not satisfied, then the design will need to be modified accordingly to increase the strength of the overall system or of those components of the system that do not satisfy the criteria.

3.3.2 Serviceability

For the serviceability analysis, the best-estimate (unfactored) values of foundation resistances and stiffnesses are employed and the serviceability limit state (SLS) loads are applied. The design will be satisfactory if the computed deflections and rotations are within the specified allowable limits.

3.3.3 Structural Design Requirements

For structural design of the raft and the piles, the results of the ULS analysis are not considered to be relevant because the loads that can be sustained by the piles are artificially reduced by the geotechnical reduction factor. Consequently, it is suggested that the most rational approach is one in which a separate ULS analysis is carried out using the various ULS load combinations but in which the <u>unfactored</u> resistances of the foundation components are employed. The consequent computed foundation actions (i.e. pile forces and, if appropriate, raft moments and shears) are then multiplied by a structural action factor (for example 1.5) to obtain the values for structural design.

4. EXAMPLE OF PILED RAFT vs PILE GROUP PERFORMANCE

4.1 Introduction

To gain an understanding of the significance of including the raft in the foundation analysis, a relatively simple example of a field test is considered first. A lateral load test was performed by Rollins and Sparks (2002) on a group of 9 piles having a cap that was buried in fill as shown in Figure 1. The piles were 324mm outside diameter steel pipe piles with a wall thickness of 9.5mm (elastic modulus of the steel E_{steel} = 200GPa), and were driven to about 9.1m. Prior to testing, the piles were instrumented and then the pipe filled with concrete. The piles were spaced at a nominal spacing of 3 pile diameters.

The pile cap was 2.74m square and 1.22m deep and extended 0.3m beyond the outer edge of the piles. Horizontal load was applied to the pile cap using two hydraulic jacks such that the load was applied 0.4m above the base of the pile cap.

The soil consisted of various layers of clay, sand and sandy silt. The soil profile and the soil parameters used in the analysis are shown in Figure 2. The unit weight of the fill was 24kN/m^3 and that of the soil was taken as 18kN/m^3.

The passive pressure against the face of the pile cap was treated as a load on the pile cap that increased with displacement and then reached a constant value at passive failure. The passive force on the cap was calculated to be 755kN assuming a log spiral failure mode and an angle of shearing resistance of 36^o. The displacement at which the passive pressure reached a maximum was taken to occur at a cap displacement to cap height ratio of 0.04, based on experimental data for retaining walls. The friction on the base of the cap was calculated from the normal load of the cap (that is due to the self weight of the cap) times the tangent of the angle of friction between the cap and the bedding layer that was taken as 36^o. There was no friction against the sides of the cap in the test as the cap was cast in a trench.

Yield pressures against the face of the piles were taken to be 9s_u (where s_u is undrained shear strength) as is customary in lateral pile loading, and soil moduli were taken to be 200s_u in the clay and silt layers and $3K_p\sigma'_v$ in the sand layers (where K_p is the passive earth coefficient and σ'_v is the vertical effective stress). For the upper soil layers, the lateral resistance is often neglected over one diameter to allow for surface disturbance, but here the lateral resistance was reduced by 0.7 for the upper three layers.

4.2 Lateral Foundation Responses

Figure 3 shows the measured load-deflection relationship and those computed by using the Finite Layer program APRILS (Analysis of Piled Rafts In Layered Soils; Chow (2007)) for three cases:

1. A free-standing pile group, with no contact between the raft and the underlying soil;
2. A piled raft, with contact between the raft and the underlying soil, but no contact between any of the sides of the raft and the soil. Normal load is due to the weight of the raft only;
3. A piled raft, with the raft in contact with both the underlying soil and with the soil in front of the raft (in the direction of displacement), but no side contact. Normal load is due to the weight of the raft only.

The following observations can be made:

(a) When the passive pressure acting on the side of the pile cap and the friction on the base of the cap is neglected, the computed deflections are much larger than the measured values.

(b) The effect of the passive resistance on the lateral deflection is much larger than the effect of friction on the base of the cap which is fairly small for this problem (due to weight of cap only).

(c) By considering forces acting on the cap, a fairly good estimate can be made of the measured load-deflection behaviour of the piled raft subjected to lateral loading as shown in Figure 3.

