Design Challenges for Tall Building Foundations

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ABSTRACT: This paper discusses some of the challenges in designing foundations for high-rise buildings. It sets out the principles of a limit state design approach to design a pile or piled raft foundation system, which involves three sets of analyses:

1. An overall stability geotechnical analysis.
2. A geotechnical serviceability analysis.
3. An analysis to obtain foundation loads, moments and shears for structural design of the foundation system.

The importance of appropriate parameter selection and load testing is emphasized.
The approach is illustrated via its application to high-rise buildings in Dubai, Korea and Saudi Arabia.

1 INTRODUCTION

The last two decades have seen a remarkable increase in the rate of construction of "super-tall" buildings in excess of 300m in height. Fig. 1 shows the significant growth in the number of such buildings either constructed (to 2010) or projected (2015 and beyond). A large number of these buildings are in the Middle East or in China. Dubai has now the tallest building in the world, the Burj Khalifa, which is 828m in height, while in Jeddah Saudi Arabia, the Kingdom Tower is currently under construction and will eventually exceed 1000m in height.

Super-tall buildings are presenting new challenges to engineers, particularly in relation to structural and geotechnical design. Many of the traditional design methods cannot be applied with any confidence since they require extrapolation well beyond the realms of prior experience, and accordingly, structural and geotechnical designers are being forced to utilize more sophisticated methods of analysis and design. In particular, geotechnical engineers involved in the design of foundations for super-tall buildings are leaving behind empirical methods and are increasingly employing state-of-the art methods.

This paper will summarize, relatively briefly, some of the challenges that face designers of foundations for very tall buildings, primarily from a geotechnical viewpoint. Some characteristic features of such buildings will be reviewed and then the options for foundation systems will be discussed. The process and tools for foundation design will be described and then three case histories will be presented, briefly, to illustrate how some of these challenges have been addressed.

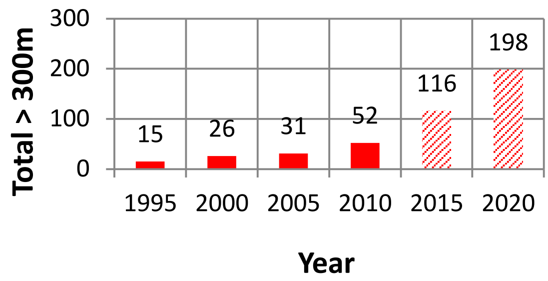


Fig. 1 Total number of buildings in excess of 300m tall (after CTBUH, 2011)

2 CHARACTERISTICS OF TALL BUILDINGS

There are a number of characteristics of tall buildings that can have a significant influence on foundation design, including the following:

- The building weight, and thus the vertical load to be supported by the foundation, can be substantial. Moreover, the building weight increases non-linearly with height, and so both ultimate bearing capacity and settlement need to be considered carefully.
- High-rise buildings are often surrounded by low-rise podium structures which are subjected to much smaller loadings. Thus, differential settlements between the high- and low-rise portions need to be controlled.
- The lateral forces imposed by wind loading, and the consequent moments on the foundation system, can be very high. These moments can impose increased vertical loads on the foundation, especially on the outer piles within the

foundation system. The structural design of the piles needs to take account of these increased loads that act in conjunction with the lateral forces and moments.

- The wind-induced lateral loads and moments are cyclic in nature. Thus, consideration needs to be given to the influence of cyclic vertical and lateral loading on the foundation system, as cyclic loading has the potential to degrade foundation capacity and cause increased settlements.
- Seismic action will induce additional lateral forces in the structure and also induce lateral motions in the ground supporting the structure. Thus, additional lateral forces and moments can be induced in the foundation system via two mechanisms:
  - Inertial forces and moments developed by the lateral excitation of the structure;
  - Kinematic forces and moments induced in the foundation piles by the action of ground movements acting against the piles.
- The wind-induced and seismically-induced loads are dynamic in nature, and as such, their potential to give rise to resonance within the structure needs to be assessed. The risk of dynamic resonance depends on a number of factors, including the predominant period of the dynamic loading, the natural period of the structure, and the stiffness and damping of the foundation system.

3 FOUNDATION OPTIONS

The common foundation options include the following:

1. Raft or mat foundations;
2. Compensated raft foundations;
3. Piled foundations;
4. Piled raft foundations;
5. Compensated piled raft foundations.

The majority of recent high rise buildings are founded on the latter three foundation types. In particular, piled raft foundations have been used increasingly. Within a piled raft foundation, it may be possible for the number of piles to be reduced significantly (as compared with a fully piled system) by considering the contribution of the raft to the overall foundation capacity. In such cases, the piles provide the majority of the foundation stiffness while the raft provides a reserve of load capacity. In situations where a raft foundation alone might be used, but does not satisfy the design requirements (in particular the total and differential settlement requirements), it may be possible to enhance the performance of the raft by the addition of piles. In such cases, the use of a limited number of piles, strategically located, may improve both the ultimate load capacity and the settlement and differential settlement performance of the raft and may allow the design requirements to be met. 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).

4 THE DESIGN PROCESS

There are commonly three broad stages employed in foundation design:

1. A preliminary design stage, which provides an initial basis for the development of foundation concepts and costing.
2. A detailed design stage, in which the selected foundation concept is analysed and progressive refinements are made to the layout and details of the foundation system. This stage is desirably undertaken collaboratively with the structural designer, as the structure and the foundation act as an interactive system.
3. A final design phase, in which both the analysis and the parameters employed in the analysis are finalized.

