COMPUTER DESIGN OF PILE FOUNDATION SYSTEMS FOR HIGH-RISE BUILDINGS

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ABSTRACT

High rise buildings are usually founded on some form of piled foundation which is subjected to a combination of vertical, lateral and overturning forces. Conventional methods of assessing foundation stability may not be adequate when designing such foundations because they tend to focus primarily on foundation resistance under vertical loading.

This paper sets out the principles of using computer programs for pile group response to design a pile foundation system according to the Australian Piling Code AS2159-1995. The approach involves three sets of analyses:

a. An overall stability analysis;
b. A serviceability analysis;
c. An analysis to obtain the structural actions for structural design of the raft and the piles.

The approach is illustrated via its application to a simple example problem and then to a high-rise building in the Middle East. The computer program DEFPIG has been used to assess overall foundation stability while the program GARP has been used for the serviceability analysis.

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. Conventional methods of assessing foundation stability may not be adequate when designing such foundations because they tend to focus primarily on foundation resistance under vertical loading.

This paper sets out the principles of using computer programs for pile group response to design a pile foundation system according to the Australian Piling Code AS2159-1995.

2 DESIGN REQUIREMENTS FOR PILED FOUNDATIONS

2.1 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.

2.2 DESIGN CRITERIA

The Australian Piling Code (1995) sets out the design criteria for the ultimate limit state as follows:

$$R_s^* \geq S^* \qquad \textbf{(1)}$$

$$R_g^* \geq S^* \qquad \textbf{(2)}$$

where R_s^* = design structural strength = $\phi_s . R_{us}$ **(3)**

R_g^* = design geotechnical strength = $\phi_g . R_{ug}$ **(4)**

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 judgement and takes into account a number of factors that may influence the foundation performance.

2.3 LOAD COMBINATIONS

The required load combinations for which the structure and foundation system have to be designed will usually be dictated by an appropriate structural loading code. In some cases, a large number of combinations may need to be considered. For example, for the Emirates Project described below, a total of 18 load combinations was analysed for each tower, these being 1 loading set for the ultimate dead and live loading only, 4 groups of 4 loading sets for various combinations of dead, live and wind loading for the ultimate limit state and 1 set for the long-term serviceability limit state (dead plus live loading).

2.4 DESIGN FOR CYCLIC LOADING

In addition to the normal design criteria, as expressed by equations 1 and 2, Poulos and Davids (2005) have suggested that an additional criterion be imposed for the whole foundation of a tall building to cater for the effects of repetitive loading from wind action, as follows:

$$\eta R_{gs}^* \geq S_c^* \qquad \textbf{(5)}$$

where R_{gs}^* = design geotechnical shaft capacity

S_c^* = half amplitude of cyclic axial wind-induced load

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. For the Emirates project in Dubai, η was selected as 0.5, based on laboratory tests. S_c^* can be obtained from computer analyses which give the cyclic component of load on each pile for various wind loading cases.

2.5 SOIL-STRUCTURE INTERACTION ISSUES

For structural design of the foundation system, soil-structure interaction needs to be considered for the geotechnical ultimate limit state (for example, the bending moments in the raft of a piled raft foundation system). It should be recognised that the worst response may not occur when the pile and raft capacities are factored downwards. As a consequence, additional calculations need to be carried out for geotechnical reduction factors both less than 1 and greater than 1. As an alternative to this duplication of analyses, it would seem reasonable to adopt a reduction factor of unity for the pile and raft resistances, and then factor up the computed moments and shears (for example, by a factor of 1.5) to allow for the geotechnical uncertainties. The structural design of the raft and the piles will also incorporate appropriate reduction factors.

2.6 SERVICEABILITY LIMIT STATE

The design criteria for the serviceability limit state are as follows:

$$\rho_{max} \leq \rho_{all} \quad \textbf{(6)}$$

$$\theta_{max} \leq \theta_{all} \quad \textbf{(7)}$$

where ρ_{max} = maximum computed settlement of foundation
ρ_{all} = allowable foundation settlement,
θ_{max} = maximum computed local angular distortion
θ_{all} = allowable angular distortion.

Values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Table 1 sets out some suggested criteria from work reported by Zhang and Ng (2006). This table also includes values of intolerable settlements and angular distortions. The figures quoted in Table 1 are for deep foundations, but the authors also consider separately allowable settlements and angular distortions for shallow foundations, for different types of structure, different soil types and different building usage. Criteria specifically for very tall buildings do not appear to have been set, but it should be noted that it may be unrealistic to impose very stringent criteria on very tall buildings on clay deposits, as they may not be achievable. In addition, experience with tall buildings in, Frankfurt, Germany suggests that total settlements well in excess of 100mm can be tolerated without any apparent impairment of function.

It should also be noted that the allowable angular distortion, and the overall allowable building tilt, reduce with increasing building height, both from a functional and a visual viewpoint. It can also be noted that in Hong Kong the limiting tilt for most public buildings is 1/300 in order for lifts (elevators) to function properly.

Table 1: Suggested Serviceability Criteria for Structures (Zhang and Ng, 2006).