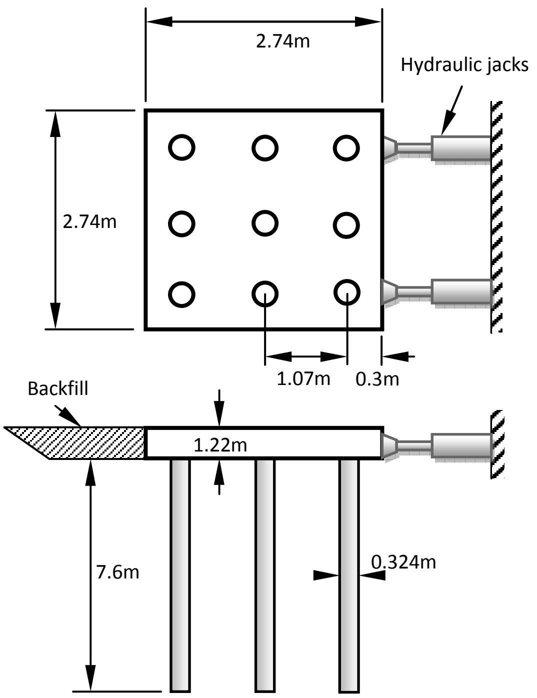

Figure 1. Lateral pile loading test layout

Figure 2. Soil properties used in analysis.

Material		Thickness (m)	Strength	OCR	Modulus (MPa)	Lateral yield pressure p_y (kPa)
Compacted sandy GRAVEL fill		1.22	$\phi' = 42^o$	—	Against cap	Against cap
Bedding	∇	0.15	$\phi' = 42^o$	—	45	239
Sandy SILT - ML		0.6	s_u = 37kPa	5	7.4	233
Gray CLAY - CL		1.5	s_u = 42kPa	3	9	264
Light grey Sandy SILT - ML		0.85	s_u = 30kPa	2.1	6	189
Poorly graded light brown SAND - SP		1.7	$\phi' = 39^o$	—	45	844
Grey Clay - CH		0.3	s_u = 25kPa	1.25	5	225
Grey clay - CL		0.3	s_u = 25kPa	1.25	5	540
Light grey SILT - ML		0.6	s_u = 60kPa	1.4	12	540
Light brown Sandy SILT - ML		0.9	s_u = 60kPa	1.25	12	540
Light grey Silty SAND - SM		1.4	$\phi' = 36^o$	—	30	1121
Grey SILT - ML		1.5	s_u = 75kPa	1.13	15	675

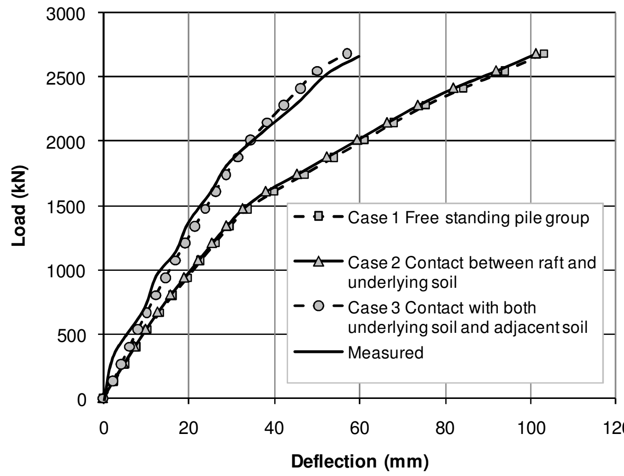

Figure 3. Load-deflection behaviour considering different pile cap conditions.

4.3 Vertical Foundation Response

Although no vertical load test was carried out on the group in Figure 1, it was considered instructive to compare the computed load-settlement behaviours for the 9-pile group and the piled raft with 9 piles. The program APRILS was again used, and the computed load-settlement curves are shown in Figure 4. It can be seen that the piled raft has a slightly stiffer initial response but a much longer "tail" after the pile capacities are fully mobilized and the raft continues to carry additional load. The redundancy provided by such a system is thus demonstrated.

