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The design of foundations for high-rise buildings

High-rise buildings are usually founded on some form of piled foundation subject to a combination of vertical, lateral and overturning forces. However, conventional methods for assessing stability may not be adequate when designing such foundations because they tend to focus on resistance under vertical loading. This paper sets out an ultimate-limit-state approach for computer-based design of pile foundation systems for high-rise buildings and provides an example application on a 151-storey tower in South Korea.

Conventional methods of assessing foundation stability tend to focus primarily on foundation resistance under vertical loading but, for tall buildings, the resistance to combined vertical, lateral and moment loadings must be considered. 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 – the predominant types currently used.

The key characteristics of piled rafts are outlined and then the principles of the design approach are set out. An example of the application of this approach is described for a 151-storey tower on reclaimed land in Incheon, South Korea (Figure 1).

Piled-raft foundation systems

In a piled-raft foundation system, the piles provide most of the stiffness for controlling settlements at serviceabil-

Figure 1. Artist's impression of the 601 m tall Incheon 151 Tower in Songdo, Korea, which is due for completion in 2015

ity loads and the raft element provides additional capacity at ultimate loading. A geotechnical assessment for designing such a foundation system therefore needs to consider not only the capacity of the pile and 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 idealised 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.

It has been found that the performance of a piled-raft foundation can be optimised 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).

Design requirements

Design issues

The following issues usually need to be addressed in the design of foundations for high-rise buildings (Poulos, 2009).

- Ultimate capacity of the foundation under vertical, lateral and moment loading combinations.
- Influence of the cyclic nature of wind, earthquakes and wave loadings (if appropriate) on foundation capacity and movements.
- Overall settlements.
- Differential settlements, both within the high-rise footprint and between high-rise and low-rise areas.
- 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 to have close co-operation and interaction between geotechnical and structural designers.
- Possible effects of externally-imposed ground movements on the foundation system, for example movements arising from excavations for pile caps or adjacent facilities.
- 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.
- 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.

Design criteria

In limit state format (such as in the Australian Piling Code AS2159-1995 (Standards Australia, 1995)), the ultimate limit state (ULS) criteria may be expressed as follows

$$R_s^* \geq S^* \tag{1}$$

$$R_g^* \geq S^* \tag{2}$$

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

The above criteria 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 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 the designer to exercise judgement and take into account a number of factors that may influence the foundation performance.

Load combinations

The required load combinations for which the structure and foundation system have to be designed will usually be dictated by a structural loading code and a large number of combinations may need to be considered.

For example, for the Emirates Towers project in Dubai (Poulos and Davids, 2005), 18 load combinations were analysed for each tower, these being one loading set for the ultimate dead and live loading only, four groups of four loading sets for various combinations of ultimate dead, live and wind loads and one set for the long-term serviceability limit state (dead plus live loading).

Design for cyclic loading

In addition to the normal design criteria, Poulos and Davids (2005) have suggested an additional criterion for the whole foundation of a tall building, to cater for the effects of repetitive loading from wind action, as follows

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

where R_{gs}^* is the design geotechnical shaft capacity; S_c^* is the half amplitude of cyclic axial wind-induced load and η is a factor assessed from geotechnical laboratory testing.

This criterion attempts to avoid the full mobilisation of shaft friction along the piles, thus reducing the risk of cyclic loading leading to degradation of shaft capacity. For the Emirates Towers project, 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.

Serviceability limit state

The design criteria for the serviceability limit state (SLS) are as follows

$$\rho_{max} \leq \rho_{all} \quad (4)$$

$$\theta_{max} \leq \theta_{all} \quad (5)$$

where ρ_{max} is the maximum computed settlement of foundation; ρ_{all} is the allowable foundation settlement; θ_{max} is the maximum computed local angular distortion and θ_{all} is the allowable angular distortion.

The values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Some suggested criteria are reported by Zhang and Ng (2006). Criteria specifically for very tall buildings do not appear to have been set but it may be unrealistic to impose very stringent criteria on tall buildings on clay deposits, as these may not be achievable.

Experience with tall buildings in Frankfurt, Germany suggests that total settlements well in excess of 100 mm can be tolerated without any apparent impairment of function (Katzenbach *et al*., 2000).

Analysis methods

Overall stability

For consideration of the overall stability of the foundation system, the ULS load combinations are applied and the analysis uses geotechnical and structural resistances of the foundation components, which are reduced by a geotechnical reduction factor and a structural reduction factor respectively.