Quantity	*Value*	*Comments*
Limiting Tolerable Settlement mm	106	Based on 52 cases of deep foundations. Std. Deviation = 55mm. Factor of safety of 1.5 recommended on this value
Observed Intolerable Settlement mm	349	Based on 52 cases of deep foundations. Std. Deviation = 218mm
Limiting Tolerable Angular Distortion rad	1/500	Based on 57 cases of deep foundations. Std. Deviation = 1/500 rad
Limiting Tolerable Angular Distortion rad	1/250 (H<24m) 1/330 (24<H<60m) 1/500 (60<H<100m) 1/1000 (H>100m)	From Chinese Code (MOC, 2002) H = building height
Observed Intolerable Angular Distortion rad	1/125	Based on 57 cases of deep foundations. Std. Deviation = 1/90 rad

3 ANALYSIS APPROACH

3.1 INTRODUCTION

This section will set out briefly an approach that can be employed to address the design issues discussed in the previous section and will then summarize the requirements for the analyses involved.

3.2 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 requirements of the code (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 5) 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 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 (equations **6** and **7**).

3.4 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 unfactored 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.

3.5 ANALYSIS PROGRAM REQUIREMENTS

In order to undertake the above analyses, the programs(s) used should ideally have the following abilities:

1. For overall stability, the program should be able to consider:
    a. Non-homogeneous and layered soil profiles
    b. Non-linearity of pile and, if appropriate, raft behaviour
    c. Geotechnical and structural failure of the piles (and the raft)
    d. Vertical, lateral and moment loading (in both lateral directions) including torsion
    e. Piles having different characteristics within the same group.
2. For serviceability analysis, the above characteristics are also desirable and, in addition, the program should have the ability to consider:
    a. Pile-pile interaction and ,if appropriate, raft-pile and pile-raft interaction
    b. Flexibility of the raft or pile cap
    c. Some means by which the stiffness of the supported structure can be taken into account.

There do not appear to be any commercially available software packages that have all of the above desirable characteristics, other than three-dimensional finite element packages such as PLAXIS 3D or ABAQUS or the finite difference program FLAC3D. The programs REPUTE, PIGLET and DEFPIG have some of the requirements, but fall short of a number of critical aspects, particularly in their inability to include raft -soil contact and raft flexibility.

The author has developed the pile group analysis packages that, between them, provide most of the features listed above. The programs are as follows:

1. PIGS (Pile Group Settlement): PIGS is a proprietary FORTRAN program that analyses the settlement and load distribution within a group of piles subjected to axial and moment loading, and can also consider (in an approximate manner) the effects of externally-imposed vertical ground movements such as those due to swelling or

consolidation of the soil profile. Different pile types can be specified within the pile group, as can varying soil profiles. The basis of the analysis implemented by the program PIGS is the simplified elastic analysis of Randolph and Wroth (1978) for the stiffness of a single pile in an elastic soil mass underlain by a stiffer base layer. PIGS has been used to assess the overall stability of the tower foundation under ultimate loading conditions (not including lateral loading) and foundation settlements and tilt under working loads. It runs quickly: for example, a 400 pile group can be analysed to obtain the load-settlement behaviour top failure in about 100 seconds. It can handle piles of different lengths and diameters in varying soil profiles and can consider sequential loading sequences and cases in which the pile group configuration changes during the loading process. No cap or raft contact is considered and the piles are assumed to be pinned to a rigid cap or, alternatively, each pile in the group can be subjected to specified loads. The effects of external piles (not a part of the group) can also be accounted for approximately.

2. DEFPIG is a FORTRAN program that is commercially available. It computes the distributions of axial and lateral deflections, rotations, axial and lateral loads and moments at the top of a group of piles subjected to a combination of vertical load, lateral loads in a single horizontal direction together with moment in the same horizontal direction. The piles can be attached to a rigid pile cap, or else subjected to specified loadings. Raking piles can be present in the group,and the rake can be in both horizontal directions. The program can take into account the effects of the pile cap being in contact with the ground, but only on axial response. The effects on lateral response are ignored. Using a simplified boundary element approach, DEFPIG computes the single pile flexibility values and the two-pile interaction factors. When calculating the pile flexibilities, it allows for non-linear pile-soil behaviour by limiting the axial and lateral pile-soil pressures to the ultimate values specified by the user. Interaction factors are computed using a purely elastic analysis. A limitation of the DEFPIG analysis is that it considers only a rigid pile cap or raft, and hence cannot readily model the differential settlements that will occur within the foundation system because of the finite stiffness of the raft and superstructure. DEFPIG is used for assessing overall stability under vertical, lateral and moment loading in each horizontal direction. This program takes longer to run than PIGS, but can consider cap-soil contact under vertical loading, although it is limited to a single pile type in its original form. A modified version of DEFPIG (DAMPIG) has been developed so that the capacity and stiffness values of selected piles within a group can be modified to simulate different pile types or defective piles.

