Lessons Learned from Designing High-rise Building Foundations

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ABSTRACT: The design of tall building foundations involves a systematic process which incorporates ground investigation, ground characterization, preliminary design of the foundation system for the anticipated structural loads, detailed foundation design, load testing of the proposed foundations, modification of the foundation design, if appropriate, and monitoring of the foundation performance as construction proceeds. This paper will describe this process and some of the tools available for implementing the process. It will then set out a series of lessons learned during the design of such foundations, and illustrate these lessons with examples from projects in Asia and the Middle East.

KEYWORDS: Analysis, Design, Foundations, Ground Interpretation, Piled raft, Settlement.

1. INTRODUCTION

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. 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 describes what is considered to be a logical process of foundation design that has been applied to a series of tall buildings. In the application of this process, a number of lessons have been learned, and some of these are summarized in the paper, with examples to illustrate each one.

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 parts of 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:
    - o Inertial forces and moments developed by the lateral excitation of the structure;
    - o 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. THE DESIGN PROCESS

3.1 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 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; de Sanctis et al, 2002).

3.2 Design Issues

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

- Ultimate capacity and global stability of the foundation system under vertical, lateral and moment loading combinations.
- The influence of the cyclic nature of wind and earthquakes on foundation capacity and movements.
- Overall foundation settlements.
- Differential settlements, both within the tower footprint, and between high-rise and adjacent low-rise areas.
- Possible effects of any externally-imposed ground movements on the foundation system, for example, movements arising from

excavation and construction operations.
- 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 forces.
- Structural design of the foundation system; including the load-sharing among the various components of the system (i.e. the piles and the supporting raft), and the distribution of loads within the piles.

3.3 Design Procedure

The following geotechnical assessment and foundation design process has been developed for high-rise building projects:

1. Geotechnical site characterization based on available ground investigation information and published data.
2. Development of representative geotechnical model(s) for the site. For geologically complex sites, more than a single model may be required.
3. Assessment of foundation requirements for ultimate limit state (bearing capacity, overall stability under combined loadings).
4. Assessment of foundation performance under serviceability loads (foundation settlements, differential settlements and lateral movements).
5. Assessment of effects of cyclic loading on foundation capacity and deformations (including cyclic degradation).
6. Assessment of loads and bending moments required for structural design of the foundation elements.
7. Assessment of dynamic response (stiffness and damping) of the foundation system.
8. Assessment of possible seismic effects, including site amplification, kinematic and inertial loadings on foundations, and liquefaction potential.
9. Consideration of the effects of dewatering, excavation and other construction activities.
10. Evaluation of load test data and modification, if necessary, of foundation design parameters.
11. Evaluation of measured performance in relation to predicted performance.

It is sound practice in high-rise building projects for the geotechnical designer to work closely with the structural designer. The superstructure and the foundation are interacting components of a single system, and should not be treated as independent entities. Such interaction can lead to more effective structural design of the foundation elements, and also, in many cases, to more realistic loadings and foundation responses.

It is also highly desirable for the geotechnical designer to be involved in the measurements of foundation performance during and after construction, particularly settlements, to allow proper assessment of that performance in relation to design expectations. If there are major differences, then it may still be possible to make amendments to the foundation design if that is deemed to be necessary.

3.4 Stages of Design

The following design stages can be employed for foundation design:

1. Concept Design;
2. Detailed Design;
3. Final Design.

These stages are described in more detail below, together with the activities that are required for each of the stages.

3.4.1 Concept Design

The aim of the Concept Design stage is to firstly establish the foundation system and to evaluate the approximate foundation behaviour. Firstly, a preliminary ground model is developed, based on the available borehole information in the vicinity of the site, supplemented with any published data and information from other sources.

In collaboration with the structural designers, a concept foundation layout is developed and its performance under preliminary ultimate and serviceability loadings is assessed.

One 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.

In using the equivalent pier method for assessment of the average 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. It should be recognized 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.

Output from the Concept Design stage includes pile geotechnical capacities for a range of pile diameters and preliminary pile layout options for various pile diameters. A Concept Design Stage report is prepared, summarizing the preliminary geotechnical model, the findings of the Concept Stage analysis and details of the most feasible foundation options to be considered in the Detailed Design stage.

3.4.2 Detailed and Final Design Stages

For the detailed and final design stages, more refined techniques are generally required than for preliminary design. The programs 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), including 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. Some proprietary programs, such as GARP (Small and Poulos, 2007) remove some of these limitations, and such programs are useful tools for the Detailed Design stage, provided their limitations are recognised and (if possible) compensated for.

4. GROUND INVESTIGATION AND CHARACTERIZATION

The assessment of a geotechnical model and the associated parameters for foundation design should first involve a review the geology and hydrogeology 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 highly desirable. 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 tests (Osterberg Cell, Osterberg, 1989) to allow calibration of the foundation design parameters and hence, to better predict the 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.

5. ASSESSMENT OF GEOTECHNICAL DESIGN PARAMETERS

5.1 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 tend to decrease as either the stress or strain level increases.

5.2 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.

5.3 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 identify any 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:

$$G_{max} = \rho \, v_s^{\,2} \tag{1}$$

where ρ = 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 Figure 1 for foundations on clay soils, 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.

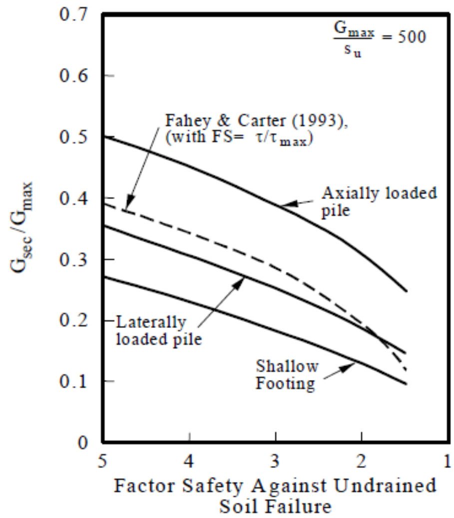

Figure 1 Example of secant shear modulus to small-strain value (Poulos et al, 2001)

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 modeling 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 Figure 2 for a circular foundation, with an overall factor of safety of 2, and 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.

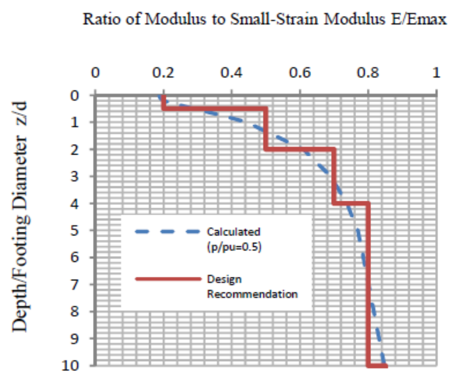

Figure 2 Ratio of operative modulus to small-strain modulus versus relative depth below circular foundation. p/p_u = ratio of applied pressure to ultimate pressure.

6. LESSONS LEARNED

6.1 Introduction

In this section, the following six lessons learned from the author's experience (out of a far larger number) will be discussed:

- Present the geotechnical design model as clearly and simply as possible.
- Proper ground characterization is at least as important as advanced numerical analysis.
- Check computer analyses with simpler methods.
- Beware of the rigid raft assumption.
- Beware of how interaction factor methods are used to estimate the settlement of large pile groups.
- Pile testing is essential.