It should be noted that the geotechnical parameters used for each stage may change as more knowledge of the ground conditions, and the results of in-situ and laboratory testing, become available. The parameters for the final design stage should ideally incorporate the results of foundation load tests.

5 DESIGN ISSUES

The following issues will generally 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. Possible effects of externally-imposed ground movements on the foundation system, for example, movements arising from excavations for pile caps or adjacent facilities.
6. Dynamic response of the structure-foundation system to wind-induced (and, if appropriate, wave) forces.
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. 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.

The analyses required to examine the above design issues range from hand calculation methods for preliminary design to detailed three dimensional finite element analyses for final design. However, the latter analyses should always be checked for reasonableness by comparison with simpler methods and with available prior experience.

6 DESIGN ANALYSES

6.1 *Ultimate Limit State*

There is an increasing trend for limit state design principles to be adopted in foundation design, for example, in the Eurocode 7 requirements and those of the Australian Piling Code AS2159-2009. In terms of limit state design using a load and resistance factor design approach (LRFD), the design criteria for the ultimate limit state are as follows:

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

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

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

The above criteria are applied to the entire foundation system, while the structural strength criterion (Eq. 1) is also applied to each individual pile. It is not considered to be good practice to apply the geotechnical criterion (Eq. 2) to each individual pile within the group, as this can lead to considerable over-design. R_s^* and R_g^* can be obtained from the estimated ultimate structural and geotechnical capacities, multiplied by appropriate reduction factors.

The structural and geotechnical reduction factors are often specified in national codes or standards. The selection of suitable values of ϕ_g requires engineering judgment and should take into account a number of factors that may influence the foundation performance. As an example, the Australian Piling Code AS2159-2009 specifies an approach involving a subjective risk assessment, with lower values of $\phi_g$being associated with greater levels of uncertainty and higher values being relevant when ground conditions are reasonably well-known and a significant amount of load testing is to be carried out (see Section 6.6).

If any of the design 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.

6.1.1 *Cyclic Loading*

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

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

where, R_{gs}^* = design geotechnical shaft capacity,
S_c^* = maximum amplitude of wind loading,
η = a reduction factor.

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. In most cases, it is suggested that η can be taken as 0.5, while S_c^* can be obtained from computer analyses which give the cyclic component of load on each pile, for various wind loading cases.

6.2 *Serviceability*

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

$$\rho_{max} \leq \rho_{all} \tag{4}$$
$$\theta_{max} \leq \theta_{all} \tag{5}$$

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

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 (Eqs. 4 and 5).

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 Zhang and Ng also consider allowable settlements and angular distortions for shallow foundations, 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 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 rotation, and the overall allowable building tilt, reduce with increasing building height, both from a functional and a visual viewpoint.

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.
Observed Intolerable Settlement mm	349	Based on 52 cases of deep foundations.
Limiting Tolerable Angular Distortion rad	1/500	Based on 57 cases of deep foundations.
Limiting Tolerable Angular Distortion rad	1/250 (H<24m) to 1/1000 (H>100m)	From 2002 Chinese Code. H = building height
Observed Intolerable Angular Distortion rad	1/125	Based on 57 cases of deep foundations.

6.3 *Dynamic Loading*

Issues related to dynamic wind loading are generally dealt with by the structural engineer, with geotechnical input being limited to an assessment of the stiffness and damping characteristics of the foundation system. However, the following general principles of design can be applied to dynamic loadings:

- The natural frequency of the foundation system should be greater than that of the structure it supports, to avoid potential resonance phenomena. The natural frequency depends primarily on the stiffness of the foundation system and its mass, although damping characteristics may also have some influence.
- The amplitude of dynamic motions of the structure-foundation system should be within tolerable limits. The amplitude will depend on the stiffness and damping characteristics of both the foundation and the structure.

The dynamic stiffness and damping of the foundation system can be estimated via solutions and analyses such as those provided by Gazetas (1983, 1991). For high-rise structures, the first natural period is relatively large, and so dynamic effects on stiffness may be relatively small, while the radiation damping may also be small, especially for rotational modes of excitation. Thus, damping may arise primarily from internal damping of the soil. Higher modes of vibration may then become more significant.

6.4 *Earthquake Loading*

Soil deposits at a site subjected to an earthquake may experience the following effects:

- Increases in pore pressure;
- Time-dependent vertical ground movements during and after the earthquake;
- Time-dependent lateral ground movements during and after the earthquake.

In foundation design, consideration must therefore be given to possible reductions in soil strength arising from the build-up of excess pore pressures during and after the earthquake. In extreme cases, the generation of pore pressures may lead to liquefaction in relatively loose sandy and silty soils.

As a consequence of the earthquake-induced ground movements, piles and other deep foundations will be subjected to two sources of additional lateral loading:

a. Inertial loadings – these are forces that are induced in the piles because of the accelerations generated within the structure by the earthquake. Consideration is generally confined to lateral inertial forces and moments, which are assumed to be applied at the pile heads.
b. Kinematic loadings – these are forces and bending moments that are induced in the piles because of the ground movements that result from the earthquake. Such movements will interact with the piles and, because of the difference in stiffness of the piles and the moving soil, lateral stresses will be developed between the pile and the soil, resulting in the development of shear forces and bending moments in the piles. These actions will be time-dependent and need to be considered in the structural design of the piles.

Thus, in addition to the usual design considerations for static loading, the above factors of strength reduction, inertial loadings, and kinematic loadings, need to be incorporated into the design process.

With respect to the strength and stiffness of soils, consideration should also be given to the effects of the rapid rate of loading that occurs during a seismic event. In addition to generating an undrained response in finer grained soils, loading rate effects tend to increase both the strength and stiffness of such soils.