GARP (General Analysis of Rafts with Piles) is based on a finite element analysis of the raft, and a boundary element analysis of the piles. The contact stress that acts between the raft and the soil is assumed to be made up of a series of uniform blocks of pressure that act over each element in the raft. Each of the piles is assumed to apply a reaction to the raft at a point (corresponding to a node in the raft). The boundary element analysis is used to calculate the interaction of pairs of piles, or of a pile with the raft. In doing this, it is assumed that the soil is an elastic material. If the soil is layered, a weighted average of the properties of the soil layers is used in determining the equivalent elastic properties of the overall soil mass. If the behaviour of the piles is non-linear, this is modeled by allowing the stiffness of the piles to reduce with load level according to a hyperbolic law. However, the interaction between piles and between the barrettes and raft is assumed to be related to the original pile stiffness. Also, when a pile within the group reaches its design capacity, the GARP program redistributes any excess reactions in the piled raft system. Loading on the raft can include point loads, uniformly distributed loads,and moment loadings. The raft can have different thicknesses assigned to the elements that make up the mesh in order to represent areas of varying raft thickness. The deflections, shear forces and moments in the raft and the vertical loads on the piles due to the loading can be assessed. Small and Poulos (2007) describe the basis of the GARP analysis. GARP is used to compute the settlements and differential settlements under vertical and moment loadings. Because it can take raft (or pile cap) flexibility into account, it is suitable for assessing serviceability requirements. It is also useful for obtaining the axial stiffness of the piles within the group, which can then be passed on to the structural designer for incorporation into the overall structural analysis. In this way, it is possible to obtain more reliable bending moments and shears within the raft than is obtained directly from GARP, since account is taken of the stiffness of the supported structure.

4 SIMPLE EXAMPLE

As a simple example of the application of the suggested approach, the case shown in Figure 1 has been analysed. In this case, a 4 m square raft, 1m thick, is considered (this would normally be one portion of a larger foundation system). For convenience, only a single set of loadings has been considered for the ultimate and serviceability states, as follows:

Ultimate:
Vertical = 12 MN
Horizontal (in the x-direction) = ±1 MN
Moment (acting in the x-direction) = ±7 MNm.

Serviceability:
Vertical = 9MN

The loads and moment are assumed to be applied via a central column 0.8m square. It is assumed that piles 0.4 m in diameter and 20 m long will be driven into a stiff clay layer with the following properties:

Undrained shear strength s_u = 150 kPa
Drained Young's modulus (for vertical loading) = 45 MPa
Undrained Young's modulus for lateral loading = 31.5 MPa
Ultimate skin friction = 60 kPa
Ultimate end bearing pressure = 1 MPa
Ultimate raft bearing pressure = 0.9 MPa.
Ultimate lateral pressure = N_c s_u where $N_c = 2+2z/d \leq 9$, z = depth below surface and d = pile diameter.

The ultimate geotechnical strength in compression is 1.64 MN and 1.52 MN in tension, while the ultimate structural strength of the piles is 3.8MN in compression and 2.5 MN in tension.

Two cases have been considered:

9 piles in a 3x3 square configuration, at a centre-to-centre spacing of 1.6 m in each direction

16 piles in a 4x4 square configuration, at a centre to-centre spacing of 1.2 m in each direction.

4.1 OVERALL STABILITY

For the overall stability assessment, the program DEFPIG has been used, and a geotechnical reduction factor of 0.65 has been applied to both the vertical and lateral resistances. The raft is assumed to be in contact with the underlying soil.

Table 2 summarizes the results of the analyses. While the computed settlements, lateral movements and rotations are not meaningful, they do at least provide an indication of the proximity to overall collapse of the foundation system. It can be seen that both the 9-pile and 16-pile systems satisfy the ultimate limit state criterion in that they do not collapse when the resistances are factored down and the ultimate limit state loadings are applied. It is also clear that the 16-pile group is further from failure than the 9-pile group, as the computed movements and rotation in that case are much smaller.

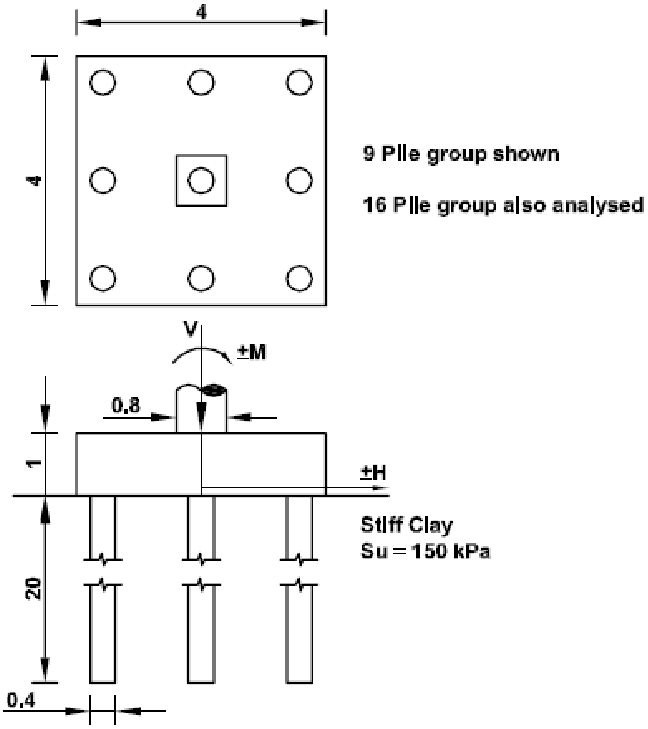


Figure 1: Simple Example Analysed.