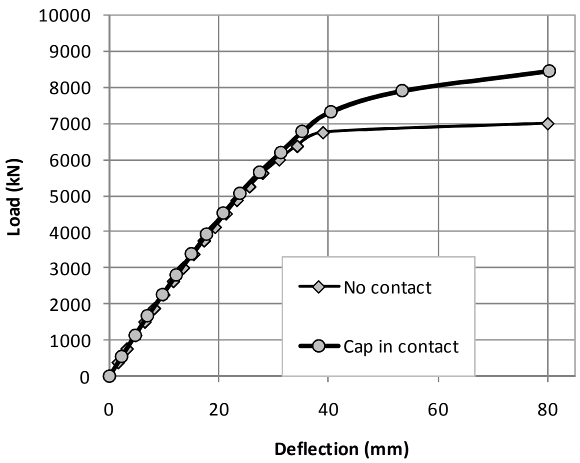

Figure 4. Computed load-settlement curves for pile group and piled raft.

5. INCHEON TOWER, KOREA

5.1 Introduction

Currently, a 151 storey super high-rise building project is under design, located in reclaimed land constructed on soft marine clay in Songdo, Korea. The foundation system considered comprises 172 x 2.5m diameter bored piles, socketed into the soft rock layer and connected to a 5.5m thick raft. This building is illustrated in Figure 5 and is described in detail by Badelow et al. (2009); thus, only a brief summary is presented here.

5.2 Ground Conditions and Geotechnical Model

The Incheon area has extensive sand/mud flats and near shore intertidal areas. The site lies entirely within an area of reclamation, which is likely to comprise approximately 8m of loose sand and sandy silt, constructed over approximately 20m of soft to firm marine silty clay, referred to as the Upper Marine Deposits (UMD). These deposits are underlain by approximately 2m of medium dense to dense silty sand, referred to as the Lower Marine Deposits (LMD), which overlie residual soil and a profile of weathered rock.

The lithological rock units present under the site comprise granite, granodiorite, gneiss (interpreted as possible roof pendant metamorphic rocks) and aplite. The rock materials within about 50 metres from the surface have been affected by weathering which has reduced their strength to a very weak rock or a soil-like material. This depth increases where the bedrock is intersected by closely spaced joints, and sheared and crushed zones that are often related to the existence of the roof pendant sedimentary / metamorphic rocks. The geological structures at the site are complex and comprise geological boundaries, sheared and crushed seams - possibly related to faulting movements, and jointing.

Figure 5. Incheon 151 Tower (artist's impression)

From the available borehole data for the site, inferred contours were developed for the surface of the "soft rock" founding stratum within the tower foundation footprint and it was found that there was a potential variation in level of the top of the soft rock (the pile founding stratum) of up to 40m across the foundation.

The footprint of the tower was divided into eight zones which were considered to be representative of the variation of ground conditions and geotechnical models were developed for each zone. Appropriate geotechnical parameters were selected for the various strata based on the available field and laboratory test data, together with experience of similar soils on adjacent sites. One of the critical design issues for the tower foundation was the performance of the soft UMD under lateral and vertical loading, hence careful consideration was given to the selection of parameters for this stratum. Typical parameters adopted for the foundation design are presented in Table 1.

Table 1. Summary of Geotechnical Parameters

Strata	E_v MPa	E_h MPa	f_s kPa	f_b MPa
UMD	7 – 15	5 -11	29 - 48	-
LMD	30	21	50	-
Weathered Soil	60	42	75	-
Weathered Rock	200	140	500	-
Soft Rock (above EL-50m)	300	210	750	12
Soft Rock (below EL-50m)	1700	1190	750	12

E_v = Vertical Modulus f_s = Ultimate shaft friction
E_h = Horizontal Modulus f_b = Ultimate end bearing

5.3 Foundation Layout

The foundation comprises a 5.5 m thick concrete mat and piles supporting columns and core walls. The numbers and layout of piles and the pile size were obtained from a series of trial analyses through collaboration between the geotechnical engineer and the structural designer. The pile depth was determined by considering the performance and capacity of piles of various diameters and length. The pile depths required to control settlement of the tower foundation were greater than those required to provide the geotechnical capacity required. The pile design parameters for the weathered/soft rock layer are shown in Table 2 and were estimated on the basis of the pile test results in the adjacent site and the ground investigation data such as pressuremeter tests and rock core strength tests.