Appropriate assessment of the geotechnical parameters for earthquake response is a critical component of geotechnical design for seismic actions, as it is for other types of imposed loadings. Reference can be made to sources such as Kramer (1996) who discusses such issues as the effects of strain, cyclic loading and loading rate on soil stiffness and damping.

6.5 *Structural Design and Soil-Structure Interaction*

6.5.1 *Factoring of Resistances*

When considering soil-structure interaction to obtain foundation actions for structural design (for example, the bending moments in the raft of a piled raft foundation system), the most critical response may not occur when the pile and raft capacities are factored downwards. For example, at a pile location where there is not a column, load acting, the negative moment may be larger if the pile capacity is factored up.

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

6.5.2 *Stiffening Effect of Superstructure*

It is common in geotechnical design to analyse a raft or piled raft without considering the stiffening effect of any structure that is supported by the raft. Methods of incorporating the stiffness of a structure into a raft analysis have been examined by several authors including Lee and Brown (1972), Poulos (1975) and Brown and Yu (1986). Zhang and Small (1994) analysed 3-dimensional framed buildings on raft foundations, and demonstrated that the larger the relative stiffness of the building frame, the smaller the differential deflections in the raft. Such approaches can be extended to piled raft foundations.

GusmaoFilho and Guimaraes (1997) have looked at construction sequence and have noted that the loads in columns reach a maximum (or minimum) value as more stories are added to the building, leading to the idea of the building reaching a “limit stiffness”.

It may be concluded that the stiffness of a structure will influence the calculated settlements and differential settlements of a raft or piled raft foundation, but this depends on the stiffness of the structure relative to the raft. For buildings with rigid shear walls, the stiffening effect on the raft will be significant. However, for flexible light framed structures, the effect of the structure on a thick raft, will be small.

When undertaking a piled raft analysis, it may be convenient to represent the stiffness of the structure by using thicker raft elements at locations where are walls and larger columns. While not providing any information on the structural behaviour, such an approach can provide a more realistic assessment of differential settlements within the footprint of a structure (Russo et al, 2013).

A convenient approach to foundation-structure interaction is for the piles to be represented by springs, the stiffness of which is computed by the geotechnical engineer and which include the important effects of interaction among the piles and the raft. Such interaction can significantly reduce the axial and lateral stiffness of piles within a group, as compared with the values for an isolated single pile. In this way, a more reliable analysis can be undertaken to compute not only the structural forces, but also the pile loads, the raft moments and the distribution of settlement within the foundation system.

6.5.3 *Estimation of Pile Load Distribution*

In checking the structural loads within the piles in a piled raft system, it is essential to give proper consideration to the flexibility of the raft. Making the common assumption that the raft is rigid can lead to very misleading outcomes, as it tends to over-estimate the loads in the outer piles within the system. In addition, consideration of the superstructure stiffness in a piled raft analysis can also have a significant influence on the computed distribution of axial pile loads.

6.6 *Summary of Design Analysis Process*

A summary of the analyses that are recommended to be carried out for building foundation design is given in Table 2. These analyses involve various combinations of factored/unfactored geotechnical strengths and Ultimate Limit State (ULS) or Serviceability Limit State (SLS) loadings.

The assessment of the geotechnical reduction factor ϕ_g is an important part of the design process. Procedures are described in various codes and standards, for example Eurocode 7, and Standards Australia Piling Code (AS2159-2009). Various attempts have also been made to rationalise the selection of ϕ_g based on probabilistic methods and the achievement of a target reliability index.

In practice, a series of factors need to be considered in making an assessment of ϕ_g, such as the following (in AS2159-2009):

- The geological complexity of the site;
- The extent of ground investigation;
- The amount and quality of geotechnical data;
- Experience with similar foundations in similar geological conditions;
- The method of assessment of geotechnical parameters for design;
- The design method adopted;
- The method of utilizing the results of in-situ test data and pile installation data;
- The level of construction control;
- The level of performance monitoring of the supported structure during and after construction.

ϕ_g can typically range between 0.4 for conservative designs involving little or no pile testing and uncertain ground conditions, to 0.8 for cases in which a significant amount of testing is carried out and the ground conditions and design parameters have been carefully assessed.

Table 2. Summary of Design Analyses

Case	Purpose	Factor applied to geotechnical strength parameters	Load case
i	Geotechnical design capacity	φ_g	ULS
ii	Structural design capacity	1.0	ULS
iii	Serviceability	1.0	SLS

7 DESIGN TOOLS

7.1 *Preliminary Design*

For preliminary design, use can make use of spreadsheets, MATHCAD sheets or simple hand or computer methods which are based on reliable but simplified methods. It can often be convenient to simplify the proposed foundation system into an equivalent pier and then examine the overall stability and settlement of this pier. For the ultimate limit state, the bearing capacity under vertical loading can be estimated from the classical approach in which the lesser of the following two values is adopted:

1. The sum of the ultimate capacities of the piles plus the net area of the raft (if in contact with the soil);
2. The capacity of the equivalent pier containing the piles and the soil between them, plus the capacity of the portions of the raft outside the equivalent pier.

For assessment of the <u>average</u> foundation settlement under working or serviceability loads, the elastic solutions for the settlement and proportion of base load of a vertically loaded pier (Poulos, 1994) can be used, provided that the geotechnical profile can be simplified to a soil layer overlying a stiffer layer. Figs. 2a and 2b reproduce these solutions, from which simplified load-settlement curves for an equivalent pier containing different numbers of piles can be estimated, using the procedure described by Poulos and Davis, 1980). In these figures, the symbol definition is as follows:

P = applied load
E_s = Young's modulus of soil
E_{pe} = Young's modulus of equivalent pier (pile + soil)
d_e = diameter of equivalent pier
Is = settlement influence factor
P_b = load on base of equivalent pier.