Table 2: Summary of Computed Behaviour under Ultimate limit State Conditions

Quantity	9-Pile Group	16-Pile Group
Maximum settlement mm	32	11
Lateral movement mm	10	4
Rotation rad	1/217	1/833
Stable ?	YES	YES

4.2 CYCLIC LOADING ASSESSMENT

DEFPIG has been used to compute the cyclic component of vertical load in the piles due to the lateral and moment loadings. In this case, the pile resistances within the analysis are unfactored. Table 3 summarizes the results and computes the ratio of the cyclic axial load to the design ultimate shaft capacity of each pile. In this case 0.99 MN, the ultimate value of 1.52 MN multiplied by a geotechnical reduction factor of 0.65, assuming the skin friction is the same in both compression and uplift. It can be seen that the 16 pile group satisfies the cyclic loading criterion but the 9 pile group does not.

4.3 SERVICEABILITY ANALYSIS

The program GARP has been used to compute the distributions of settlement, pile load, bending moment and shear within the foundation system under the serviceability loading. The results are summarized in Table 4 and reveal that there is relatively little difference between the settlement and differential settlement for the two groups, and relatively little difference in group performance,

Table 3: Summary of Cyclic Loading Assessment

Quantity	9-Pile Group	16-Pile Group
Maximum cyclic axial load S_c*MN	0.59	0.47
Design Geotechnical Shaft Capacity R_{gs}* MN	0.99	0.99
Ratio $\eta = S_c^*/ R_{gs}^*$	0.60	0.48
Criterion (eq. 5) satisfied? ($\eta \leq 0.5$)	NO	YES

Table 4: Summary of GARP Serviceability Analyses (Vertical Loading)

Quantity	9-Pile Group	16-Pile Group
Maximum Settlement mm	8.3	6.3
Minimum settlement mm	7.1	5.0
Maximum angular rotation rad	1/4760	1/4660
Maximum moment in raft MNm/m	1.44	1.55
Maximum shear in raft MN/m	1.21	1.30
Maximum pile load MN	0.72	0.51

4.4 ANALYSIS FOR STRUCTURAL DESIGN

For this analysis, the ultimate limit state loadings have been applied and the pile and raft resistances have not been factored down. The key objective here is to obtain the maximum pile loads and pile head moments so that the components of the system can be designed to satisfy the structural requirements. Ideally, a program such as GARP would be used, but GARP does not have the capability of considering lateral loadings. Hence, the program DEFPIG has been used again. Table 5 summarizes the key outputs from the DEFPIG analyses, namely the pile head loads and moments. Also shown in this table are the corresponding values that would be obtained from an analysis in which the pile resistances were factored down by a geotechnical reduction factor of 0.65. The following points can be noted:

- Larger moments and axial loads are computed for the 9-pile group, as would be expected.

- When the pile resistances are factored down, unrealistically large bending moments are computed, while the axial loads are equal to the factored-down design resistances, i.e. the axial loads are artificially limited to these values.
- When the pile resistances are factored down, a larger proportion of the load is computed to be carried by the raft. This in turn would imply that the raft actions are larger than if ϕ_g is taken as 1.0.

On the basis of these results, it would seem preferable to obtain the design actions for the piles by factoring up (for example, by $1/\phi_g$) the values computed by applying the ultimate limit state loads and not factoring down the pile resistances.

Table 5: Summary of Analyses for Structural Actions

Quantity	9-Pile Group		16-Pile Group	
	$\phi_g = 1.0$	$\phi_g = 0.65$	$\phi_g = 1.0$	$\phi_g = 0.65$
Maximum Axial Pile Head Load MN	1.52	1.06	1.43	1.06
Maximum Lateral Load at Pile Head MN	0.117	0.118	0.076	0.076
Maximum Pile Head Moment MNm	0.035	0.057	0.023	0.023
Proportion of Load Carried by Raft %	24	35	12	19

4.5 SUMMARY

This simple example illustrates the process followed to assess the suitability of a foundation system to support ultimate load combinations and to perform satisfactorily under serviceability loadings. Table 6 summarizes the computed performance in relation to the various design criteria considered. In this case, it appears that a critical factor is the cyclic wind loading. Were it not for this criterion, a 9-pile group would be satisfactory, but due to the large proportion of the shaft resistance that is mobilized by the cyclic wind loading, a 16-pile group is required.

Table 6: Summary of Assessment Criteria.

Criterion	*Satisfied by 9-pile Group?*	*Satisfied by 16-pile Group?*
Overall stability	YES	YES
Cyclic loading	NO	YES
Serviceability	YES	YES

5 APPLICATION TO HIGH-RISE TOWER IN MIDDLE EAST

5.1 INTRODUCTION

A high-rise tower is currently under construction in Doha Qatar. The tower will be in excess of 400 m tall and will have 74 storeys and three basement levels. It is founded on a pile-supported raft, with piles extending 40 m to 50 m below the base of the raft. A low-rise podium area is to be located adjacent to the tower. As part of the foundation design process, the author undertook a peer review of the geotechnical aspects of the project and the foundation design. The process adopted to assess the foundation design is described below.

5.2 GEOTECHNICAL MODEL

A total of 23 boreholes were drilled at the site, to depths of up to 120m. The in-situ testing consisted of the following:

- SPT tests in upper superficial deposits and at some lower levels where the rock was weak and core recovery was poor.
- Geophysical investigations, including cross-hole tomographic imaging, downhole seismic surveys, a 750 point microgravity survey and a 6-line resistivity survey.
- 53 pressuremeter tests within four of the boreholes beneath the tower, to measure strength and deformation characteristics of the various strata.
- 53 packer tests within seven boreholes, to measure permeability within the various strata.
- 6 standpipes to monitor the groundwater levels.

