The Design of High-Rise Building Foundations

Harry G. Poulos

*Coffey Geotechnics, Sydney, Australia E-mail: harry_poulos@coffey.com*

**Keywords:** foundation; high-rise; limit state design; Korea; Middle East; piled raft; piles; settlement; stability; ultimate capacity.

**ABSTRACT:** This paper sets out the principles of design for a pile or piled raft foundation system for tall buildings via a limit state design approach. This 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.

2. A separate ULS analysis is carried out 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.

3. 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 approach is illustrated via its application to a simple hypothetical case, and then to high-rise buildings in the Middle East and Korea.

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 a limit state design approach for tall building foundation systems, with attention being focused on piled and piled raft foundation systems, which are the predominant types currently used. Some of the characteristics of piled rafts are outlined, and then the principles of the design approach are set out. Examples of the application of this approach are described for a hypothetical problem, and then for two high-rise buildings, one in Doha Qatar, and another on reclaimed land in Incheon South Korea.

2. **PILED RAFT FOUNDATION SYSTEMS**

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. The beneficial characteristics of piled rafts have been described by Randolph (1994), Poulos (2001) and Phung (2010), among others.

A geotechnical assessment for design of such a foundation system therefore needs to consider not only the capacity of the raft and pile elements, but their combined capacity and their interaction under serviceability loading. This section sets out some of the basic design issues to be considered in piled raft design, and some of the characteristics of behaviour that are found from numerical analyses.

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 also contribute to the foundation stiffness, especially after the pile capacity has been fully mobilized.

Conversely, there are some situations which are less favourable, including:

- Soil profiles containing soft clays near the surface
- Soil profiles containing loose sands near the surface
- Soil profiles which contain soft compressible layers at relatively shallow depths
- Soil profiles which are likely to undergo consolidation settlements
- Soil profiles which are likely to undergo swelling movements due to external causes.

In the first two cases, the raft may not be able to provide significant load capacity and stiffness, while in the third case, long-term settlement of the compressible underlying layers may reduce the contribution of the raft to the long-term stiffness of the foundation. The
latter two cases should be treated with caution, as the ground movements can affect adversely the foundation performance. It has also been found that the performance of a piled raft foundation can be optimized by selecting suitable locations for the piles below the raft. In general, the piles should be concentrated in the most heavily loaded areas, while the number of piles can be reduced, or even eliminated, in less heavily loaded areas (Horikoshi and Randolph, 1998).

3. DESIGN REQUIREMENTS

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 capacity

\( S^* \) = design action effect

\( \phi_s \) = structural reduction factor

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).

The geotechnical strength criterion will be satisfied if the foundation system with the reduced strengths does not collapse under the ultimate limit state design action effects (factored-up load combinations).

\( R_s^* \) 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 \( \phi_s \) requires judgment and takes into account a number of factors that may influence the foundation performance.

3.2 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 in Dubai (Poulos and Davids, 2005), a total of 18 load combinations was analyzed 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).

3.3 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:

\[ \eta R_{gs}^* \geq S_c^* \quad (3) \]

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

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

\( \eta \) = 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, $\eta$ 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.

### 3.4 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 recognized 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.

### 3.5 Serviceability Limit State

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

\[
\rho_{\text{max}} \leq \rho_{\text{all}} \quad (4)
\]
\[
\theta_{\text{max}} \leq \theta_{\text{all}} \quad (5)
\]

where $\rho_{\text{max}}$ = maximum computed settlement of foundation

$\rho_{\text{all}}$ = allowable foundation settlement,

$\theta_{\text{max}}$ = maximum computed local angular distortion

$\theta_{\text{all}}$ = allowable angular distortion.

Values of $\rho_{\text{all}}$ and $\theta_{\text{all}}$ depend on the nature of the structure and the supporting soil.

Some suggested criteria reported by Zhang and Ng (2006) are shown in Table 1. This table also includes values of intolerable settlements and angular distortions. 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. 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.

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

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limiting Tolerable Settlement, mm</td>
<td>106</td>
</tr>
<tr>
<td>Observed Intolerable Settlement, mm</td>
<td>349</td>
</tr>
<tr>
<td>Limiting Tolerable Angular Distortion, rad</td>
<td></td>
</tr>
<tr>
<td>$1/250$ (H&gt;$24m$)</td>
<td></td>
</tr>
<tr>
<td>$1/330$ (24&lt;$H&lt;$60m)</td>
<td></td>
</tr>
<tr>
<td>$1/500$ (60&lt;$H&lt;$100m)</td>
<td></td>
</tr>
<tr>
<td>$1/1000$ (H&gt;100m)</td>
<td></td>
</tr>
<tr>
<td>Observed Intolerable Angular Distortion, rad</td>
<td>1/125</td>
</tr>
</tbody>
</table>

### 3.6 Design Tools and Analysis Requirements

In order to undertake the above design analyses, it is almost essential to use a computer program. Such a program 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 include PIGS which analyzes the settlement and load distribution within a group of piles subjected to axial and moment loading, CLAP, that 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 and moments in each horizontal direction, and a torsional load, and GARP, which analyses the behaviour of a piled raft subjected to vertical and moment loading.
Details of this program are given by Small and Poulos (2007).