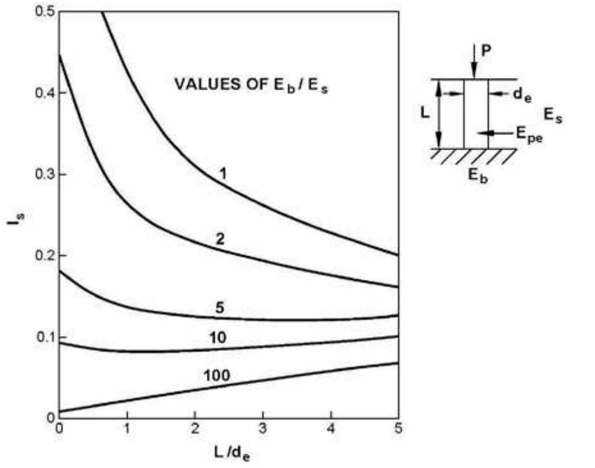


Fig. 2a Settlement of equivalent pier in soil layer (Poulos, 1994). $S = P.I_s / d_e.E_s$

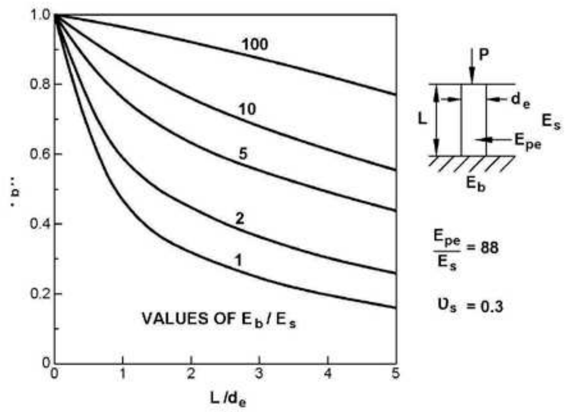


Fig. 2b.Proportion of base load for equivalent pier (Poulos, 1994).

An alternative approach can be adopted, using the "PDR" approach described by Poulos (2001). In this approach, the simplified equations developed by Randolph (1994) can be used to obtain an approximate estimate of the relationship between average settlement and the number of piles, and between the ultimate load capacity and the number of piles. From these relationships, a first estimate can be made of the number of piles, of a particular length and diameter, to satisfy the design requirements.

The load – settlement curves for a raft with various numbers of piles can be computed with the aid of a spreadsheet or a mathematical program such as MATHCAD. In this way, it is simple to compute the relationship between the number of piles and the average settlement of the foundation. Such calculations provide a rapid means of assessing whether the design philosophies for creep piling or full pile capacity utilization are likely to be feasible.

It should be recognised that such simplified methods cannot readily consider the effects of lateral and moment loading, which can have a significant effect on foundation design. Such loadings are generally dealt with during the detailed and final phases of design.

7.2 *Detailed and Final Design*

For the detailed and final design stages, more refined techniques are generally required than for preliminary design. The programs(s) used should ideally have the capabilities listed below.

1. For overall stability, the program should be able 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), and torsion;
   - 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:
   - 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 or ABAQUS, or the finite difference program FLAC3D. The pile group analysis 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.

8 GROUND INVESTIGATION AND CHARACTERIZATION

The assessment of a geotechnical model and the associated parameters for foundation design should first involve a review the geology of the site to identify any geological features that may influence the design and performance of the foundations. A desk study is usually the first step, followed by site visits to observe the topography and any rock or soil exposures. Local experience, coupled with a detailed site investigation program, is then required. The site investigation is likely to include a comprehensive borehole drilling and *in-situ* testing program, together with a suite of laboratory tests to characterize strength and stiffness properties of the subsurface conditions. Based on the findings of the site investigation, the geotechnical model and associated design parameters are developed for the site, and then used in the foundation design process.

The in-situ and laboratory tests are desirably supplemented with a program of instrumented vertical and lateral load testing of prototype piles (e.g. bi-directional load cell (Osterberg Cell) tests) to allow calibration of the foundation design parameters and hence to better predictthe foundation performance under loading. Completing the load tests on prototype piles prior to final design can provide confirmation of performance (i.e. pile construction, pile performance, ground behaviour and properties) or else may provide data for modifying the design prior to construction.

9 ASSESSMENT OF GEOTECHNICAL DESIGN PARAMETERS

9.1 *Key Parameters*

For contemporary foundation systems that incorporate both piles and a raft, the following parameters require assessment:

- The ultimate skin friction for piles in the various strata along the pile.
- The ultimate end bearing resistance for the founding stratum.
- The ultimate lateral pile-soil pressure for the various strata along the piles
- The ultimate bearing capacity of the raft.
- The stiffness of the soil strata supporting the piles, in the vertical direction.
- The stiffness of the soil strata supporting the piles, in the horizontal direction.
- The stiffness of the soil strata supporting the raft.

It should be noted that the soil stiffness values are not unique values but will vary, depending on whether long-term drained values are required (for long-term settlement estimates) or short-term undrained values are required (for dynamic response to wind and seismic forces). For dynamic response of the structure-foundation system, an estimate of the internal damping of the soil is also required, as it may provide the main source of damping. Moreover, the soil stiffness values will generally vary with applied stress or strain level, and will tend to decrease as either the stress or strain level increases.