An extensive program of laboratory testing was undertaken, both conventional and specialized. The conventional tests included particle size distribution, unconfined compressive strength, point load strength and carbonate content tests. The specialized tests included the following:

- Stress path triaxial tests, to measure deformation properties of the strata.
- Resonant column tests, to measure the small-strain modulus values of the rock core samples.
- Cyclic undrained triaxial tests, to assess the effects of cyclic loading on the strength and stiffness of rock core samples.
- Constant normal stiffness direct shear tests, to measure the pile-soil skin friction and the effects of cyclic loading.

Finally, a program of pile load testing was undertaken, consisting of four compression tests on piles of various length (3 with 1.5 m diameter and one with 0.9 m diameter) and two tension tests on piles about 26 m long, one 0.9 m in diameter and the other 0.75 m diameter.

On the basis of the above information, a geotechnical model was progressively developed for the site. The site was quite uniform laterally, so only a single model was necessary. Table 7 summarises the final model adopted for the foundation design verification process. The modulus and skin friction parameters were influenced heavily by the results of the pile load tests. It will be noted that the strata generally become weaker with increasing depth, and no reliable end bearing stratum was found within an acceptable depth. For the raft, an ultimate bearing pressure of 2.1 MPa was assessed.

5.3 FOUNDATION DETAILS AND LAYOUT

The foundation design concept involved the use of a piled raft for the high-rise tower and piles for the podium areas. The basement levels varied below the tower, and were between 15.6 m and 21.6 m below existing ground level. The raft beneath the tower was typically 2.5 m thick and the piles were 1.5 m in diameter, with lengths of 12 m, 22 m or 32 m below the raft, depending on their location. The slab for the podium was to be 1 m thick and was not to be joined to the tower slab. Figure (2) shows the foundation layout for the tower. There is a total of 232 piles, with 40 12 m long piles beneath the wings, 163 piles 22 m long below the main foundation area and 29 piles 32 m long below the lift pits.

Table 8 summarizes the loads which were adopted for the design verification. Various combinations of factored loads were used as per the Australian Loading Code AS 1170-2000, and the combinations adopted for the overall stability analyses are summarised in Table 9.

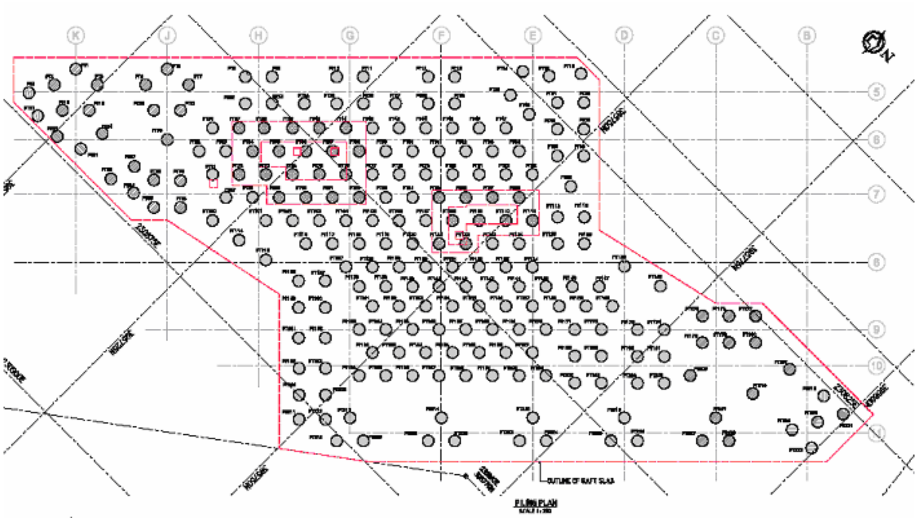


Figure 2: Foundation Layout for Tower.

Table 7: Geotechnical Model Adopted for Verification of Tower Foundation Design

Material	*RL at top of Stratum*[2] *(m QNHD)*	*Thickness m*	*Typical UCS MPa*	*Young's Modulus MPa (Short Term)*	*Young's Modulus MPa (Long-Term)*	*Ultimate Skin Friction*[1] *kPa*	*Ultimate End Bearing MPa*
Limestone	-5	15	15	1650	1500	560	15
Transition Zone	-20	3	4	720	600	675	12
Shale	-23	3	4	720	600	525	4.6
Chalk –1	-26	20	0.6	315	150	400	4.8
Chalk – 2	-46	66	0.2	315	150	250	3.4
Umm Er Radhuma	-112	>25	2	1100	1000	-	-

1 For compression loading. Values for tension were reduced from these values.
2 The raft base level varied between from 15.6 to 21.6 m below existing ground level (deeper levels below lift pits).