The following sections will give examples to illustrate each of these lessons, and their consequences.

6.2 Lesson 1: Present the geotechnical design model as clearly and simply as possible

This was a lesson learned very early in the writer's career when working together with Dr T. William Lambe. Since those days, the advantages of presenting of geotechnical data on a single page (as far as is possible) have become increasingly obvious. For example, anomalies in the perceived nature of the various strata can be more easily identified when the data is all on a single sheet, rather than spread over several disparate pages. If necessary, the detail can be presented in separate diagrams, but the overall profile and characteristics should still be on a single sheet.

Another useful technique if to present not only the geotechnical profile, with the description and factual data on a single page, but also to have the developed design model and associated parameters shown on a separate diagram.

6.2.1 Case History – Tower in Jeddah, Saudi Arabia

The author carried out an independent assessment of a tall tower in Jeddah, Saudi Arabia to assess the possible effects of limestone cavities on the settlement of the tower. An architect's impression of the tower is shown in Figure 3.

Figure 3 Architectural rendering of tower in Jeddah, Saudi Arabia

As part of this assessment, a preliminary geotechnical model was developed on the basis of very limited early information. Figure 4 (from a hand-drawn original) summarises the available information and Table 1 illustrates the preliminary geotechnical model developed for the initial assessment. The parameters were derived on the basis of the data shown in Figure 4. On the basis of judgement and prior experience, the long-term values of Young's modulus were taken to be 70% of the short-term (undrained) values, except for the lowermost coralline rock layer.

In most cases, for detailed design, there would be considerably more information than that shown in Figure 4, for example, data on rock strength, pressuremeter test data, shear wave velocity data from down-hole or cross-hole geophysics, and laboratory classification test data (for soil strata).

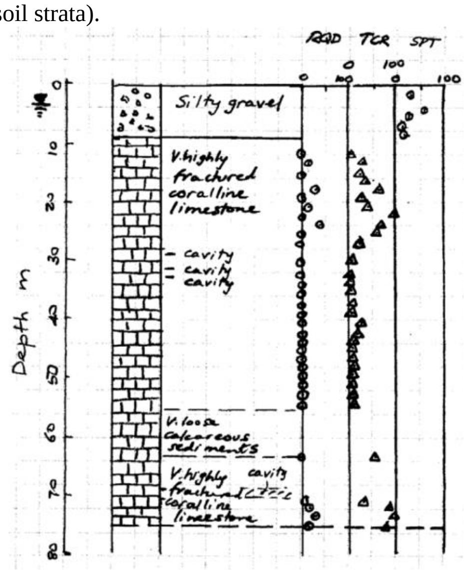

Figure 4 Geotechnical details for Jeddah site

Table 1 Summary of geotechnical model for Jeddah site

RL at bottom of geo-model (m)	Description of Geo-Unit	E_v (MPa)	E_h (MPa)	f_s (MPa)	f_b (MPa)	f_y (MPa)
-20	Coralline Limestone (1)	200	140	0.2	2	2
-50	Coralline Limestone (2)	200	140	0.2	9.8	2
-70	Coralline Limestone (3)	400	280	0.35	9.8	4
-100	Coralline Limestone (4)	1000	1000	0.4	9.8	4

The development and portrayal of geotechnical models in this way facilitates the analysis and design of the foundation system by having the key parameters all in one place, with their relevance being able to be checked against the factual data. It also facilitates the tasks of reviewers and checkers of the foundation design.

6.3 Lesson 2: Proper ground characterization is at least as important as advanced numerical analyses

It is generally well-recognized that ground investigation and characterization are critical aspects of any geotechnical engineering project, and this is particularly so for tall buildings. Important issues that should be addressed include:

- The geological history of the site, both ancient and recent. Input from an experienced engineering geologist can be critical for this stage;
- The subsurface stratigraphy of the site;
- The current and future groundwater conditions;
- The uniformity of the stratigraphy over the site;
- The in-situ stress state within the various strata and the state of overconsolidation;
- The strain levels that are likely to prevail in the various strata;
- The assessment of the key geotechnical parameters set out in Section 5.1 above, on the basis of available field and laboratory data.

6.3.1Case History – the Emirates Project, Dubai

The Emirates Project is a twin tower development in Dubai, United Arab Emirates, and is shown in Figure 5.

Figure 5 Emirates twin towers, Dubai

The towers are triangular in plan, with a face dimension of approximately 50 m to 54 m. The taller Office Tower has 52 floors and rises 355 m above ground level, while the shorter Hotel Tower is 305 m tall. The foundation system for both towers involved the use of large diameter piles in conjunction with a raft. Poulos and Davids (2005) discuss this project in detail.

The geotechnical model for foundation design under static loading conditions was based on the relevant available in-situ and laboratory test data, and is shown in Table 2. The ultimate skin friction values were based largely on laboratory constant normal stiffness direct shear tests, while the ultimate end bearing values for the piles were assessed on the basis of correlations with UCS data (Reese and O'Neill, 1988). The values of Young's modulus were derived from the data obtained from the following tests:

- seismic data (reduced by a factor of 0.2 to account for a strain level appropriate to the foundation);
- resonant column tests (at a strain level of 0.1%);
- laboratory stress path tests;
- unconfined compression tests (at 50% of ultimate stress).

While inevitable scatter existed among the different values, there was a reasonably consistent general pattern of variation of modulus with depth. Considerable emphasis was placed on the laboratory stress path tests, which, it was felt, reflected realistic stress and strain levels within the various units. The values for the upper two units were obtained from correlations with SPT data.

Table 2 Geotechnical model for Emirates Towers

Stratum	Layer Thickness m	Undrained Young's Modulus MPa	Drained Young's Modulus MPa	Ultimate Shaft Friction kPa	Ultimate End Bearing MPa
Silty Sand	5	40	30	18	0.1
"	5	125	100	73	1.5
Calcareous Sandstone	14	700	500	200	2.3
Silty Sand	10	125	100	150	1.9
Calcisiltite	20	500	400	450	2.7
"	16	90	80	200	2.0
"	10	600	600	450	2.7

In order to provide some guidance on the expected behaviour of the piles during the test pile program, "Class A" predictions of the load-deflection response of the test piles were carried out and communicated to the main consultant prior to the commencement of testing. These predictions were made using the simplified boundary element program PIES (Poulos, 1990), which was capable of incorporating non-linear pile-soil response, and of considering the effects of the reaction piles. The input parameters for the predictions were those used for the design (Table 2). Comparisons between predicted and measured test pile behaviour were made after the results of the pile tests were made available and revealed a fair measure of agreement, as shown in Figure 6. The predicted settlements slightly exceeded the measured values, and the maximum applied load of 30 MN exceeded the estimated ultimate load capacity of about 23 MN.

For the prediction of settlement of the of the piled raft foundation systems for the towers, the same geotechnical model was used as for the prediction of the settlement of the test piles. In the final design, the piles were primarily 1.2 m diameter, and extended 40 or 45 m below the base of the raft. In general, the piles were located directly below 4.5 m deep walls which spanned between the raft and the Level 1 floor slab. These walls acted as "webs" which forced the raft and Level 1slab to act as the flanges of a deep box structure. This deep box structure created a relatively stiff base to the tower superstructure, although the raft itself was only 1.5 m thick.