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 to increase the strength of the overall system or of those components of the system that do not satisfy the criteria.

Serviceability

For the serviceability analysis, the SLS loads are applied and the best-estimate (unfactored) values of foundation resistances and stiffnesses are employed.

The design will be satisfactory if the computed deflections and rotations are within the specified allowable limits (Equations 4 and 5).

Structural design requirements

For structural design of the raft and piles, the results of ULS analysis are not considered relevant because the loads that can be sustained by the piles are artificially reduced by the geotechnical reduction factor and the worst response may not occur when the pile and raft capacities are factored downwards.

Consequently, the most rational approach appears to be 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, raft moments and shears) are then multiplied by a structural action factor (for example 1·5) to obtain values for structural design.

Analysis program requirements

To undertake the above analyses, a computer program should ideally have the following abilities. For overall stability, it should have the ability to consider

- non-homogeneous and layered soil profiles
- non-linearity of pile and, if appropriate, raft behaviour
- geotechnical and structural failure of the piles (and the raft)
- vertical, lateral and moment loading (in both lateral directions), including torsion
- piles having different characteristics within the same group.

For serviceability analysis, the above characteristics are also desirable, together with the ability to consider

- pile–pile interaction, and if appropriate, raft–pile and pile–raft interaction
- flexibility of the raft or pile cap
- 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 (Plaxis, 2009), or the finite-difference program Flac3D (Itasca, 2009). The commercially available programs Repute (Geocentrix, 2009), Piglet (Randolph, 2004) and Defpig (Poulos, 1990) 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 pile-group analysis packages that, between them, provide most of the features listed above. The programs include Pigs, which analyses the settlement and load distribution within a group of piles subjected to axial and moment loading; Clap, which computes the distributions of axial and lateral deflections, rotations, loads and moments at the top of a group of piles subjected to a combination of vertical load, lateral loads and moments; and Garp, which analyses the behaviour of a piled raft subjected to vertical and moment loading (Small and Poulos, 2007).

Application to Incheon Tower, Korea

A 151-storey super-high-rise building project is currently under construction on reclaimed land on soft marine clay in Songdo, Incheon, Korea. Due for completion in 2015, the 601 m tall '151 Incheon Tower' is illustrated in Figure 1 and the geotechnical aspects are described in detail by Badelow *et al.* (2009).

Ground conditions and geotechnical model

The site lies entirely within an area of reclamation, which typically comprises approximately 8 m of loose sand and sandy silt, constructed over approximately 20 m of soft to firm marine silty clay, referred to as upper marine deposits. These are underlain by approximately 2 m of medium dense to dense silty sand, referred to as lower marine deposits, which overlie residual soil and a profile of weathered rock.

It is intended that the fill will be improved to reduce the risk of liquefaction during earthquakes. However, as the tower will be founded on the upper marine deposits liquefaction will not be a direct issue.

The lithological rock units within about 50 m of 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 roof-pendant sedimentary and 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.

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 2. 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 40 m 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 upper marine deposits under lateral and vertical loading. Typical parameters adopted for the foundation design are presented in Table 1.

Foundation layout

The foundation system considered comprises 172 no. 2·5 m diameter bored piles, socketed into the soft rock layer and connected to a 5·5 m thick raft supporting columns and core walls. The number and layout of piles and the pile size were obtained from trial analyses through collaboration between the geotechnical and structural designers.

The pile depth was evaluated 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 3.

Loadings

Typical loads acting on the tower were as follows

- Vertical dead plus live load: $Pz_{(DL+LL)}$ = 6622 MN
- Horizontal wind loads: $Px_{(WL)}$ = 146 MN, $Py_{(WL)}$ = 112 MN
- Horizontal earthquake loads: $Px_{(E)}$ = 105 MN, $Py_{(E)}$ = 105 MN
- Wind load moments: $Mx_{(WL)}$ = 12 578 MN.m, $My_{(WL)}$ = 21 173 MN.m
- Wind load torsional load: $Mz_{(WL)}$ = 1957 MN.m

The vertical load $Pz_{(DL+LL)}$ and overturning moments $Mx_{(WL)}$ and $My_{(WL)}$ 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.