9.2 Methods of Parameter Assessment

The following techniques are used for geotechnical parameter assessment:

1. Empirical correlations – these are useful for preliminary design, and as a check on parameters assessed from other methods.
2. Laboratory testing, including triaxial and stress path testing, resonant column testing, and constant normal stiffness testing.
3. In-situ testing, including various forms of penetration testing, pressuremeter testing, dilatometer testing, and geophysical testing.
4. Load testing, generally of pile foundations at or near prototype scale. For large diameter piles or for barrettes, it is increasingly common to employ bi-directional testing to avoid the need for substantial reaction systems.

9.3 Geophysical Testing

Geophysical testing is becoming more widely used in geotechnical investigations. At least three major advantages accrue by use of such methods:

1. Ground conditions between boreholes can be inferred.
2. Depths to bedrock or a firm bearing stratum can be estimated.

3. Shear wave velocities in the various layers within the ground profile can be measured, and tomographic images developed to portray both vertical and lateral inhomogeneity.
4. From the measured shear wave velocity, v_s, the small-strain shear modulus, G_{max}, can be obtained as follows:

$$G_{max} = \rho v_s^2 \quad (6)$$

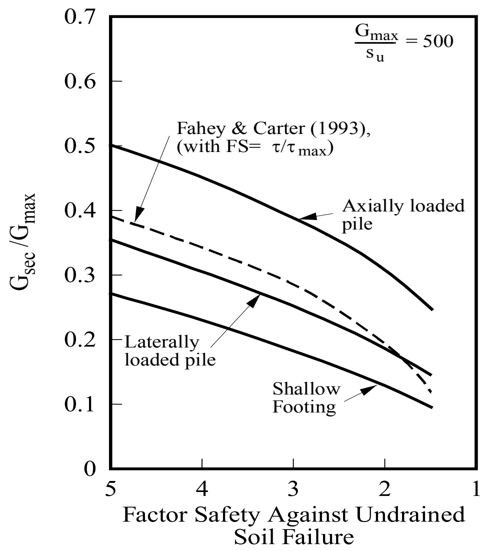
whereρ = mass density of soil.

For application to routine design, allowance must be made for the reduction in the shear modulus because of the relatively large strain levels that are relevant to foundations under normal serviceability conditions. As an example, Poulos et al (2001) have suggested the reduction factors shown in Fig. 3 for the case where $G_{max}/ s_u = 500$ (s_u = undrained shear strength). This figure indicates that:

- The secant modulus for axial loading may be about 20-40% of the small-strain value for a practical range of factors of safety;
- The secant modulus for lateral loading is smaller than that for axial loading, typically by about 30% for comparable factors of safety.

An important outcome of the strain-dependence of soil stiffness is that the operative soil modulus below the foundation system will tend to increase with depth, even within a homogeneous soil mass. When modelling a foundation system using a soil model that does not incorporate the stress- or strain–dependency of soil stiffness, it is still possible to make approximate allowance for the increase in stiffness with increasing depth below the foundation by using a modulus that increases with depth. From approximate calculations using the Boussinesq theory to compute the distribution of vertical stress with depth below a loaded foundation, it is possible to derive a relationship between the ratio of the modulus to the small strain value, as a function of relative depth and relative stress level. Such a relationship is shown in Fig. 4 for a circular foundation, together with a simplified recommendation for use in design. This may be used as an approximate means of developing a more realistic ground model for foundation design purposes. When applied to pile groups, the diameter can be taken as the equivalent diameter of the pile group, and the depth is taken from the level of the pile tips.

Fig. 3 Example of ratio of secant shear modulus to small-strain value (Poulos et al, 2001)



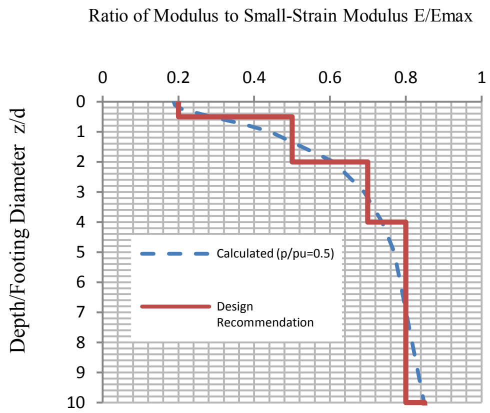


Fig. 4.Ratio of operative modulus to small-strain modulus versus relative depth below a circular foundation.Note: p/pu = ratio of applied pressure to ultimate pressure.

Haberfield (2013) has demonstrated that, when allowance is made for strain level effects, modulus values derived from geophysical tests can correlate well with those from pressuremeter tests. He shows an example in which a reduction factor of 0.2 has been applied to the small-strain modulus values along the pile, derived from cross-hole seismic test results. The modulus values so derived were found to be consistent with values obtained from subsequent pile load tests.

10 TYPICAL HIGH-RISE FOUNDATION SETTLEMENTS

It can be useful to review the settlement performance of some high-rise buildings in order to gain some appreciation of the settlements that might be expected from two foundation types founded on various deposits. Poulos (2012) summarizes details of the foundation settlements of some tall structures founded on raft or piled raft foundations, based on documented case histories in Hemsley (2000), Katzenbach et al (1998), and from the author's own experiences. The average foundation width in these cases ranges from about 40m to 100m. The results are presented in terms of the settlement per unit applied pressure, and it can be seen that this value decreases as the stiffness of the founding material increases. Typically, recorded foundation settlements have been as follows:

- between 25 and 45mm/MPa on limestone;
- between 30 and 66mm/MPa on weak rock;
- between 80 and 130mm/MPa on dense sand;
- between 220 and 310mm/MPa on stiff clay.