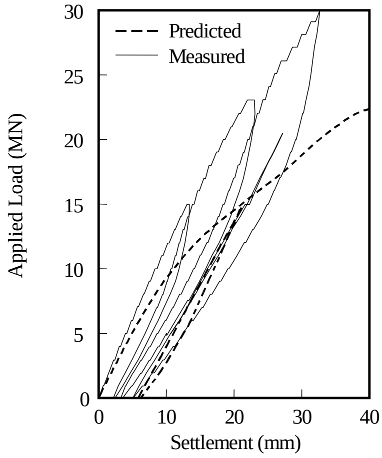

Figure 6 Predicted and measured load-settlement behaviour for single pile P3(H)

Conventional pile capacity analyses were used to assess the ultimate geotechnical capacity of the piles and raft. In additional to the conventional analyses, more complete analyses of the foundation system were undertaken with the computer program GARP (Poulos, 1994). This program utilized a simplified boundary element analysis to compute the behaviour of a rectangular piled raft when subjected to applied vertical loading, moment loading, and free-field vertical soil movements. The raft was represented by an elastic plate, the soil was modelled as a layered elastic continuum, and the piles were represented by hyperbolic springs which can interact with each other and with the raft. Beneath the raft, limiting values of contact pressure in compression and tension were specified, so that some allowance could be made for non-linear raft behaviour. In addition to GARP, the program DEFPIG (Poulos and Davis, 1980) was used for the pile stiffness values and pile-pile interaction factors, and for computing the lateral response of the piles.

For the analysis of settlements under the design loads, the same values of Young's modulus were used as for the single piles. The time-settlement predictions were based on the predicted final settlement, an assumed rate of construction, and a rate of settlement computed from three-dimensional consolidation theory. The fair agreement obtained between prediction and measurement in the pile tests had given rise to expectations that a similar level of agreement would be obtained for the foundation systems for the two towers.

Measurements were available only for a limited period during the construction process and these are compared with the predicted time-settlement relationships in Figure 7 for typical points within the Hotel Tower. To the author's disappointment, it was found that, for both towers, the actual measured settlements were significantly smaller than those predicted, being only about 25% of the predicted values after 10-12 months.

The disappointing lack of agreement between measured and predicted settlement of the towers prompted a "post-mortem" investigation of possible reasons for the poor predictions. At least four such reasons were examined:

1. Some settlements may have occurred prior to the commencement of measurements;
2. The assumed time-load pattern may have differed from that assumed;
3. The rate of consolidation may have been much slower than predicted;
4. The interaction effects among the piles within the piled raft foundation may have been over-estimated.

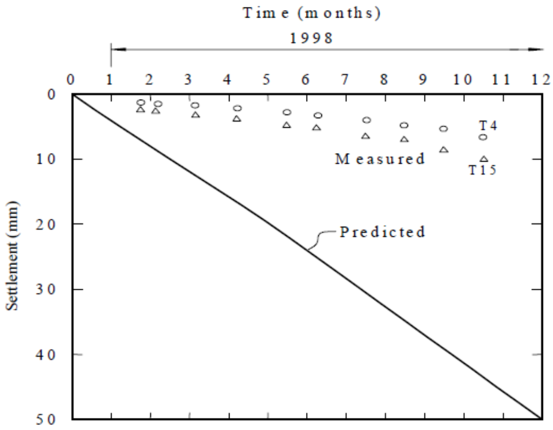

Figure 7 Predicted and measured time-settlement behaviour of Hotel tower

Of these, based on the information available during construction, the first three did not seem likely, and the last was considered to be the most likely cause. Calculations were therefore carried out to assess the sensitivity of the predicted settlements to the assumptions made in deriving interaction factors for the piled raft analysis with GARP. In deriving the interaction factors originally used, it had been assumed that the soil or rock between the piles had the same stiffness as that around the pile, and that the rock below the pile tips had a constant stiffness for a considerable depth. In reality, the ground between the piles is likely to be stiffer than near the piles, because of the lower levels of strain, and the rock stiffness below the pile tips is likely to increase significantly with depth, both because of the increasing level of overburden stress and the decreasing level of strain. The program DEFPIG was therefore used to compute the interaction factors for a series of alternative (but credible) assumptions regarding the distribution of stiffness both radially and with depth. The ratio of the soil modulus between the piles to that near the piles was increased to 5, while the modulus of the material below the pile tips was increased from the original 70 MPa to 600 MPa (the value assessed for the rock at depth). The various cases are summarized in Table 3.

Revised settlement calculations, on the basis of these interaction factors, gave the results shown in Table 3. The interaction factors used clearly have a great influence on the predicted foundation settlements, although they have almost no effect on the load sharing between the raft and the piles. The maximum settlement, for Case 4, is reduced to 29% of the value originally predicted, while the minimum settlement was only 25% of the original value. If this case were used for the calculation of the settlements during construction, the settlement at Point T15 after 10.5 months would be about 12 mm, which is in reasonable agreement with the measured value of about 10 mm.

Table 3 Summary of revised calculations for Hotel tower

Case	Modulus below 53 m MPa	Ratio of max. to near-pile modulus	Max. Settlement mm	Min. Settlement mm	% Load on Piles
Original calculations	80	1	138	91	93
Case 2	80	5	122	85	93
Case 3	200	5	74	50	92
Case 4	600	5	40	23	92
Case 5	600	1	58	32	92

The Emirates project has therefore demonstrated the vital importance of proper characterization of the ground, not only along the piles, but also beneath the piles. Especially with foundation systems of considerable width (as is typical of tall buildings), the assumptions made about ground stiffness at depth can have a profound effect on the computed settlements. In addition, if use is made of a method of analysis which involves interaction factors, such assumptions will also influence the computed values of interaction factor. This issue is examined further in Section 6.6.

6.4 Lesson 3: Check computer analyses with simpler methods

With the ready availability of powerful computer programs for foundation analysis, such as PLAXIS 3D, FLAC3D and ABAQUS, it is all too easy to become totally reliant on these to provide the analyses required to carry out a detailed foundation design. It should however be borne in mind that, even if available, such programs may not be appropriate for all stages of design, especially the earlier stages when the amount of geotechnical data may be limited. In addition, it would be imprudent to assume that the detailed analysis results are necessarily correct, as errors can occur in the data development and modeling, and some of the default values within the program may be accepted without proper consideration as to their applicability.

For these and many other reasons, simpler methods of analysis are essential for checking the foundation performance under both ultimate limit state and serviceability limit state conditions. An important aspect of the latter state is the estimation of foundation settlement, for which a number of techniques can be employed. Two of these that are widely used are the equivalent raft method and the equivalent pier method. These are discussed below.

6.4.1 Equivalent raft method

The equivalent raft method has been used extensively for estimating pile group settlements. It relies on the replacement of the pile group by a raft foundation of some equivalent dimensions, acting at some representative depth below the surface. There are many variants of this method, but the one suggested by Tomlinson (1986) appears to be a convenient and useful approach. In Tomlinson's approach, the representative depth varies from $2L/3$ to L, depending on the assessed founding conditions; the former applies to floating pile groups, while the latter value is for end bearing groups. The load is spread at an angle which varies from 1 in 4 for friction piles, to zero for end bearing groups. Once the equivalent raft has been established, the settlement can be computed from normal shallow foundation analysis.