The final design employed 172 piles of 2.5m diameter, with lengths below the base of the raft varying from about 36m to 66 m, depending on the depth to the desired founding level. The base of the raft was about 14.6m below ground surface level. The pile layout was selected from the various options considered, and is presented in Figure 6.

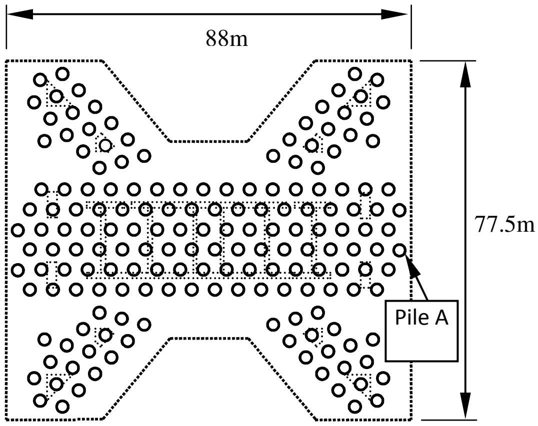

Figure 6. Foundation Layout

Table 2. Ultimate Capacities for Pile Analysis

Material	Ultimate Friction f_s(kPa)	Ultimate End Bearing f_b(MPa)
Weathered Rock	500	5
Soft Rock	750	12

5.4 Overall Stability

The ultimate limit state (ULS) combinations of load were input into a series of non-linear pile group analyses using a computer program CLAP (Combined Load Analysis of Piles) developed by Coffey (2007). The pile axial and lateral capacities were reduced by geotechnical reduction factors of 0.65 for axial load, and 0.40 for lateral load). The smaller factors for lateral load reflected the greater degree of uncertainty for lateral response. In all cases analyzed, the foundation system was found to be stable, i.e. the computed foundation movements were finite, and generally the maximum computed settlement under the ULS loadings was less than 100mm.

5.5 Predicted Performance Under Vertical Loading

For the settlement analysis of the foundation system, the computer program GARP (Small and Poulos, 2007) was used as the main analysis tool. However, to provide a check on the GARP analyses, analyses were also carried out using the commercially-available program PLAXIS 3-D Foundation. For the purposes of this paper, the PLAXIS analyses were extended to examine the effects of including the presence of the raft, and two separate cases were analysed:

1. The piles being connected to the raft which is in contact with the underlying soil but not with the surrounding soil above the raft base level (Case 1). This is the usual case considered for a piled raft, where only contact below the raft is taken into account.
2. The piles being connected to the raft, which is in contact with both the underlying soil and the soil surrounding the basement walls of the foundation system (Case 2). This is the actual case that is to be constructed. In this case, account is taken of vertical basement walls that are 9.1m high (or 14.6m high if we include the 5.5m thick raft) and 1.2m in thickness.

In the finite element analysis, plate elements had to be fixed to the bottom of the solid elements of the raft and the pile heads fixed to the plate as this is required in PLAXIS if the pile heads are to rotate with the raft.

The finite element mesh for the problem (Case 2) is shown in Figure 7, and it may be seen that the soil is divided into layers representing the materials of Table 1. The sides of the excavation are supported by retaining walls that are modelled in the mesh. In the PLAXIS model, the soil layers were treated as being Mohr-Coulomb materials to allow for non-linear effects.

Table 3. Load components for Incheon Tower

Load component	Value
Dead Load	6036MN
Live Load	651MN
Horizontal load x-direction	149MN
Horizontal load z-direction	115MN
Earthquake load x-direction	110MN
Earthquake load z-direction	110MN
Moment in x-direction	21,600MN-m
Moment in z-direction	12,710MN-m