4 A 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 4m 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 1.6m in each direction;
- 16 piles in a 4x4 square configuration, at a centre to-centre spacing of 1.2m 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 Analyzed](image)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>9-Pile Group</th>
<th>16-Pile Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum settlement, mm</td>
<td>32</td>
<td>11</td>
</tr>
<tr>
<td>Lateral movement, mm</td>
<td>10</td>
<td>4</td>
</tr>
<tr>
<td>Rotation, rad</td>
<td>1/217</td>
<td>1/833</td>
</tr>
<tr>
<td>Stable Overall?</td>
<td>YES</td>
<td>YES</td>
</tr>
</tbody>
</table>

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, 0.99MN, being 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.
Table 3. Summary of Cyclic Loading Assessment

<table>
<thead>
<tr>
<th>Quantity</th>
<th>9-Pile Group</th>
<th>16-Pile Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum cyclic axial load $S_c$ MN</td>
<td>0.59</td>
<td>0.47</td>
</tr>
<tr>
<td>Design Geotechnical Shaft Capacity</td>
<td>0.99</td>
<td>0.99</td>
</tr>
<tr>
<td>Ratio $\eta = S_c/Rgs*$</td>
<td>0.60</td>
<td>0.48</td>
</tr>
<tr>
<td>Criterion satisfied? ($\eta \leq 0.5$)</td>
<td>NO</td>
<td>YES</td>
</tr>
</tbody>
</table>

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 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 4 Serviceability Analysis Results

<table>
<thead>
<tr>
<th>Quantity</th>
<th>9-Pile Group</th>
<th>16-Pile Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. Settlement mm</td>
<td>8.3</td>
<td>6.3</td>
</tr>
<tr>
<td>Min. Settlement mm</td>
<td>7.1</td>
<td>5.0</td>
</tr>
<tr>
<td>Max. Angular Rotation rad</td>
<td>1/4760</td>
<td>1/4600</td>
</tr>
<tr>
<td>Max. Raft Moment MN/m</td>
<td>1.44</td>
<td>1.53</td>
</tr>
<tr>
<td>Max. Raft Shear MN/m</td>
<td>1.21</td>
<td>1.30</td>
</tr>
<tr>
<td>Max. Pile Load MN</td>
<td>0.72</td>
<td>0.51</td>
</tr>
</tbody>
</table>

4.4 Analysis for Structural Design

For this analysis, the ultimate limit state loadings have not 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:

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

b. 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.

c. 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 $\phi_g$ 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

<table>
<thead>
<tr>
<th>Quantity</th>
<th>9-Pile Group</th>
<th>16-Pile Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_g = 1.0$ $\phi_g = 0.65$</td>
<td>1.52</td>
<td>1.06</td>
</tr>
<tr>
<td>$\phi_g = 1.0$</td>
<td>1.43</td>
<td>1.06</td>
</tr>
<tr>
<td>$\phi_g = 0.65$</td>
<td>0.117</td>
<td>0.118</td>
</tr>
<tr>
<td>$\phi_g = 0.076$</td>
<td>0.076</td>
<td>0.076</td>
</tr>
<tr>
<td>Maximum Pile Head Load MN</td>
<td>0.035</td>
<td>0.057</td>
</tr>
<tr>
<td>Carried by Raft</td>
<td>0.023</td>
<td>0.023</td>
</tr>
</tbody>
</table>

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 3 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.

5. APPLICATION TO HIGH-RISE TOWER IN QATAR

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 to 50m 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:

a. SPT tests in upper superficial deposits and at some lower levels where the rock was weak and core recovery was poor.

b. Geophysical investigations, including cross-hole tomographic imaging, downhole seismic surveys, a 750 point microgravity survey and a 6-line resistivity survey.

c. 53 pressuremeter tests within four of the boreholes beneath the tower, to measure strength and deformation characteristics of the various strata.

d. 53 packer tests within seven boreholes, to measure permeability within the various strata.

e. 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 1.5m diameter and one with 0.9m diameter) and two tension tests on piles about 26m long, one 0.9m in diameter and the other 0.75m diameter.