Poulos (1993) has examined the applicability of the equivalent raft method to groups of friction piles and also end bearing pile groups. He concluded that the equivalent raft method gives a reasonably accurate prediction of the settlement of groups containing more than about 16 piles (at typical spacing of 3 pile diameters centre-to-centre). This is consistent with the criterion developed by van Impe (1991), who has concluded that the equivalent raft method should be limited to cases in which the pile cross-sections exceed about 10% of the plan area of the group.

Thus, at the very least, the equivalent raft method is a very simple and useful approach for a wide range of pile group geometries, and also provides a useful check for more complex and complete pile group settlement analyses.

Much of the success of the equivalent raft method hinges on the selection of the representative depth of the raft and the angle of load spread. Considerable engineering judgement must be exercised here, and firm rules cannot be employed without a proper consideration of the soil stratigraphy. Poulos et al (2001) have explored this issue in more detail.

6.4.2 Equivalent pier method

In this method, the pile group is replaced by a pier of similar length to the piles in the group, and with an equivalent diameter, d_e, estimated as follows (Poulos, 1993):

$$d_e \cong (1.13 \, to \, 1.27).(A_G)^{0.5} \tag{2}$$

where A_G = plan area of pile group, including the soil between the piles.

The lower value in Eq. 2 is more relevant to predominantly end bearing piles, while the larger value is more applicable to predominantly friction or floating piles.

Numerical solutions for the settlement of a pier are shown in Figure 8. Clearly, some measure of judgment needs to be exercised to assign relevant average values of Young's modulus along the shaft, and below the base, of the pier.

Poulos (1993) and Randolph (1994) have examined the accuracy of the equivalent pier method for predicting group settlements, and have concluded that it gives good results. Poulos (1993) has examined group settlement as a function of the number of piles, for a group of end bearing piles. Solutions from the computer program DEFPIG, the equivalent raft method and the equivalent pier methods were compared, and for more than about 9 piles, the settlements given by all three methods agreed reasonably well.

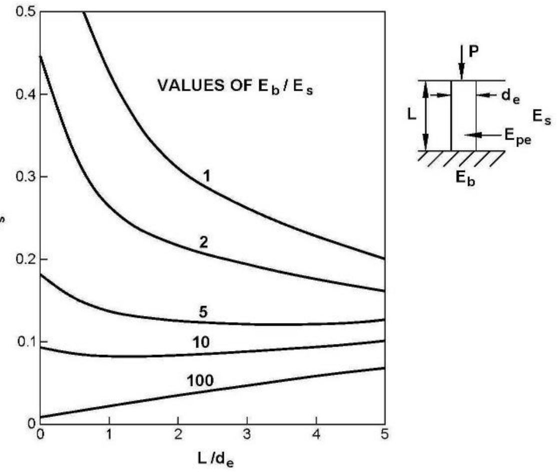

Figure 8 Settlement of equivalent pier in soil layer (Poulos, 1994)

$$S = P.I_s / d_e.E_s$$

Randolph (1994) has related the accuracy of the equivalent pier method to the aspect ratio R, of the group, where:

$$R = (ns / L)^{0.5} \tag{3}$$

where n = number of piles; s = pile centre-to-centre spacing; L = pile length.

The equivalent pier method tends to over-predict stiffness for values of R less than about 3, but the values appear to be within about 20% of those from a more accurate analysis for values of R of 1 or more, provided that the pile spacing is not greater than about 5 diameters.

6.4.3 Case History – the Burj Khalifa, Dubai

The Burj Khalifa is currently the world's tallest building and has become an iconic symbol of Dubai (see Figure 9).

Figure 9 The Burj Khalifa, Dubai

The foundation system consists of a piled raft system, with a raft 3.7m thick, supporting 196 piles, 1.5m in diameter and 47.5m long, Details of the geotechnical profile and the foundation design are given by Poulos and Bunce (2008). 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.

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. At a depth of about 20m below the pile tips, and on the basis of the lessons learned in the Emirates project, the drained Young's modulus was increased to 1200 MPa to reflect the increase in modulus with decreasing strain levels.

As part of the design, independent settlement calculations were carried out by Hyder Consulting, the foundation designers, and Coffey Geosciences, the geotechnical peer reviewers. A variety of calculation methods were used by each company, ranging from simple hand calculation methods to complex three-dimensional finite element methods. Table 4 summarises the final settlements estimated by the various methods employed. It was gratifying to note that the various methods, ranging from simplified equivalent pier method to a complex 3D finite element method, gave settlements of a similar order.

The settlements measured during construction were consistent with, but comfortably smaller than, those most of those predicted (between 70 and 80mm), with a maximum settlement of about 44mm being measured towards the end of construction, when about 80-90% of the dead load had been applied. It was anticipated that the long-term settlement would be between 50 and 60mm, which was similar to, but less than, the predicted long-term settlements.

It should be noted that a re-assessment of the settlement predictions was undertaken by Russo et al (2013), who calculated as final settlement of between about 50 and 60mm, using the computer program NAPRA.

Table 4 Summary of estimated final settlement via various methods

Method Classification	Analysis	Final Settlement mm	
		Rigid Found	Flex. Found.
Simplified	Equiv. pier	57	-
	VDISP	46	72
Pile group, interaction factors	PIGLET	62	-
	REPUTE	45	-
	PIGS	-	74
Advanced numerical	FLAC2D (FD axi-ymm)	-	73
	ABAQUS (3D FE)	-	66

6.5 Lesson 4: Beware of the rigid raft assumption!

When designing or analysing pile groups or piled rafts, it is common to make the simplifying assumption that the pile cap or raft is perfectly rigid. Because rafts in some modern high-rise buildings can be as thick as 5-6m, a rigid raft assumption may at first sight seem very reasonable. However, making this common assumption can lead to very misleading outcomes, as it tends to over-estimate the loads in the outer piles within the system and under-estimate the loads in inner piles. As a consequence, the computed values of pile head stiffness may also be affected.

This leads on to the following important question: how thick does a raft have to be to be considered as rigid? To answer this question, recourse may be made to the work of Brown (1969), who considered the behaviour of a flexible circular raft on a finite elastic laye.. Brown defined the relative flexibility of the raft via a factor K, given by:

$$K = E_r(1-\nu_s^2)(t/a)^3 /E_s \quad (4)$$

where E_r= Young's modulus of raft
ν_s = Poisson's ratio of soil
t = raft thickness
a = raft radius
E_s = Young's modulus of soil.

Brown's results indicated that a raft could be considered as perfectly flexible if $K \leq 0.01$, and virtually rigid if $K \geq 10$.

The criterion for rigidity can be facilitated by assuming that the factor K also applies to a rectangular raft having an area equal to that of the circular raft. If the average dimension of the raft is B, so that the area is B^2, then the requirement for rigidity can be approximated as follows:

$$(t/B)_{rigid} \approx \sqrt{\pi}. [K_{rigid}/(E_r/E_s). (1-\nu_s^2)]^{1/3} \quad (5)$$

Where K_{rigid} = value of K for a rigid raft, i.e. 10.

A similar equation can be written for the relative thickness, $(t/B)_{flex}$, when a raft is perfectly flexible, by substituting, in Eq. 5, the value of K for a flexible raft (i.e. 0.01) instead of that for a rigid raft.