Some of the buildings supported by piled rafts in stiff Frankfurt clay have settled more than 100mm, and despite this apparently excessive settlement, the performance of the structures appears to be quite satisfactory. It may therefore be concluded that the tolerable settlement for tall structures can be well in excess of the conventional design values of 50-65mm. A more critical issue for such structures may be overall tilt, and differential settlement between the high-rise and low-rise portions of a project.

11 CASE HISTORIES

11.1 *The Burj Khalifa, Dubai*

The Burj Khalifa project in Dubai comprised the construction of a 160 storey high rise tower, with a podium development around the base of the tower, including a 4-6 storey garage. The Burj Khalifa Tower (originally denoted as the Burj Dubai prior to completion and opening) is currently the world's tallest building at 828m. It is founded on a 3.7m thick raft supported on 196 bored piles, 1.5 m in diameter, extending approximately 47.5 m below the base of the raft. Fig. 5 shows the completed tower.

The key challenges in this case were to undertake an economical foundation design for the world's tallest building, where the founding conditions were relatively weak rock and where significant wind loadings were to be resisted. The foundation design was undertaken by Hyder Consulting UK, with peer review by Coffey Geosciences. The final design involved the use of advanced three dimensional finite element analyses. A detailed description of this case is given by Poulos and Bunce (2008).

The geotechnical investigation was carried out in four phases and involved the drilling of 33 boreholes, with SPT testing, pressuremeter testing and geophysical testing being undertaken.

Fig. 5 The Burj Khalifa

The ground conditions comprised a horizontally stratified subsurface profile which was complex and highly variable, due to the nature of deposition and the prevalent hot arid climatic conditions. Medium dense to very loose granular silty sands (Marine Deposits) were underlain by successions of very weak to weak sandstone interbedded with very weakly cemented sand, gypsiferous fine grained sandstone/siltstone and weak to moderately weak conglomerate/calcisiltite.

Groundwater levels were generally high across the site and excavations were likely to encounter groundwater at approximately +0.0m DMD (approximately 2.5m below ground level). The ground conditions encountered in the investigation were consistent with the available geological information.

The ground profile and derived geotechnical design parameters assessed from the investigation data are described by Poulos and Bunce (2008). The values of ultimate shaft friction selected ranged between 250 kPa and 400 kPa, the drained modulus values between 40 MPa and 450 MPa, and the ultimate pile end bearing capacity was taken as 2.7 MPa.

Two programs of static load testing were undertaken for the Burj Khalifa project:

- Static load tests on seven trial piles prior to foundation construction.
- Static load tests on eight works piles, carried out during the foundation construction phase (i.e. on about 1% of the total number of piles constructed).

In addition, dynamic pile testing was carried out on 10 of the works piles for the tower and 31 of the 750 piles for the podium, i.e. on about 5% of the total works piles. Sonic integrity testing was also carried out on a number of the works piles.

Both the preliminary test piling program and the tests on the works piles provided very positive and encouraging information on the capacity and stiffness of the piles. The measured pile head stiffness values were well in excess of those predicted, and those expected on the basis of the experience with the nearby Emirates Towers. The capacity of the piles also appeared to be in excess of that predicted, and none of the tests appeared to have fully mobilized the available geotechnical resistance. The works piles performed even better than the preliminary trial piles, and demonstrated almost linear load-settlement behaviour up to the maximum test load of 1.5 times working load.

The settlements measured during construction were consistent with, but comfortably smaller than, those predicted (between 70 and 80mm), with a maximum settlement of about 44mm being measured. Overall, the performance of the piled raft foundation system exceeded expectations.

As with other high-rise projects, the Burj Khalifa involved close interaction between the structural and geotechnical designers in designing piled raft foundations for this complex and significant high-rise structure. Such interaction has some major benefits in avoiding over-simplification of geotechnical matters by the structural engineer, and over-simplification of structural matters by the geotechnical engineer, thus promoting more effective and economical foundation and structural designs.

11.2 *Incheon 151 Tower, South Korea*

A 151 storeysuper high-rise building project is currently under design,to be located in reclaimed land constructed on soft marine clay in Songdo, South Korea. This building is illustrated in Fig. 6, and is described in detail by Badelow et al (2009) and Abdelrazaq et al (2011).

The challenges in this case relate to a very tall building, sensitive to differential settlements, which is to be constructed on a reclaimed site with very complex geological conditions.

Fig. 6 Incheon 151 Tower (artist's impression)

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

A fractured zone was identified on the site, and the depth to bedrock also varied significantly. Hence, the footprint of the tower was divided into eight zones which were considered to be representative of the variation of ground conditions, with different geotechnical models being 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 silty clay under lateral and vertical loading, and hence careful consideration was given to the selection of parameters for this stratum.

The foundation comprised a concrete raft 5.5m thick with 172 piles, 2.5m in diameter, with the number and layout of piles and the pile size being obtained from a series of trial analyses through collaboration between the geotechnical and structural designers. The piles were founded a minimum of 2 diameters into the "soft" rock, or below a minimum toes level of El -50m, whichever was deeper.
The use of a suite of commercially available and in-house computer programs allowed the detailed analysis of the large group of piles to be undertaken, incorporating pile-soil-pile interaction effects, varying pile lengths and varying ground conditions in the foundation design. During final design, an independent finite element analysis was

used to include the effect of soil-structure interaction and to include the impact of the foundation system on the overall behaviour of the tower.