On the basis of the above information, a geotechnical model was progressively developed for the site. The site was quite uniform laterally, and so only a single model was necessary. Table 6 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 and 21.6m below existing ground level, which was typically at a reduced level (RL) of +2m QNHD (Qatar National Height Datum). The raft beneath the tower was typically 2.5m thick and the piles were 1.5m in diameter, with lengths of 12m, 22m or 32m below the raft, depending on their location. The slab for the podium was to be 1m 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 12m long piles beneath the wings, 163 piles 22m long below the main foundation area, and 29 piles 32 m long below the lift pits.

Table 7 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.

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

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).
3 UCS = uniaxial compressive strength
Table 7. Summary of Loads on Foundation System

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Vertical Load MN</th>
<th>Horizontal Load MN</th>
<th>Moment Loading MNm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead (G)</td>
<td>3069</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Live (Q) – assumed to be 10% of G</td>
<td>307</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Wind in x-direction W_{ux}</td>
<td>0</td>
<td>62</td>
<td>11200</td>
</tr>
<tr>
<td>Wind in y-direction W_{uy}</td>
<td>0</td>
<td>82.3</td>
<td>15430</td>
</tr>
</tbody>
</table>

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 (12, 22 and 32m) could be considered. The computed compressive capacities of the three pile lengths were 42.9MN, 60.9MN and 68.9MN respectively, while the corresponding axial pile head stiffness values were 4098, 4293 and 3822MN/m. The lower stiffness for the longer pile reflected the influence of the weaker Chalk layer at depth.

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.

For all load combinations considered, the foundation group was found to be stable. This conclusion was consistent with the foundation designer’s assessment.

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.

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 8 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
indicated 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 184 mm computed by the original designer using a non-linear three-dimensional finite element analysis with the program LUSAS. The minimum settlement of about 44 mm occurs at the edge of the foundation footprint. The analysis indicated that the raft carried about 23% of the applied load under serviceability conditions, due to the stiff limestone on which the raft was founded.

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 reported in Section 5.4. 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.

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 three dimensional analysis together with the computed values of stiffness for each of the piles. The design pile loads, and raft moments and shears, were obtained from this analysis.
6. APPLICATION TO INCHEON TOWER, KOREA

6.1 Introduction
Currently, a 151 storey super highrise building project is under design, located in reclaimed land constructed on soft marine clay in Songdo, Korea. The foundation system considered comprises 172 No. 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 6 and is described in detail by Badelow et al (2009); thus, only a brief summary is presented here.

6.2 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 natural surface is at an elevation of about EL0.

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 6. 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. These are reproduced in Figure 7. It can be seen that there is 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 by Badelow et al (2009).

6.3 Foundation Layout
The foundation comprises a 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 the geotechnical engineer, considering the performance and capacity of piles. The pile layout was selected from the various options considered, and is presented in Figure 8.

Figure 7 Inferred Contours of Top of Soft Rock – Incheon Tower
6.4 Loadings

Typical loads acting on the tower were as follows:

- Vertical dead plus live load: \( P_z(DL+LL) = 6622 \text{MN} \)
- Horizontal wind loads:
  \[ P_x(WL) = 146 \text{MN} \quad P_y(WL) = 112 \text{MN} \]
- Horizontal earthquake loads:
  \[ P_x(E) = 105 \text{MN} \quad P_y(E) = 105 \text{MN} \]
- Wind load moments:
  \[ M_x(WL) = 12578 \text{MN} \cdot \text{m} \quad M_y(WL) = 21173 \text{MN} \cdot \text{m} \]
- Wind torsional load:
  \[ M_z(WL) = 1957 \text{MN} \cdot \text{m} \]

The vertical loads (DL+LL) and overturning moments (Mx, My) were represented as vertical load components at column and core locations. The load combinations, as provided by the structural designer, were adopted throughout the geotechnical analysis, and 24 wind load combinations were considered.

6.5 Assessment of Pile Capacities

The geotechnical capacities of piles were determined by the shaft friction and end bearing capacities of pile, and the required pile length was generally assessed based on these geotechnical capacities to provide the required load capacity. For a large pile group founding in weak rock, the overall settlement behavior of the pile group could control the required pile lengths rather than the overall geotechnical capacity. In this case, the soft rock layer was considered to be a more appropriate founding stratum than the overlying weathered rock, in particular the soft rock below Elevation EL-50m. This is because this stratum provides a more uniform stiffness and therefore is likely to result in a more consistent settlement behavior of the foundation. The basic guide lines to establish the pile founding depth were:

- Minimum socket length in soft rock = 2 diameters;
- Minimum toe level = EL-50m.

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 8 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.

<table>
<thead>
<tr>
<th>Material</th>
<th>Ultimate Friction ( f_s )(kPa)</th>
<th>Ultimate End Bearing ( f_b )(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Rock</td>
<td>500</td>
<td>5</td>
</tr>
<tr>
<td>Soft Rock</td>
<td>750</td>
<td>12</td>
</tr>
</tbody>
</table>

Pile load tests prior to commencement of the main piling works have been planned. Based on the interpreted findings of these tests, the pile capacities will be verified and the pile design confirmed.