Figure 10 plots the relationship between the relative raft thickness, t/B, for both rigid and flexible rafts, for typical values of E_r (30000MPa) and ν_s (0.3). Rafts with a t/B value on or above the line for a rigid raft would be classed as rigid, those falling on or below the line for a flexible raft would be flexible, while those falling between the lines for rigid and flexible rafts would be classed as partially flexible.

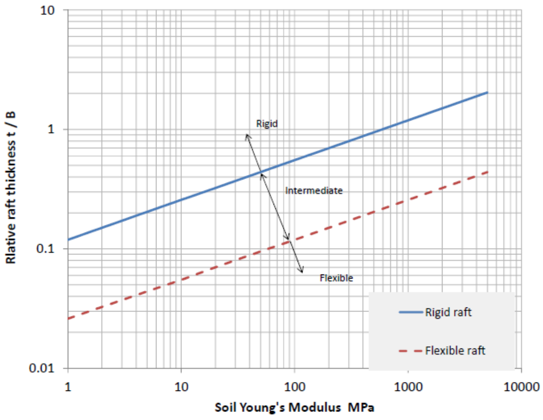

Figure 10 Thickness requirements for rigid and flexible rafts

The following points can be noted:

1. The value of $(t/B)_{rigid}$ for a rigid raft increases as the soil modulus increases.
2. Even for very soft soils, for example Es = 10 MPa, $(t/B)_{rigid}$ is about 0.25. Thus, for an average dimension of 50m, the raft would need to be about 12.5m thick to b truly rigid.
3. For a very stiff soil layer, for example, Es = 500 MPa, $(t/B)_{rigid}$ is almost 1.0. Thus, for an average dimension of 50m, the raft would need to be about 50m thick!
4. For a more common raft thickness of 3m, a raft with an average dimension of 50m would have t/B = 0.06, and this would be almost perfectly flexible even for a soft soil, and certainly perfectly flexible for the very stiff soil.

It therefore seems clear that rafts and piled rafts supporting high-rise structures are likely to tend towards the perfectly flexible category.

6.5.1 Case History – the Incheon 151 tower in South Korea

A 151 storey super 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 Figure 11, 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, to be constructed on a reclaimed site with very complex geological conditions.

Figure 11 Incheon 151 tower – architect'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 ("soft") rock.

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 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.

The detailed design analyses were carried out using an in-house computer program CLAP (Combined Load Analysis of Piles) for the ultimate limit state load cases (ULS) and the program GARP (Small and Poulos, 2007) for serviceability (SLS) loadings. As part of the design process, estimates were required of the maximum axial loads in each pile within the foundation system, and initially, the program CLAP was used. CLAP implicitly assumes that the raft supporting the piles is rigid, and as a consequence, the computed axial loads on some piles were found to be very large.

To investigate the effect of the rigid raft assumption, the foundation performance was re-assessed using GARP, and taking the flexibility of the raft into account. The serviceability load case (i.e dead and live loads) was considered and the loads were applied at column and core locations.

Table 5 presents a summary of foundation settlement, axial loads and stiffness on the corner, centre edge and centre piles of the foundation (see Figure 12). The maximum predicted settlement occured within the heavily loaded core area, while the computed pile stiffness values were greatest for the outer piles. As the analysis considered non-linear pile behaviour, the higher stiffness (and hence larger loads) for the outer piles degraded more rapidly under increasing loading than the central piles.

Considering a rigid raft for the foundation, the total and differential settlements were predicted to be smaller, with higher pile head loads for corner and centre-edge piles, thus resulting in higher vertical pile stiffness values, especially on the outer piles, when compared with those for a flexible raft.

When the flexibility of the raft was incorporated, the pile load distribution was found to be fairly uniform, with slightly higher pile loads being predicted at the centre of the foundation where the heavily loaded core is located. The loads on piles for a rigid raft case were approximately two times the loads for a flexible raft, except for the centre piles.

Table 5 Summary of foundation performance

		Rigid Raft	Flexible Raft
Pile Load (MN)	Centre Pile	24	49
	Centre Edge Pile	65	33
	Corner Pile	85	43
Pile Stiffness (MN/m)	Centre Pile	511	726
	Centre Edge Pile	1418	932
	Corner Pile	1604	1292
Raft Settlement (mm)	Maximum	52	67
	Minimum	26	28

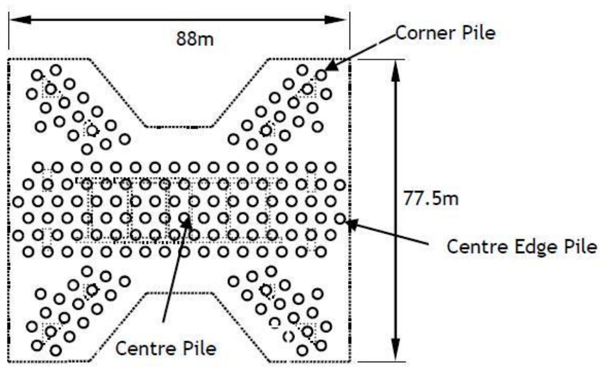

Figure 12 Foundation Layout for Incheon Tower

It is interesting to refer to Figure 10 to assess the relative flexibility of the 5.5m thick raft. The average dimension of the foundation was about 70m, so that the ratio t/B was about 0.08. The average Young's modulus within a depth equal to this dimension was about 275 MPa, and for this modulus, the value of t/B for a rigid raft would be about 0.75, i.e. the raft would need to be about 52.5m thick. In fact, even for a flexible raft, the value of t/B from Figure 10 would be about 0.17, so that the raft, with a t/B of less than half this value, could be classed as perfectly flexible. Based on the assessment, it is concluded that it is important to model the flexibility of the raft to avoid having to design for unrealistically large loads in the outer piles within the group.

6.6 Lesson 5: Beware of using interaction factor methods to estimate the settlement of large pile groups

6.6.1 The original interaction factor approach

One of the common means of analyzing pile group behaviour is via the interaction factor method described by Poulos and Davis (1980). In this method, referring to Figure 13, the settlement w_i of a pile i within a group of n piles is given as follows:

$$w_i = \sum_{j=l}^{n} (P_{av} S_1 \alpha_{ij}) \tag{6}$$

where P_{av} = average load on a pile within the group; S_1 = settlement of a single pile under unit load (i.e., the pile flexibility); α_{ij} = interaction factor for pile i due to any other pile (j) within the group, corresponding to the spacing s_{ij} between piles i and j.

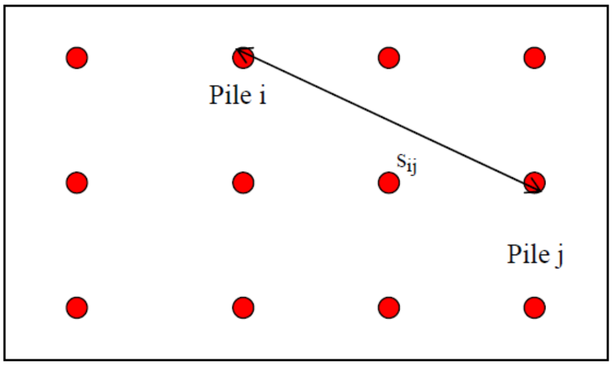

Figure 13 Superposition via the interaction factor method – plan of pile group

Eq. 6 can be written for each pile in the group, thus giving a total of n equations, which together with the equilibrium equation, can be solved for two simple cases:

1. Known load on each pile, in which case the settlement of each pile can be computed directly. In this case, there will usually be differential settlements among the piles in the group.
2. A rigid (non-rotating) pile cap, in which case all piles settle equally. In this case, there will be a uniform settlement but a non-uniform distribution of load in the piles.