Table 8: Summary of Loads on Foundation System.

oad Type	*Vertical Load MN*	*Horizontal Load MN*	*Moment Loading MNm*
Dead (G)	3069	0	0
Live (Q) – assumed to be 10% of G	307	0	0
Wind in x-direction W_{ux}	0	62	11200
Wind in y-direction W_{uy}	0	82.3	15430

Table 9: Loading Cases for Stability Check of Tower

Loading Case	Load Factors					
	Dead Load (G)	Live Load (Q)	Ultimate Wind Load – Long Face		Ultimate Wind Load – Short Face	
			Positive (W_u+)	Negative (W_u-)	Positive (W_u+)	Negative (W_u-)
C1	1.2	0.4	1.0	–	–	–
C2	1.2	0.4	–	1.0	–	–
C3	1.2	0.4	–	–	1.0	–
C4	1.2	0.4	–	–	–	1.0
C5	0.9	–	1.0	–	–	–
C6	0.9	–	–	1.0	–	–
C7	0.9	–	–	–	1.0	–
C8	0.9	–	–	–	–	1.0

5.4 OVERALL STABILITY ASSESSMENT

The program DAMPIG, a development of the DEFPIG program, was used for the assessment of the overall stability of the tower foundation system. This program allowed for the modification of the capacity and stiffness of selected piles within the system so that the three different lengths of pile could be considered. Table 10 summarizes the computed capacity and stiffness of the three pile lengths.

A geotechnical reduction factor of 0.65 was applied to the pile and raft capacities, given that a significant amount of in-situ and pile load testing had been carried out.

TABLE 10: Computed Single Pile Capacities And Stiffness

Pile Type	*Length (m)*	*Diameter (m)*	*Ultimate Compression Capacity (MPa)*	*Ultimate Tension Capacity (MPa)*	*Pile Head Stiffness (MN/m)*
1	12	1.5	42.91	23.05	4098
2	22	1.5	60.73	36.57	4293
3	32	1.5	68.91	44.03	3822

Table 11 summarises the results of the overall stability analyses. For all load combinations considered, the foundation group was found to be stable. This conclusion was consistent with the foundation designer's assessment.

TABLE 11: Analysis Results for Stability of Tower Foundation

Loading Case	*Stable/Unstable*	*Group Lateral Deflection (mm)*	*Group Rotation (Radian)*	*Maximum Pile Settlement (mm)*
1	Stable	2.1	0.000787	161
2	Stable	2.1	0.001031	171
3	Stable	1.7	0.000323	155
4	Stable	1.7	0.000534	165
5	Stable	2.1	0.000512	97
6	Stable	2.1	0.000645	103
7	Stable	1.7	0.000205	94
8	Stable	1.7	0.000326	100

5.5 CYCLIC LOADING ASSESSMENT

In the original peer review assessment, the check for cyclic loading was not carried out because the pile load testing and the laboratory constant normal stiffness direct shear tests indicated that no cyclic degradation of skin friction should occur under the anticipated combinations of mean and cyclic shear stresses imposed on the pile shaft. However, a check was made subsequently for a load combination of 0.9 times dead load plus or minus the wind loads. Typical results for loading in the x-direction are shown in Figure 3. In this case, a small number of piles have a ratio of cyclic axial load to factored-down shaft resistance close to 0.5, but not above, and thus the foundation system would be assessed to be satisfactory from the cyclic loading viewpoint.

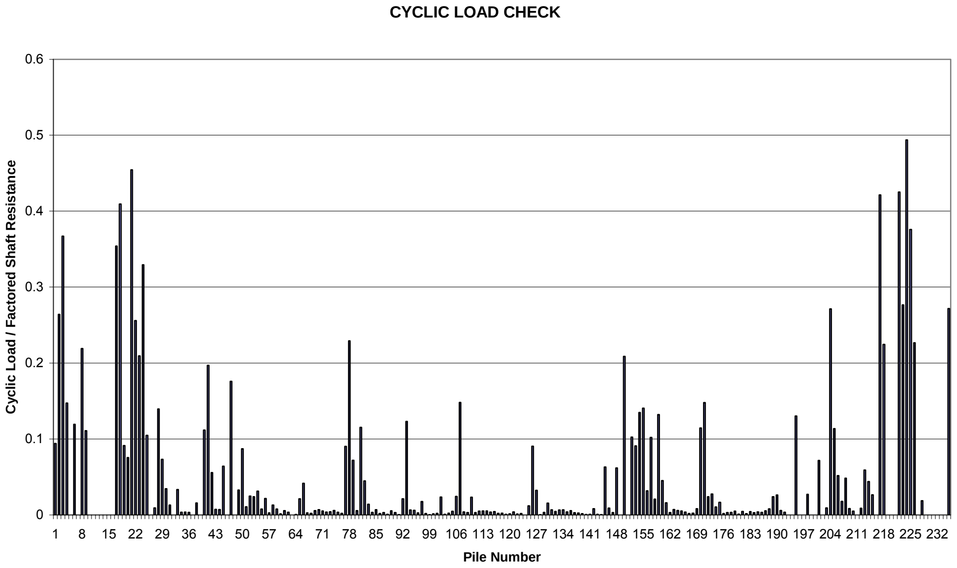


Figure 3: Summary of Cyclic Load Assessment for Wind Loading in the x-Direction.

5.6 ASSESSMENT OF SETTLEMENT UNDER SERVICEABILITY LOADING

Three sets of settlement analyses were carried out:

1. Preliminary calculations to assess the effects of pile length, using a simplified axi-symmetric foundation model with the program PLAXIS.
2. Preliminary calculations with the DEFPIG program, to obtain alternative assessments of average settlement.
3. Detailed analyses with the program GARP, taking into account the actual pile configuration, the detailed distributions of loading and the stiffness of the raft.

In all cases, the geotechnical model in Table 7 was employed.