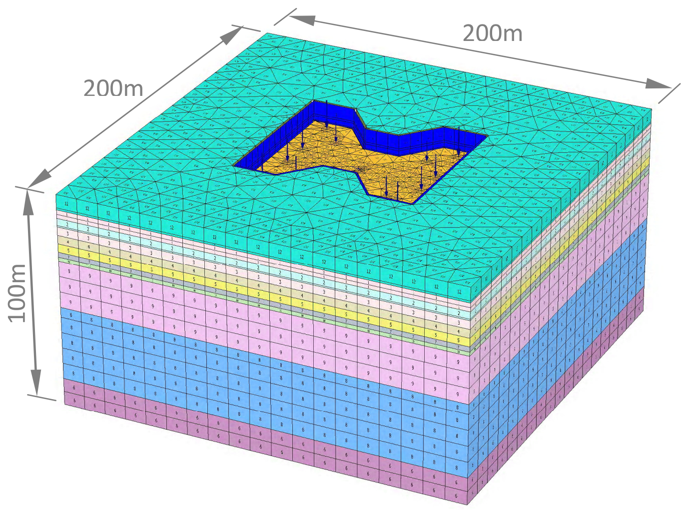

Figure 7. Finite element mesh showing material layers.

Load components applied to the foundation are shown in Table 3. Various combinations of these loads were applied to the structure in design, but here separate components were applied to gauge the effects of the raft burial.

5.5.1 Vertical loading

In the first instance, a vertical loading only (of 6687 MN) was applied to the foundation for both Case 1 and Case 2. The distribution of loads may be seen in Figure 8 with some column loads and some loads applied to the core. As may have been expected, when the soil at the sides of the basement are considered, the deflection of the raft is reduced.

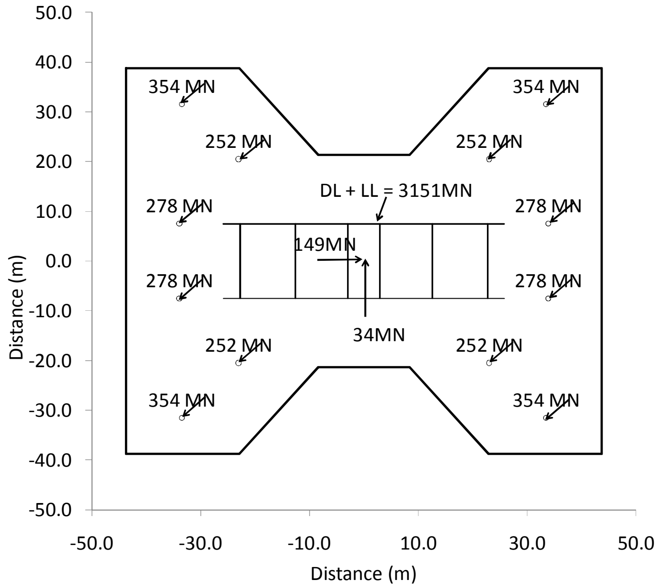

Figure 8. Magnitudes of loads applied to the raft

This may be seen from the load-deflection plot of Figure 9 where the percentage of load applied to the raft versus the vertical deflection is plotted. The reduction in vertical displacement caused by taking the embedment of the raft into consideration is about 8 mm in this case.

The deformed shape of the surface raft (Case1) is shown in Figure 10. It may be seen that the raft bends down at the corners and centre under the action of the vertical loads. When the basement and walls are considered (Case 2) the deformed shape of the raft (exaggerated) is as shown in Figure 11.

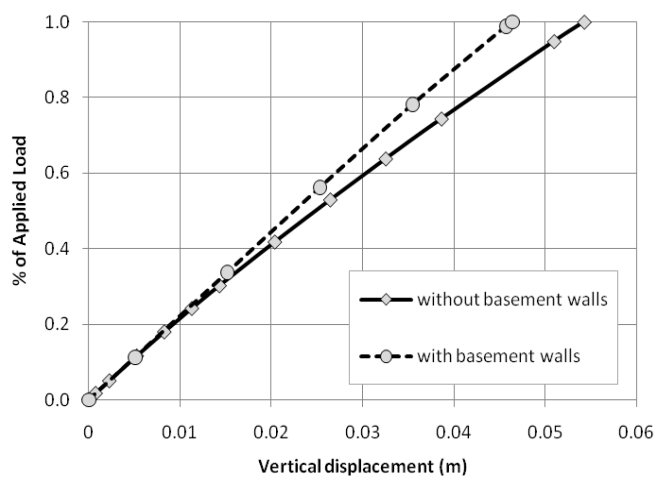

Figure 9. Load-deflection behaviour at central point of raft (vertical loading).