In the original approach, the interaction factors were computed from boundary element analysis and plotted in graphical form. They usually took the form of plots of interaction factor α versus the ratio of pile spacing to diameter (s/d). Also, the interaction factors were applied to the total flexibility S_1 of the pile, including both elastic and non-elastic components of the single pile settlement.

In recent years, simplified or closed-form expressions for the interaction factors have been developed, thus enabling a simpler computer analysis of group settlement behaviour to be carried out. For example, Mandolini and Viggiani (1997) have developed the following simplified expressions for the interaction factor, in one of the following forms:

$$\alpha = A\,(s/d)^{B} \tag{7a}$$

$$\alpha = \{C + D\ ln\,(s/d)\} \tag{7b}$$

where A,B,C,D = fitting parameters.

For four typical field cases analyzed by Mandolini and Viggiani, the values of A ranged between 0.57 and 0.98, while the range of B was –0.60 to –1.20. For one other case, values of C= 1.0 and D = -0.26 were computed. They also assumed that no interaction occurred beyond a certain limiting value of pile spacing.

6.6.2 Modified interaction approach

Mandolini and Viggiani (1997) and Randolph (1994) have argued that the interaction factor should only be applied to the elastic component of settlement of an adjacent pile, since the plastic component of settlement is due to a localized phenomenon and is not transmitted to the adjacent piles. In this case, the settlement of a pile i in the group is then given by:

$$w_i = \sum_{j=l}^{n} (P_{av} S_{1e} \alpha_{ij}) \tag{8}$$

where S_{1e} is the elastic flexibility of the pile.

By further assuming that the load-settlement behaviour of the pile is hyperbolic, Mandolini and Viggiani (1997) expressed the interaction factor, α_{ii}, for a pile *i* due to its own load as:

$$\alpha_{ii} = 1/(1 - R_f P / P_u)^q \tag{9}$$

where R_f = hyperbolic factor (taken as unity); P = load on pile *i*; P_u = ultimate load capacity of pile *i*; q = analysis exponent = 2 for incremental non-linear analysis and 1 for equivalent linear analysis.

Figure 14 shows computed load-settlement curves for a 16-pile group subjected to axial loading. Two cases are shown:

1. For interaction factors applied to the *total* settlement of each pile;
2. For interaction factors applied only to the *elastic* (recoverable) component of settlement of each pile.

It can be seen that the settlement in the first case is greater than that from the second approach, and that the difference is considerably increased as the applied load increases. It would appear desirable to employ the approach suggested by Mandolini and Viggiani (1997) and Mandolini et al (2005), as their work indicateed that better agreement with measured group behaviour when Eq. 8 was used, rather than when the traditional approach (Eq. 6) was used.

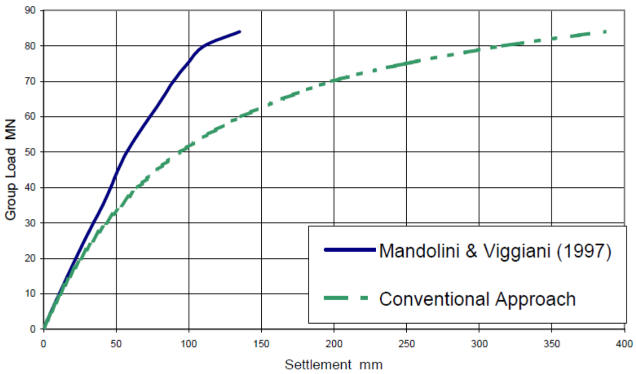

Figure 14 Effect of basis of analysis on group oad-settlement behaviour

6.6.3 Case Study – Tower in Doha, Qatar

Figure 15 shows a high-rise building in Doha, Qatar, for which an independent assessment of settlement was carried out.

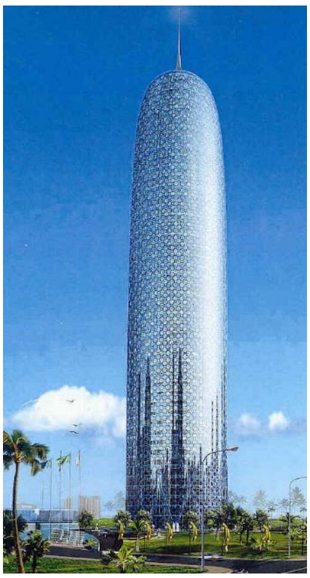

Figure 15 Doha tower

The foundation plan is shown in Figure 16 with the foundation system being designed as a piled raft. One of the design criteria was to limit the axial load in the piles to the assessed single pile safe working load (11.3MN) under the action of the working loads acting on the structure.

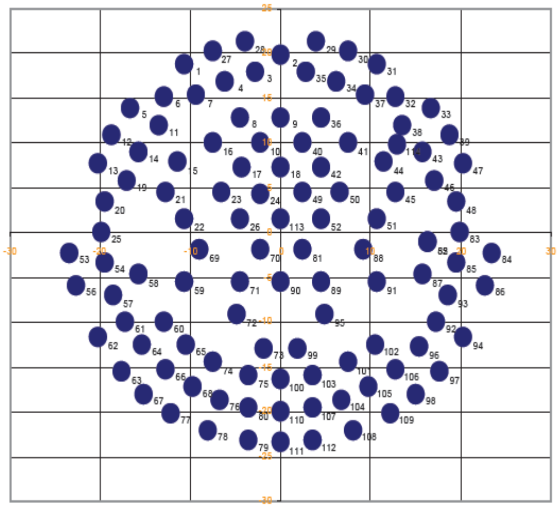

Figure 16 Foundation layout

The geotechnical model was developed on the basis of borehole data, insitu pressuremeter test data, and unconfined compression test data, and using these parameters, summarized in Table 6, the proposed 20m long, 1.2m diameter piles were estimated to have an ultimate pile capacity in compression of about 46 MN and an initial axial pile head stiffness in compression of 2360 MN/m.

Table 6 Summary of geotechnical model and parameters

Material	Layer Thickness m	Es (axial) MPa	Es (lateral) MPa	Ultimate Skin Friction KPa	Ultimate End Bearing MPa	Ultimate Lateral Pressure MPa
Limestone	10	1500	1000	560	15	15
Shale	5.5	600	400	600	10	10
Weak Limestone(1)	20	300	200	400	4.8	4.8
Weak Limestone(2)	66	150	100	-	3.4	3.4

Analyses were undertaken to compute the settlement and pile load distribution within the foundation system, taking account of the flexibility of the raft foundation. The computer program GARP (Small and Poulos, 2007) was employed, using a finite element formulation to model the raft and idealizing the piles as non-linear interacting springs. A raft thickness of 3.0m was used in the analyses, with the finite element mesh for the raft having a total of 1638 elements and 5105 nodes.