Assessment of pile capacities

The required pile length was assessed from the estimated shaft friction and end bearing capacities. For a large pile group founding in weak rock, the overall settlement behaviour of the

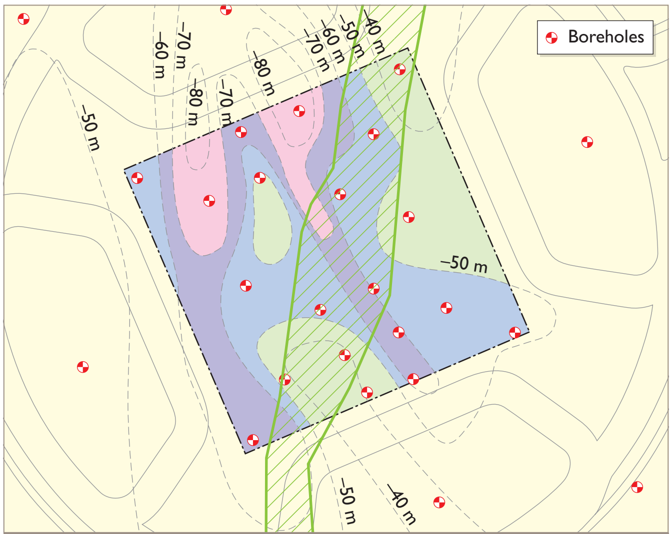

Figure 2. Borehole-derived contours of the soft rock surface under the Incheon Tower foundation – the depth varies from 40 to 80 m

Table 1. Summary of geotechnical parameters

Strata	E_v: MPa	E_h: MPa	f_s: kPa	f_b: MPa
Upper marine deposits	7–15	5–11	29–48	—
Lower marine deposits	30	21	50	—
Weathered soil	60	42	75	—
Weathered rock	200	140	500	—
Soft rock (above elevation −50m)	300	210	750	12
Soft rock (below elevation −50m)	1700	1190	750	12

E_v = vertical modulus, f_s = ultimate shaft friction, E_h = horizontal modulus, f_b = ultimate end bearing

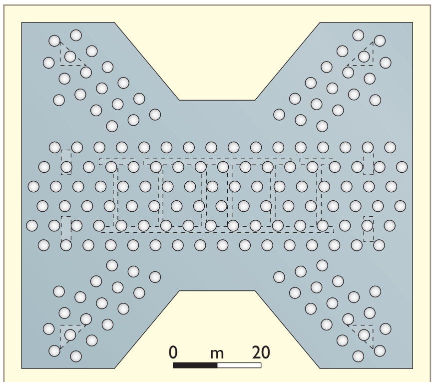

Figure 3. Layout of the 172 no. 2·5 m diameter bored piles and 5·5 m thick raft

pile group controlled 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 –50 m provides a more uniform stiffness and is likely to result in more consistent settlement behaviour of the foundation.

The basic guidelines to establish the pile founding depth were

- minimum socket length in soft rock = two diameters
- minimum toe level = elevation –50m.

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.

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

Overall stability

The ULS combinations of load were input into a series of Clap computer program 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 analysed, the foundation system was found to be stable, that is the computed foundation movements were finite and generally the maximum computed settlement under ULS loadings was less than 100 mm.

Cyclic loading assessment

Table 3 summarises the results of the cyclic loading assessment. Figure 4 shows the assessed factor η 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 in unlikely to occur.

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

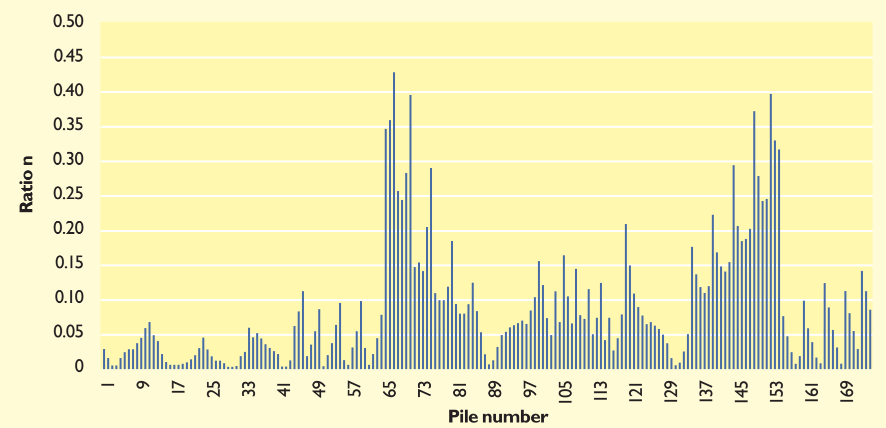

Figure 4. Assesed values of η for each pile rom cyclic loading analysis using a load case 0·75 (dead load + live load + wind load)