The GARP analysis used as the main design tool gave a maximum settlement of about 67mm and a maximum differential settlement of 34mm. These values are a little different to those from the PLAXIS analysis (56mm max. settlement and 40mm differential), and may reflect the inherent conservatism in the use of interaction factors within the GARP analysis. Nevertheless, the agreement between the two analyses is considered to be adequate.

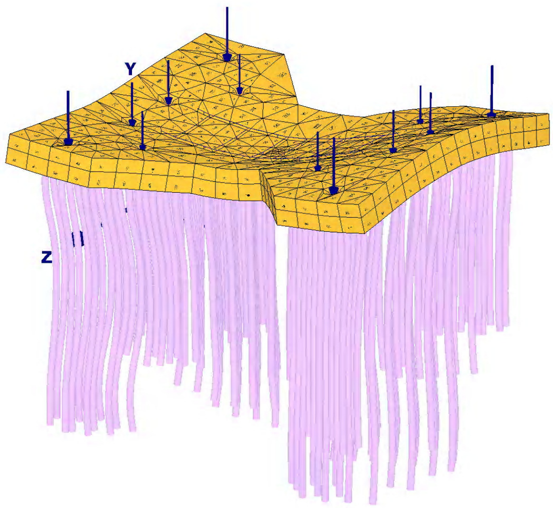

Figure 10. Deformed raft under vertical loading (Case 1)

5.5.2 Horizontal loading

In order to examine the effects of including the soil above the raft in the analysis, a PLAXIS 3-D analysis was undertaken for lateral loading only (of 149 MN in the x-direction and 34.5 MN in the z-direction (i.e. 30% of the total component) for both Case 1 and Case 2 as well as the case where the raft was assumed not to be in contact with the ground. The latter case was modelled in PLAXIS by placing a thin soft layer of soil underneath the raft. The results of the analysis showed that the predicted lateral deformation at working load was reduced when the contact or embedment of the foundation was taken into account as shown in Figure 12. This figure shows the lateral deflection at the central point of the raft versus the percentage of lateral load applied to the foundation.

Deformed meshes in the case of horizontal loading are presented in Figures 13 (Case 1) and 14 (Case 2). In the figures, the bending of the piles can be clearly seen (to an exaggerated scale), and this generates moments in the piles.

Because of the bending of the piles under lateral loading, it is of interest therefore to compare the moments induced in one of the piles in the leading row for each of the cases. This is shown in Figure 15 for a pile on the leading edge of the raft (pile A in Figure 6). It may be seen that the bending moments that were calculated are lower than for the conventional type of analysis (where the raft is assumed to make no contact) when the raft is being resisted by soil at the sides of the basement above raft level or the raft is in contact with the ground. However, in this case because of the large number of piles, the effect of the walls on the reduction of pile moment is small.

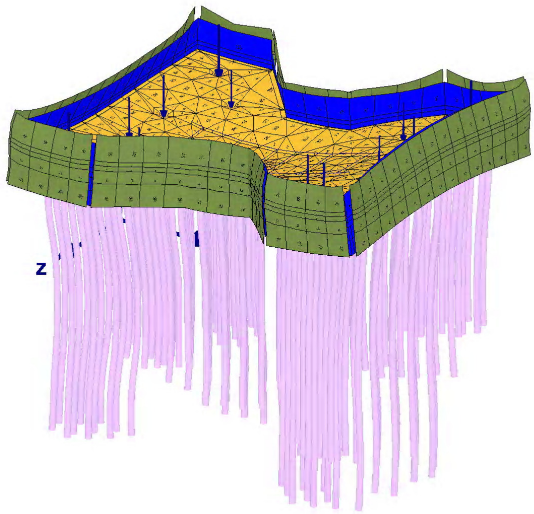

Figure 11. Deformed raft under vertical loading (Case 2).