In the initial GARP analyses, the computed axial loads were quite variable and were in some cases tensile, despite the fact that the loading was compressive. It was suspected that these unexpected and intuitively unacceptable results could be due to the computed pile settlement interaction factors being too large, due to the deep layer of more compressible weak limestone formation being present below the pile tips. Subsequently, the interaction factors were re-computed, taking account of the fact that the Young's modulus of the weak limestone formation would tend to increase with depth because of the decreasing strain levels below the foundation. The consequent interaction factors were considerably smaller and the results of the GARP analyses were then much more intuitively acceptable. Figure 17 compares the two sets of interaction factors obtained.

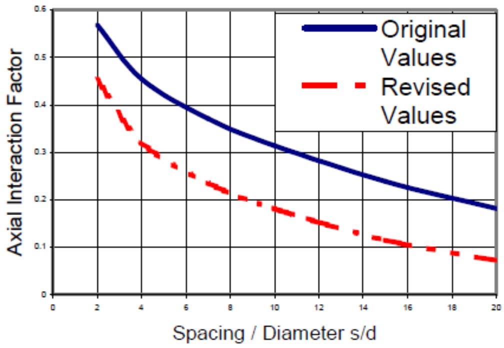

Figure 17 Influence of analysis assumption on interaction factors

The results for settlement and pile load are summarized in Table 7, and the following observations can be made:

1. With the modified interaction factors, all piles carried compressive loads under dead + live loading. The maximum pile load was about 11.3 MN, which was in fact the maximum allowable load specified.
2. The computed settlements with the modified interaction factors are considerably smaller than the originally computed values.
3. With the original interaction factors, the proposed design would be deemed to be unacceptable, because the maximum pile load is almost twice the allowable value, whereas with the revised interaction factors, the design would be deemed to be acceptable, as the computed maximum load is equal to the allowable value.

Table 7 Summary of analysis results

Quantity (Dead + Live Load)	Value with Original Interaction Factors	Value with Modified Interaction Factors
Maximum Pile Load (MN)	21.7	11.3
Minimum Pile Load (MN)	-7.3	3.4
Maximum Settlement (mm)	94	67
Minimum Settlement (mm)	25	18

It seems clear that design criteria requiring the maximum pile load to be limited to the allowable load is fraught with uncertainty and the resulting design is highly dependent on the assumptions made within the pile group analysis used. Such a sensitivity is highly undesirable and thus such criteria should be discarded. They are not only unnecessarily conservative, but they also reward a designer who uses a simplistic analysis in which pile-soil-pile interaction is ignored. In that case, under purely axial loading, all piles carry equal load, whereas it is well-known and recognized that the load distribution within a pile group is not uniform.

6.7 Lesson 6: Pile testing is essential

6.7.1 Introduction

Pile testing is a fundamental part of pile foundation design. It is one of the more effective means of dealing with uncertainties that inevitably arise during the design and construction of piles. Pile testing is usually undertaken to provide relevant information on one or more of the following issues:

1. the ultimate load capacity of a single pile;
2. the load-settlement behaviour of a pile;
3. the acceptability of the performance of a pile, as-constructed, according to specified acceptance criteria;
4. the structural integrity of a pile, as constructed.

Such information may be used in a number of ways, including:

1. construction and quality control;
2. as a means of verification of design assumptions;
3. as a means of obtaining design data on pile performance which may allow for a more effective and confident design of the piles.

Among the methods of testing that have been employed to measure the load-settlement behaviour for high-rise building foundations are the following:

- The conventional static load test;
- The Statnamic test;
- The Osterberg cell (O-cell) test.

All of these tests involve some “side effects” that have to be considered in the interpretation of the load – settlement behavior. For example, in the static load test, the presence of the reaction piles can influence the settlement of the test pile and cause an under-registration of settlement of the test pile, while for the O-cell test, the interaction between the downward and upward moving sections needs to be accounted for in assessing the overall load-settlement behavior. Further details of such side effects are given by Poulos (2000).

In addition to load-deflection testing, including lateral as well as axial loading, it has been common to employ integrity testing to assess the quality of the concrete in bored piles, usually via a system of sonic tubes placed on the reinforcement along the pile shaft.

6.7.2 Case history – the Incheon 151 Tower

Some details of this tower have been given in Section 6.5.1. As part of the foundation design program, 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 cell movement and pile head movement were measured by LVWDTs in each of four locations, and the pile strains were recorded by the strain gauges attached to the vertical steel bars. The monitoring system is shown schematically in Figure 18.

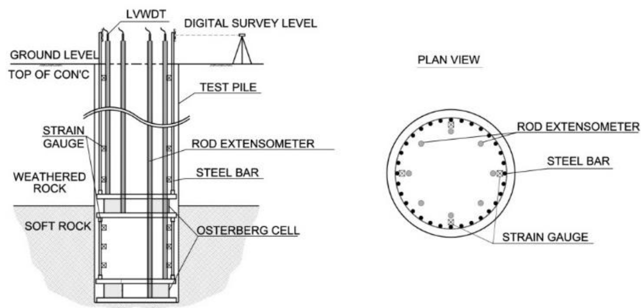

Figure 18 Schematic of monitoring for vertical pile load test

The double cell test system was planned to obtain more accurate and detailed data for the main bearing layer, and so the typical test was performed in two stages as shown in Figure 19. Stage 1 test was focused on the friction capacity of weathered rock and the movement of the soft rock socket and pile shaft in the weathered rock layer, while stage 2 focused on the friction and end bearing capacities of the soft rock, with the upper O-cell open to separate the soft rock socket from the remaining upper pile section.

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 dynamic loading-unloading test was carried out at the design loading ranges by applying 20 load cycles to obtain the dynamic characteristics of the pile rock socket.

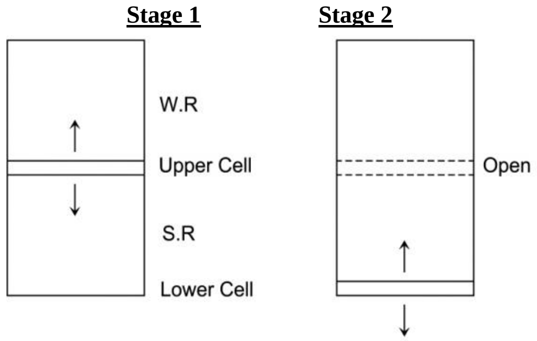

Figure 19 Typical procedure of O-Cell test

A borehole investigation was carried out at each test pile location to confirm the ground conditions and confirm the pile length and soft rock socket depth of 5-6m before piling work commenced, and also to properly match the test results to the actual ground strata. The pile tests were undertaken in mid-2010 and a summary of the vertical pile test results is shown in Table 8, which is based on the pile test interpretation performed by the Load Test Corporation.

Table 8 Summary of Vertical Pile Test Results (Allowable Pile Capacities)

Strata		*Design Value*	*Pile Test*			
			TP1	*TP2*	*TP4*	*Aver.*
Soft Rock	End Bearing(MPa)	4.0	6.3	9.0	9.2	8.1
	Friction(kPa)	350	743	897	663	767
Weathered Rock	Friction(kPa)	250	357	527	178	354

Note : F.O.S = 3 is applied for end bearing from ultimate or test load.
F.O.S = 2 for shaft friction from yield loading point.

Test Pile 3 (TP3) results are not shown in this table due to construction defects identified in the pile (Poulos et al, 2013a). The test results for Pile TP3 were ignored in obtaining the average results, but this pile is discussed further later in this section.