Table 3. Summary of cyclic loading assessment

Quantity	Value
Maximum cyclic axial load S_c*:MN	74·00
Maximum ratio, $\eta = S_c^*/R_{gs}^*$	0·43
Cyclic loading criterion satisfied?	Yes

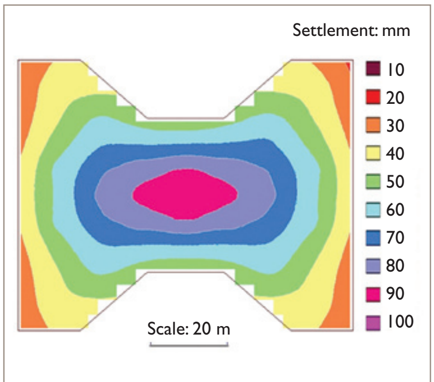

Figure 5. Typical set of computed settlement contours for the IncheonTower

Assessment of vertical pile behaviour

The vertical pile head stiffness values for each of the 172 foundation piles under serviceability loading (dead and live load) were assessed using the Clap program, which provided the geotechnical capacities, interaction factors and stiffness values for each pile under serviceability loading for input into the group assessment.

Table 4 presents a summary of the assessment, including the average vertical pile stiffness values. The pile stiffness values and interaction factors were then used in the Garp program.

Settlement

The maximum settlements predicted were about 50 mm (using the upper bound modulus) with differential settlements between the centre and edge of the slab of about 30 mm.

For the tower alone, an elastic analysis using an 'equivalent block' approach via a two-dimensional axi-symmetric finite-element analysis gave maximum and minimum settlement values of 43 mm and 18 mm, respectively, which were in good agreement with those predicted by one of the project reviewers using an equivalent block hand calculation approach (43 mm and 19 mm respectively for the tower alone).

The predicted settlements using the various analysis methods were reasonably consistent, giving a level of confidence to

Table 4. Summary of vertical pile stiffness values

Foundation layout	Minimum vertical pile stiffness: MN/m	Maximum vertical pile stiffness: MN/m	Average vertical pile stiffness: MN/m
172 piles	600	1300	820

the settlement values predicted. Figure 5 shows a typical set of settlement contours derived from the Garp analyses, while Figure 6 show the computed distribution of pile loads.

Assessment of lateral pile behaviour

A critical design issue for the tower foundation was the performance of the pile group under lateral loading. The numerical modelling packages used in the analyses comprised the following

- three-dimensional finite-element computer program Plaxis 3D Foundation
- computer program Clap.

Plaxis 3D Foundation provided an assessment of the overall lateral stiffness of the foundation. 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 5 presents typical values of computed lateral stiffness for the piled-mat foundation obtained from the analyses.

Conclusion

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.

- An overall stability analysis in which the resistances of the foundation components are reduced by the appropriate geotechnical reduction factor and the ULS load combinations are applied. The design requirements are satisfied if the foundation system does not collapse under any of the sets of ULS loadings.
- A serviceability analysis, in which the best-estimate (unfactored) values of foundation resistances and stiffnesses are employed and the SLS loads are applied. The design is satisfactory if the computed deflections and rotations are within the specified allowable limits.
- 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 and structural reduction factors. 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.

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.

For the 151 Incheon Tower in Korea, the process was used to assess overall stability, foundation settlements and pile head stiffness values as part of the design process.

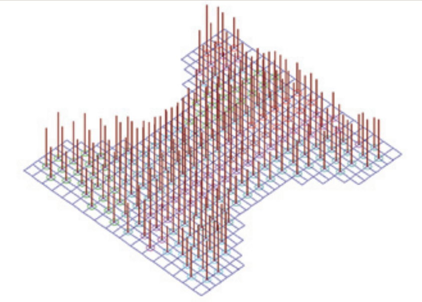

Figure 6. Computed distribution of axial pile loads for the Incheon Tower

Table 5. Summary of lateral stiffness of pile group and raft

Horizontal load: MN	Direction	Pile group displacement: mm	Lateral pile stiffness: MN/m	Lateral raft stiffness: MN/m	Total lateral stiffness: MN/m
149	X	17	8760	198	8958
115	Y	14	8210	225	8435

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