The overall settlement of the foundation system was estimated during all three stages of design, using the available data at that stage, and relevant calculation techniques. The predicted settlements ranged from 75mm from a simple equivalent pier analysis to 56mm from a PLAXIS 3D finite element analysis.

A total of five pile load tests were undertaken, four on vertically loaded piles via the Osterberg cell(O-cell) procedure, and one on a laterally loaded pile jacked against one of the vertically loaded test piles. For the vertical pile test, two levels of O-cells were installed in each pile, one at the pile tip and another at between the weathered rock layer and the soft rock layer.

The vertical test piles were loaded up to a maximum one way load of 150MN in about 30 incremental stages, in accordance with ASTM recommended procedures. The lateral pile load test was performed after excavation of about 8m of the upper soil, to simulate a similar ground condition as for the tower foundation. The lateral test pile was subjected to a maximum lateral load of 2.7MN.

Table 3. Summary of Vertical Pile Test Results

Strata	Quantity	Design Value	Av. Msd. Value
Soft Rock	Ult. End Bearing(MPa)	12.0	24.3
	Ult. Friction(kPa)	700	1533
Weathered Rock	Ult. Friction (kPa)	500	708

Table 3 summarizes the results of the vertical pile load tests and indicates that the actual performance, under both vertical and lateral loads, was superior to that predicted initially, thus providing scope for the development of a more cost-effective design.

Presently the tower site is fully reclaimed and fenced, and enabling works are being planned.

11.3 *Tower on Karstic Limestone, Saudi Arabia*

The identification of cavities in karstic limestone often creates a sense of anxiety among foundation designers, who may then proceed to take extreme measures to overcome the perceived dangers and high risks associated with the proximity of cavities to a foundation system.

For a high-rise project in Jeddah Saudi Arabia, involving a tower over 390-m high, potentially karstic conditions were identified in some parts of the site. Fig. 7 shows an architectural rendering of the tower. The key challenges in this project were to assess whether the adverse effects on foundation performance of cavities within the limestone would be within acceptable limits, or whether special treatment would be required to provide an adequate foundation system. A more complete description of this case is given by Poulos et al (2013).

All the available boreholes indicated the presence of coastal coralline limestone (coral reef deposits) which contained fresh shells and was typically cavernous in nature. Above these limestone deposits was a surficial soil layer which consisted mainly of aeolian sands and gravels that were deposited in Holocene times.

Originally, 12 boreholes were drilled to depths of between 40 and 75m, and subsequently, two deeper boreholes were drilled to 100 m. The borehole data showed that the soil profile consisted mainly of coralline limestone deposits that were highly fractured, and could contain cavities.

Fig. 7 Architectural rendering of tower in Jeddah, Saudi Arabia

The quantitative data from which engineering properties could be estimated was relatively limited, and included the following:

- Unconfined compression test (UCS);
- Shear wave velocity data;
- Pressuremeter testing;
- SPT data in the weaker strata.

A pile load test was also undertaken subsequently.

A piled raft foundation system was developed for this tower, as it was considered that such a system would allow the raft to redistribute load to other piles in the group if cavities caused a reduction of capacity or stiffness in some piles within the group.

The basement of the building was to be located at shallow depth above the water table, and the raft beneath the tower was 5.5m thick. It was to be supported on 145 bored piles 1.5 m in diameter, extending to a depth of 40m below the raft.

At the detailed design stage, analyses indicated that the maximum settlement was approximately 56 mm. The initial analyses assumed that no significant cavities existed below the pile toes. If cavities were to be found during construction, then it would be necessary to re-assess the performance of the foundation system and make provision for grouting of the cavities if this was deemed to be necessary. Thus, subsequent to the foundation design, a further series of analyses was undertaken to investigate the possible effects of random cavities on the settlements and also on the raft bending moments and pile loads. For these analyses, the commercially-available program PLAXIS 3D was used.

From this post-design investigation of the piled raft foundation system, the following results were obtained for the maximum computed settlement:

- With no cavities: 56mm
- With a single 3m wide, 2m deep cavity near the base of the piles: 58mm
- With 5 cavities within the footprint, having diameters ranging between 2 and 5m, and located near the pile toes: 74mm.

Some minor tilting also occurred because the cavities were not symmetrically located below the foundation footprint.

In the worst case considered, the maximum bending moments in the raft were increased by only about 13% due to the presence of the cavities, while the computed axial pile loads were largely unaffected.

It was thus demonstrated that the consequences of cavities, while not insignificant, may not be as serious as might be feared, because of the inherent redundancy of the piled raft foundation system. The analyses undertaken were insufficient to enable a quantitative assessment of risk to be assessed, but they did enable a good appreciation to be gained of the sensitivity of the computed foundation response to the presence of random cavities. Clearly, using a redundant piled raft foundation system may not only reduce the risks associated with building high-rise towers on karstic limestone, but may also provide a much more economical foundation than using deep foundation piles in an attempt to carry foundation loads through the karstic zones.

12 CONCLUSIONS

This paper has set out the following three-stage process for the design of high-rise building foundations.

1. A preliminary design stage, which provides an initial basis for the development of foundation concepts and costing.
2. A detailed design stage, in which the selected foundation concept is analysed and progressive refinements are made to the layout and details of the foundation system. This stage is desirably undertaken collaboratively with the structural designer, as the structure and the foundation act as an interactive system.
3. A final design phase, in which both the analysis and the parameters employed in the analysis are finalized.

It is emphasized that the geotechnical parameters used for each stage may change as more detailed knowledge of the ground conditions, and the results of in-situ and laboratory testing, become available. The parameters for the final design stage should desirably incorporate the results of foundation load tests.