A lateral pile load test was also performed after excavation of about 8m of the upper soil to, simulate a similar ground condition and performance as designed for the tower foundation. Both the test pile (TP 5) and the reaction pile (TP 4) were monitored by inclinometers to obtain the lateral displacement along the pile depth, and strain gauges were installed to obtain the stress in the pile section, and eventually the bending moment distribution along the pile shaft. An LVWDT was used for each pile head displacement measurement. A schematic diagram of the monitoring system is shown in Figure 20.

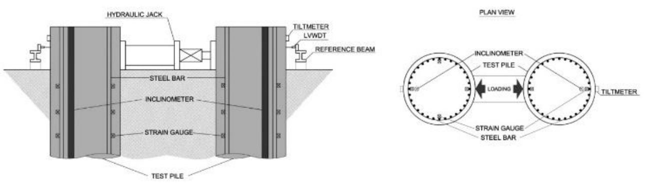

Figure 20 Schematic of monitoring for lateral pile load test

The lateral test pile was subjected to a maximum lateral load of 2.7MN. The dynamic load-unloading test was carried out at 900kN, 1350kN and 1800kN by applying 20 cycles to obtain the lateral dynamic performance of the pile, especially within the marine clay layer. The load-pile head displacement relationship from the lateral pile test is shown in the Figure 21.

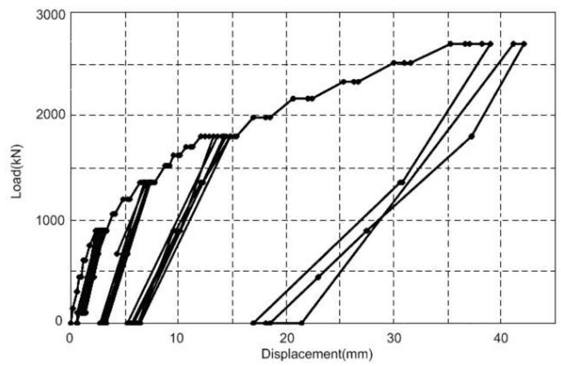

Figure 21 Load vs. Displacement curve TP5

The result indicates that the lateral stiffness of the pile was greater than expected during the initial loading stage, presumably due to the repeated loading condition and also due to the overconsolidated ground conditions arising from excavation. The stiffer behavior under cyclic loading is summarized in Table 9. This stiffer pile behavior will be also considered in the final structural design of the tower foundation system, as well as the predicted pile group movement.

Table 9 Lateral Stiffness of the Test Pile

Design Stiffness (MN/m)	*Measured Secant Stiffness of Test Pile(MN/m)*			
	Static		*Dynamic*	
	0~900kN	900~1,350kN	0~900kN	900~1,350kN
86~120	294	97	488	326

For pile TP3, a sonic logging survey was carried out 6 days after concreting of the pile was completed. An assessment of the survey results could not be carried out using the standard sonic report sheets as poor correlation was observed with apparent changes in wave velocity ("artefacts") associated with subsequent observations of irregular pipe spacing, poor pipe verticality and possible de-bonding. The summary wave trace files were therefore obtained from the testing sub-contractor, and these indicated the large range in wave speed measured and variation thereof over short and long depth intervals.

An iterative process was adopted to exclude the artefact effects mentioned above from the measured wave velocities, and the results were resolved to provide sonic tomography representations of the concrete quality along the piles length in two sections at right angles to one another. The adjusted sonic tomography plots showing variation along the pile length are shown in Figure 22.

The zones shown in red indicate concrete of sub-standard quality. The concrete quality along 70% of the pile length was reasonable, but the quality in the section of the pile within the soft rock varied, and it was considered that water entrapment may have occurred as a result of the casing being lowered to the base of the pile at the start of concreting. It was considered that the bond at the soft rock-concrete interface may have been affected in that region. The assessed 4 m long section of poor quality concrete at a depth of about 25 m was attributed to the large concrete level drop recorded during construction of the pile and possible contamination of the concrete by spoil at the top of the concrete column. This feature may also have indicated necking of the pile via a reduced pile diameter.

The results of finite element modeling indicated that the presence of irregularities in the pile cross section could result in unusually high stresses being generated within the pile section immediately below the pile over-break zones. Proper interpretation of load test data requires therefore consideration of possible non-uniformity of pile section and concrete quality. More detailed information on the integrity testing for TP3 is given by Poulos et al (2013).

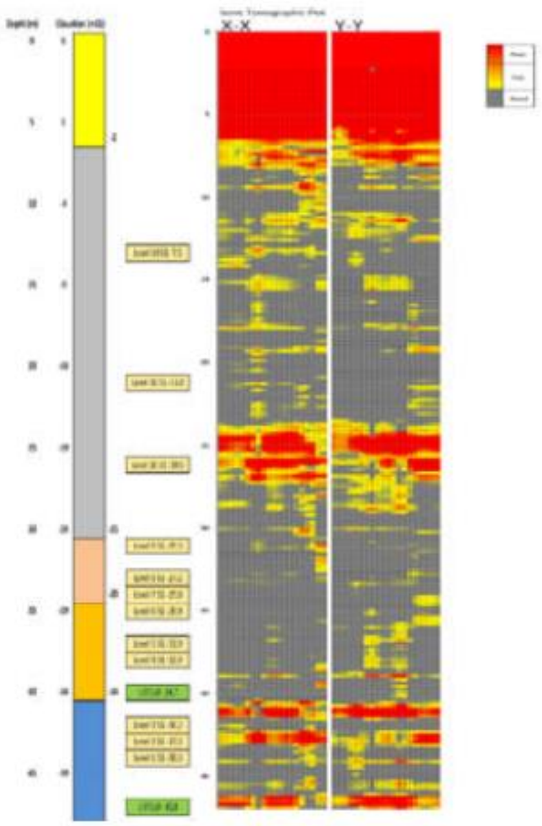

Figure 22 Tomographic image of pile

The Incheon Tower case demonstrates two important outcomes of pile load testing:

- In relation to pile performance, the tests have demonstrated that foundation economy can be achieved if the measured performance is considerably better than was predicted during design;
- In relation to pile integrity, the test on TP3 demonstrated flaws in the construction technique which could affect both the load-settlement performance and the interpretation of the detailed load distribution data. Such defects could however be eliminated in subsequent pile construction.

7. CONCLUSIONS

For the design of foundations for high-rise buildings, a three-stage process is set out involving the following stages:

- Concept design;
- Detailed;
- Final design.

The issues which have to be addressed in design are listed, and some considerations in the assessment of geotechnical parameters are discussed.

The following six lessons learned by the author have then been discussed in detail:

1. Present the geotechnical design model as clearly and simply as possible;
2. Proper ground investigation is at least as important as advanced numerical analysis;
3. Check computer analyses with simpler methods;
4. Beware of the rigid raft assumption;
5. Beware of using interaction factor methods to estimate the settlement of large pile groups;
6. Pile load testing is essential.

To illustrate each lesson, an example is presented from a case history in which the author has been involved.

Successful foundation design requires a suitable blend of relevant geotechnical data, informed geotechnical modeling, appropriate methods of analysis and checking, adequate load testing, and performance measurements. The element of geotechnical judgment remains an essential component, and one that can be enhanced by learning and heeding the lessons derived from previous projects.

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