6.6 Overall Stability

The ultimate limit state (ULS) combinations of load were input into a series of CLAP analyses with the pile axial and lateral capacities reduced by geotechnical reduction factors (0.65 for axial load, 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.

6.7 Cyclic Loading Assessment

Table 9 summarizes the results of the cyclic loading assessment while Figure 9 shows the assessed factor \( \eta \) for each pile within the foundation system. The assessment indicates that the criterion in equation 3 is satisfied, and that degradation of shaft capacity due to cyclic loading is unlikely to occur.

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Cyclic Axial Load ( S_c^* ) (MN)</td>
<td>74.0</td>
</tr>
<tr>
<td>Maximum Ratio ( \eta = S_c^<em>/R_{c_e}^</em> )</td>
<td>0.43</td>
</tr>
<tr>
<td>Cyclic Loading Criterion Satisfied?</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Figure 9. Results of cyclic loading analysis – Load Case 0.75(DL+LL+WL)
6.8 Assessment of Vertical Pile Behavior

The vertical pile head stiffness values for each of the 172 foundation piles under serviceability loading (DL + LL) were assessed using the computer program CLAP. CLAP was used to assess the geotechnical capacities, interaction factors and stiffness values for each pile type under serviceability loading for input into the group assessment. The various pile types included the varying pile founding levels, which were dependent on the level of the top of the Soft Rock. The computed axial pile head stiffness values ranged between a high of 1300MN/m and a low of 600MN/m, with an average value of 850MN/m. The pile stiffness values for each individual pile, together with the computed interaction factors, were then used in the program GARP to compute the magnitude and distribution of settlement.

6.9 Settlement

The maximum settlements predicted ranged from about 50mm (using the estimated upper bound modulus) to about 75mm (with the estimated lower bound modulus), with differential settlements between the centre and edge of the slab ranging from about 30mm to 50mm. For the tower alone, an elastic analysis using an “equivalent block” approach via a 2-D axi-symmetric finite element analysis gave maximum and minimum settlement values of 43mm and 18mm, respectively, which are in good agreement with those predicted by one of the project reviewers using an equivalent block hand calculation approach (43mm and 19mm respectively for the tower alone). The predicted settlements using the various analysis methods were reasonably consistent, giving a level of confidence to the settlement values predicted. Figure 10 shows a typical set of settlement contours derived from the GARP analyses, while Figure 11 shows the computed distribution of pile loads.

The analysis indicated that, under serviceability conditions, the raft would carry only about 2% of the applied load, due to the fact that it rested on the relatively soft marine deposits.

6.10 Assessment of Lateral Pile Behavior

One of the critical design issues for the Tower foundation was the performance of the pile group under lateral loading. The numerical modeling packages used in the analyses comprised the following:

- 3D finite element computer program PLAXIS 3D Foundation;
- The program CLAP.

PLAXIS 3D provided an assessment of the overall lateral stiffness of the foundation. The program CLAP was used to assess the lateral stiffness provided by the pile group assuming that the raft is not in contact with the underlying soil, and then a separate calculation was carried out to assess the lateral stiffness of the raft and basement. Table 10 presents typical values of computed lateral stiffness for the piled mat foundation obtained from the analyses.

<table>
<thead>
<tr>
<th>Horiz. Load (MN)</th>
<th>Dirn.</th>
<th>Pile Group Displ. (mm)</th>
<th>Lateral Pile Stiffness (MN/m)</th>
<th>Lateral Raft Stiffness (MN/m)</th>
<th>Total Lateral Stiffness (MN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>149</td>
<td>X</td>
<td>17</td>
<td>8760</td>
<td>198</td>
<td>8958</td>
</tr>
<tr>
<td>115</td>
<td>Y</td>
<td>14</td>
<td>8210</td>
<td>225</td>
<td>8435</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

This paper has set out an approach for the design of pile foundation systems for high-rise buildings, using a limit...
state design approach. This approach involves three sets of analyses:

i. 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.

ii. 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.

iii. 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.

iv. 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.

v. For high-rise towers in Qatar and Korea, the process has been used to assess overall stability, foundation settlements and pile head stiffness values.

8. ACKNOWLEDGEMENTS

The author gratefully acknowledges the contributions of Frances Badelow, Tristan McWilliam, Helen Chow and Patrick Wong in relation to the analyses for the tower described in the paper. S.H. Kim, Ahmad Abdelrazaq and their teams in Korea had a major involvement in the Incheon Tower foundation design. Professor John Small has been pivotal in developing the GARP program and implementing it in a user-friendly form.

9. REFERENCES