The application of the design principles has been illustrated via three projects, each of which has presented a different challenge to the foundation designers:

1. The Burj Khalifa in Dubai – the world's tallest building, founded on a layered deposit of relatively weak rock.
2. The Incheon 151 Tower in Incheon, South Korea- a settlement sensitive building on reclaimed land, with variable geotechnical conditions across the site.
3. A high rise tower in Jeddah, Saudi Arabia – karstic conditions were present and it was necessary to assess the sensitivity of performance to the possible presence of cavities in the supporting ground.

The value of pile load testing, in conjunction with advanced methods of analysis and design, has been emphasized, as has the importance of constructive interaction between the structural and geotechnical designers.

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REFERENCES

Abdelrazaq, A., Poulos, H.G., Badelow, F. and Sung-Ho Kim (2011).“Foundation Design and Pile Testing Program for the 151 Story Incheon Tower in a Reclamation Area, Incheon, Korea.Paper TS21-02, Int. Conf. Tall Buildings & Urban Habitat (CTBUH), Seoul, Korea.

Badelow, F., Kim, S., Poulos, H.G. and Abdelrazaq, A. (2009). “Foundation design for a tall tower in a reclamation area”.Proc. 7th Int. Conf. Tall Buildings, Hong Kong, Ed. F.T.K. Au, Research Publishing, 815-823.

Brown, P.T. and K.R. Yu, 1986. “Load sequence and structure-foundation-interaction”. J. Struct. Eng., 112: 481-488.

CTBUH (2011).Council for Tall Buildings and the Urban Habitat, Press release, December 8, 2011.

Gazetas, G. (1983) “Analysis of Machine Foundation Vibrations: State of the Art”. Int. Jnl. Soil Dynamics & Earthquake Eng., Vol. 2 (1): 2-42.

Gazetas, G. (1991). “Foundation Vibrations”.Chapter 16 of Foundation Engineering Handbook, 2nd Ed., Edited H-Y Fang. Chapman & Hall, New York.

GusmaoFilho, J.A. and Guimaraes, L.J. (1997).“Limit Stiffness in Soil Structure Interaction in Buildings”.Proc. 14th Int. Conf. Soil Mechs. Found. Eng., Hamburg, Vol. 2, 807-808.

Haberfield, C.M. (2013). “Tall Tower Foundations – from concept to construction”.Advances in Foundation Engineering, Ed. K.K. Phoon et al, Research Publishing Services, 33-65.

Hemsley, J.A. (2000).”Design Applications of Raft Foundations”. Thomas Telford, London.

Horikoshi, K. and Randolph, M.F. (1998).“Optimum Design of Piled Rafts”.Geotechnique 48(3): 301-317.

Katzenbach, R., Arslan, U. Moorman, C. and Reul, O. (1998).“Piled Raft Foundations: Interaction Between Piles and Raft”.Darmstadt Geotechnics, Darmstadt University of Technology, 4: 279-296.

Lee, I.K. and Brown, P.T. “Structure-Foundation Interaction Analysis.”Journal of Structural Division, American Society of Civil Engineers, Vol.98, No. 11, pp2413-2431.

Poulos, H.G. (1975) "Settlement Analysis of Structural Foundation Systems".Proc. 4th St.-East Asian Conf. Soil Eng., Kuala Lumpur.

Poulos, H.G. (1994). “Settlement Prediction for Driven Piles and Pile Groups".Vert. and Horizl.Deformns.ofFoundns. and Embankments, Geotech. Spec. Publ. No. 40, ASCE, New York, Vol. 2, 1629-1649.

Poulos, H.G. (2001). “Piled Raft Foundations – Design and Applications”.Geotechnique, 51(2): 95-113.

Poulos, H.G. (2012). “Foundation Design for Tall Buildings”.State-of-Practice Paper, Geotechnical Engineering State of the Art and Practice, Ed. K. Rollins & D. Zekkos, ASCE GSP 226, 786-809.

Poulos, H.G. and Bunce, G. (2008).“Foundation design for the Burj Dubai – the world’s tallest building”.Proc. 6th Int. Conf. Case Histories in Geot.Eng., Arlington, VA, Paper 1.47, CD volume.

Poulos, H.G., Carter, J.P. and Small, J.C. (2001).“Foundations and Retaining Structures-Research and Practice”.Proc. 15th Int. Conf. Soil Mechs.Geot. Eng., Istanbul, 4: 2527-2606.

Poulos, H.G. and Davis, E.H. (1980).“Pile Foundation Analysis and Design”. John Wiley, New York.

Poulos, H.G**.,** Small, J.C. and Chow, H.S.W. (2013).“Foundation Design for High-Rise Towers in Karstic Ground”.ASCE GSP229, Foundation Engineering in the Face of Uncertainty, Ed. J.L. Withiam, K-K. Phoon and M.H. Hussein, 720-731.

Randolph, M.F. (1994). “Design Methods for Pile Groups and Piled Rafts”.Proc. XIII Int. Conf. Soil Mechs.& Found. Eng., New Delhi, Vol.5, 61-82.

Russo, G., Abagnara, V., Poulos, H.G. and Small, J.C. (2013). “Re-Assessment of Foundation Settlements for the Burj Khalifa, Dubai”.ActaGeotecnica, 8(1): 3-15.

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**).**

Zhang, and Small, J.C. (1994).“Finite layer analysis of soil-raft-structure interaction*.”*Proc. 13th Int. Conf. Soil Mech. FoundnEngng,New Delhi. India. Jan. *1994*. 2, 587-590. A. A. Balkema. Rotterdam. *1994*.