The two preliminary analyses gave comparable average final settlements under full dead plus live loadings, being within the range 140 to 160mm for the range of pile lengths actually used. The PLAXIS analysis also indicated that there would be little benefit in having piles longer than about 35 to 40m, because much of the settlement was derived from the lower weak limestone layer well below a feasible pile tip level.

The finite element model for the GARP analysis is shown in Figure 4, where the concentrated and distributed vertical loadings are also shown. Figure 5 shows the computed contours of final settlement and indicates that the maximum long-term settlement occurred near the centre of the tower and was about 230 mm. This value was somewhat larger than the value of 184mm computed by the original designer using a non-linear three-dimensional finite element analysis with the program LUSAS. Settlement profiles across three typical sections are shown in Figure 6. The minimum settlement of about 44mm occurs at the edge of the foundation footprint.

The maximum computed angular rotation was of the order of 1/220 and occurred near the south-east junction between the tower and the podium. A similar value was obtained by the original designer. This value was considerably greater than the target value of 1/400, but it was considered that such a value should not lead to structural distress as a substantial proportion of the settlement and differential settlement would occur during construction and before the architectural finishes were applied.

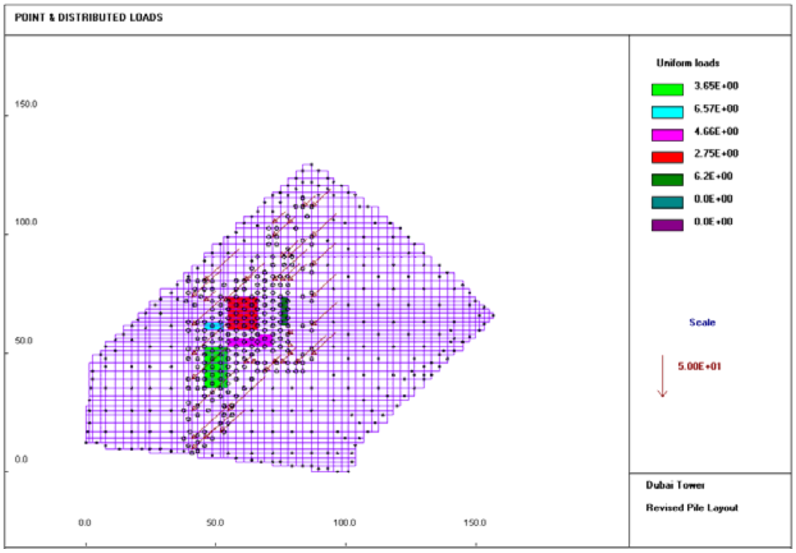


Figure 4: GARP Model Used for Tower Analysis

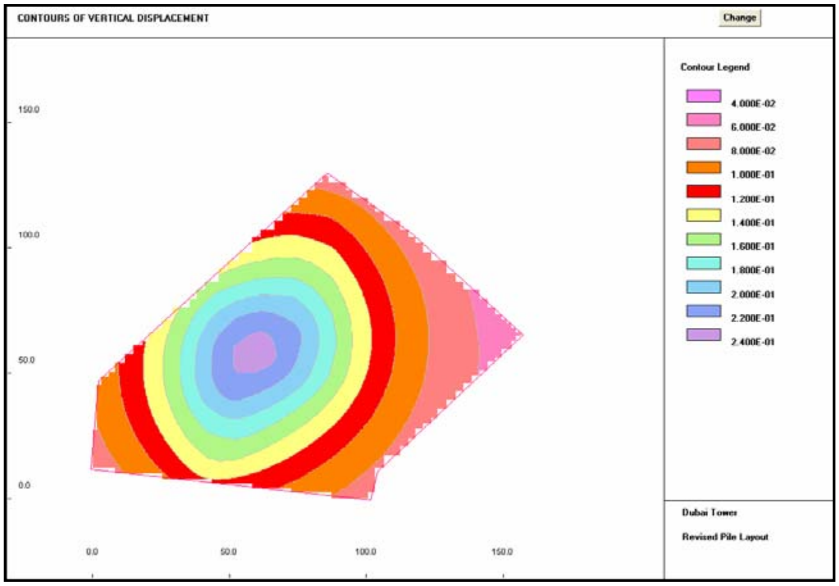


Figure 5: Contours of Computed Long-Term Settlement of Tower.

5.7 ANALYSES FOR PILE HEAD STIFFNESS VALUES

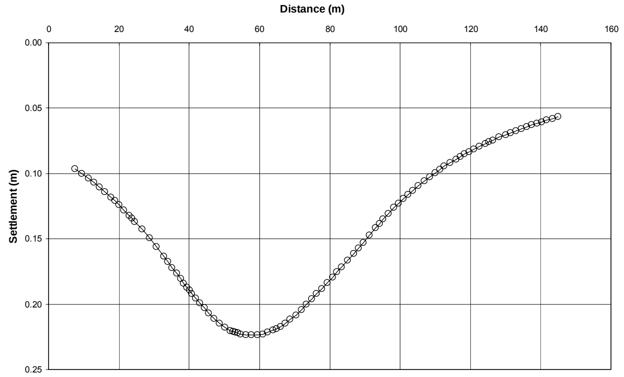
Values of the axial stiffness of each pile within the foundation were required for input into the structural analysis of the complete structure-foundation system, The computer program PIGS was used to obtain these stiffness values for the piles when subjected to both long-term and short-term loading. PIGS is a FORTRAN program that calculates the settlement of single piles and pile groups subjected to vertical loading. It allows for non-linear pile response via an assumed hyperbolic load-settlement behaviour for each pile, and the interaction factor method is employed to consider interaction among piles within a group. The group can be subjected to a series of vertical and moment loadings if the group has a rigid cap or, alternatively, each pile in the group can be subjected to a specified vertical load. The program computes the distribution of settlement and load within the group.