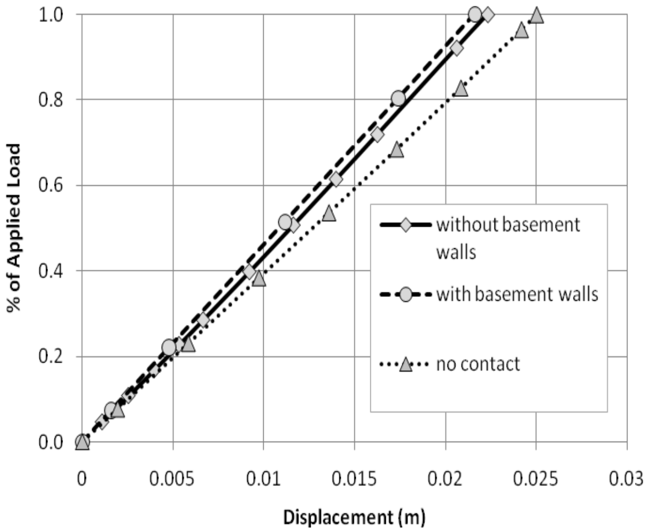

Figure 12. Load-deflection behaviour of central point of the raft (Horizontal loading).

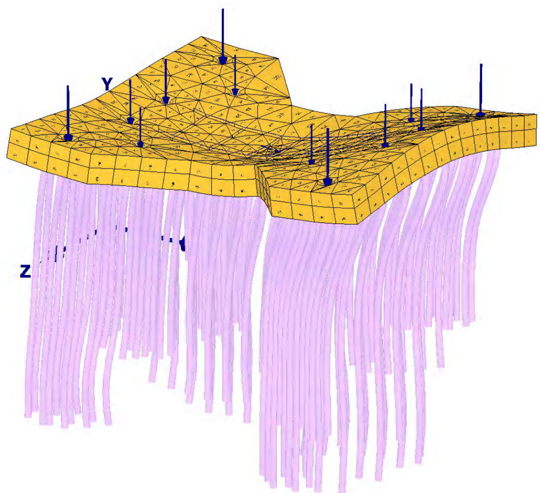

Figure 13. Deformed raft and piles under lateral loading in x-direction (Case 1)

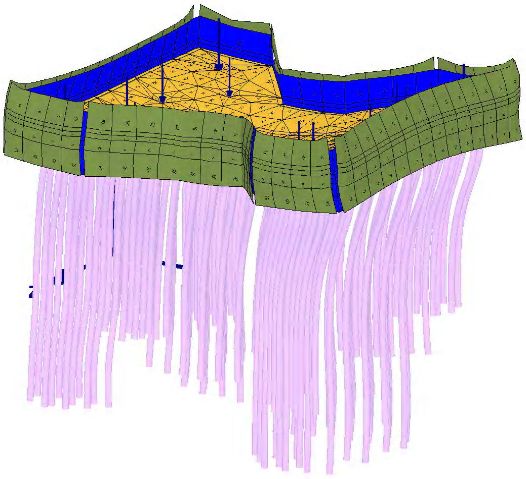

Figure 14. Deformed raft and piles under lateral loading in x-direction (Case 2)

A proprietary program, CLAP (Combined Load Analysis of Piles), which is modified version of the computer program DEFPIG (Poulos, 1990), was used as the main design tool for considering the lateral response of the foundation. It is interesting to note that CLAP gave a maximum lateral displacement of 22mm and a maximum pile bending moment of 15.7MNm. These values are comparable to those obtained from PLAXIS 3D and indicate that, for the design of piled rafts with a large number of piles, it is probably adequate to ignore the embedment of the cap when computing the lateral response of the foundation and the distribution of bending moment within the piles.

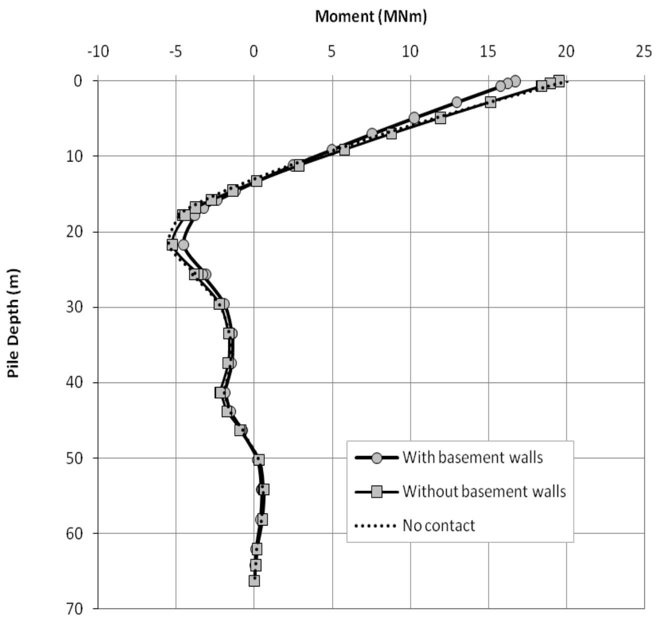

Figure 15. Moments in pile for horizontal loading Cases 1 and 2, and with no contact.

6. CONCLUSIONS

This paper sets out the principles of a limit state design approach for piled raft foundations for tall buildings. Ultimate limit state, serviceability limit state and cyclic loading conditions are addressed.

The effect of considering the embedment of the pile cap in estimation of piled raft behaviour has been examined for a small scale test and for a full sized structure.

The calculations for the small scale test indicate that the effect of the soil against the buried cap is quite significant, and therefore to use a conventional pile group analysis where this is neglected will result in a considerable underestimate of lateral load capacity and an overestimate of lateral deflection.

The finite element analyses of a full scale building (the Incheon Tower) have shown that by considering the effect of the basement, the estimated lateral deflections are smaller than the deflections of a raft on the surface only. Because the deflections are smaller, the moments in the piles are also smaller, but the effects are relatively modest in this case. This arises because the lateral stiffness of the 172 piles is large in comparison with that of the raft.

For the Incheon tower case and vertical loading, the effect of considering the embedment of the raft also has some effect on the predicted settlements, and has caused an 18% reduction in vertical settlement. A finding of practical importance is that for tall buildings supported by piled raft foundations with a large number of piles, a conventional pile group analysis may often be adequate, albeit conservative, for estimating the vertical and lateral behaviour of the foundation and the distributions of pile load and bending moment within the piles in the foundation system.

7. ACKNOWLEDGEMENTS

The authors gratefully acknowledge the contributions to the Incheon Tower analysis of Ms. Frances Badelow and Tristan McWilliam of Coffey Geotechnics, and the cooperation of Dr. Ahmad Abdelrazaq of Samsung Corporation and Mr. S.H. Kim of Jin Young ENC with structural and geotechnical aspects of the Incheon Tower.

8. REFERENCES

[1] Badelow, F., Kim, S., Poulos, H.G. and Abdelrazaq, A. (2009). "Foundation design for a tall tower in a reclamation area". Proc. 7th Int. Conf. Tall Buildings, Hong Kong, Ed. F.T.K. Au, Research Publishing, pp815-823.

[2] Chow, H.S.W. (2007) "Analysis of piled-raft foundations with piles of different lengths and diameters", PhD thesis, University of Sydney.

[3] Coffey (2007) CLAP (Combined Load Analysis of Piles) - User's Manual. Coffey Geotechnics, Sydney, Australia.

[4] Katzenbach, R., Arslan, U., Moorman, C. and Reul, O. (1998). "Piled Raft Foundations: Interaction Between Piles and Raft". Darmstadt Geotechnics, Darmstadt University of Technology, 4: pp279-296.

[5] Poulos, H.G. (1990). "DEFPIG users manual". Centre for Geotechnical Research, University of Sydney.

[6] Poulos, H.G. (1999). "The Design of Piles with Particular Reference to the Australian Piling Code", Australian Geomechanics, 34(4): pp25-39.

[7] Poulos, H.G. (2001). "Piled raft foundations: design and applications" , Géotechnique, 51 (2), pp.95-113.

[8] Small, J.C. and Poulos, H.G. (2007). "A method of analysis of piled rafts". Proc. 10th Australia New Zealand Conf. on Geomechanics, Brisbane, 1: pp550-555.

[9] Rollins, K.M. and Sparks, A. (2002). "Lateral resistance of full-scale pile cap with gravel backfill," J. Geotechnical and Geoenvironmental Engineering, ASCE, 128 (9): pp711-723.