The following simplifying assumptions were made:

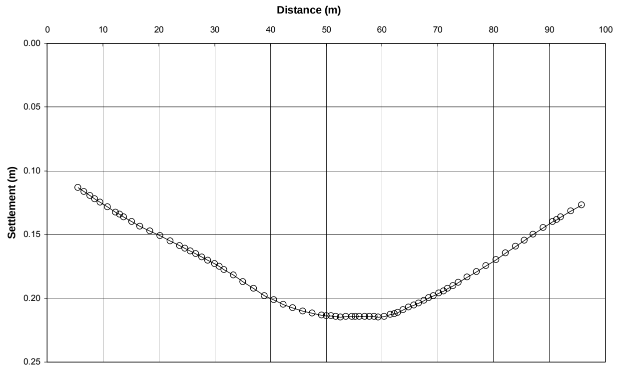
- Each pile was subjected to an axial load equal to the average long-term load of 12.07 MN.
- The effects of the raft being in contact with the soil were not taken into account.

The computed pile head stiffness values ranged between 71 MN/m and 635 MN/m, with the smaller values being for piles towards the centre of the tower and the larger values being towards the edges of the foundation footprint. These values are far less than those for a single pile shown in Table 10. This difference, together with the considerable variation of the computed stiffness values, demonstrates the vital importance of considering pile-pile interaction within the foundation system for tall buildings.

Section 1 (Y-Coordinate =52.6m)



Section 2 (X-Coordinate =48.6m)



Section 3 (X-Coordinate =77.7m)

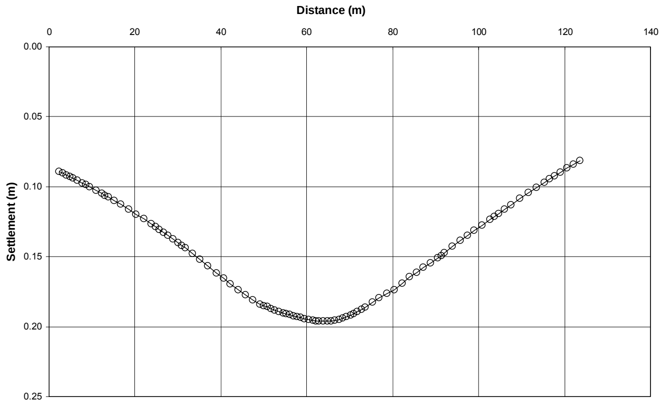


Figure 6: Computed Settlement Profiles across Typical Sections (see Figure 4 for Section Locations).

5.8 ANALYSES FOR STRUCTURAL DESIGN

For this structure, only GARP analyses for the dead plus live loading case were undertaken to check the more complete three-dimensional analyses carried out by the designer. The maximum pile axial force from GARP was found to be 38.5 MN, which was larger than the value of about 33 MN obtained by the designer. The maximum moments in the raft were found to be similar in magnitude, 15.8 MNm/m from GARP and 16.3 MNm/m from the three dimensional analysis. It was therefore concluded that the latter analysis was a suitable basis for computing the pile loads and obtaining the raft moments. For the final design, the structural analysis of the complete structure-foundation system was carried out using the computed values of stiffness for each of the piles, and the design pile loads, and raft moments and shears, were obtained from this analysis.

6 CONCLUSIONS

This paper has set out an approach for using computer programs for pile group response to design a pile foundation system according to the Australian Piling Code AS2159-1995. The approach involves three sets of analyses:

1. An overall stability analysis in which the resistances of the foundation components are reduced by the appropriate geotechnical reduction factor and the ultimate limit state (ULS) load combinations are applied. The requirements of the code will be satisfied if the foundation system does not collapse under any of the sets of ULS loadings.
2. A serviceability analysis, in which the best-estimate (unfactored) values 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. For structural design of the raft and the piles, the results of the above 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. The most rational approach appears to be to carry out a separate ULS analysis in which the ULS load combinations are applied but in which the unfactored 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 to obtain the values for structural design.
4. In addition, a check can be carried out to assess the ratio of cyclic load amplitude to factored-down pile shaft resistance. It is suggested that if this ratio for a pile is less than about 0.5, there should be a low risk of cyclic degradation of shaft resistance occurring.
5. The suggested procedure has been used for a simple example to demonstrate the means by which it can be applied.
6. For a high-rise tower in the Middle East, the process has been used to assess overall stability, foundation settlements and pile head stiffness values as part of an independent peer review process. Reasonable agreement was found between the analyses carried out in this process and the more complex three dimensional finite element analyses employed by the foundation designer.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Poulos, H.G. (1999). “The design of piles with particular reference to the Australian Piling Code”. Australian Geomechanics, 34(4): 25-39.

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: 550-555.

Zhang, L. and Ng, A.M.Y.(2006). “Limiting tolerable settlement and angular distortion for building foundations”. Geotech. Special Publication No. 170, Probabilistic Applications in Geotechnical Engineering, ASCE (on CD Rom).