Soils and Rocks
An International Journal of Geotechnical and Geoenvironmental Engineering
www.soilsandrocks.com
ISSN 1980-9743
ISSN-e 2675-5475

Deep foundations for tall buildings

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Article

Keywords
Design
Foundation
Parameter assessment
Pile foundation
Settlement
Tall buildings

Abstract

This paper will discuss the various challenges and issues associated with the design of foundations for tall buildings. A three-stage approach to design will be described, involving a preliminary stage, a detailed stage, and a final stage. In each stage, the parameters and the design analysis involved should be consistent. The various risk factors in the design will be outlined, together with the measures that can be taken to reduce these risks. The critical importance of the geotechnical characterization of the site will be emphasized and the assessment of the key parameters will be discussed. Two examples of the application of the design process to tall buildings, one in Dubai and the other in South Korea, will be presented. Finally, another issue that may be of importance in congested urban environments, multiple building interaction, will be mentioned.

1. Introduction

The number of tall buildings has continued to increase dramatically over the past two decades or so. This trend is illustrated in Figure 1 (CTBUH, 2024).

In addition to the increase in buildings numbers, the height of tall buildings has tended to increase, and there are now buildings that are planned to exceed 1000 m in height. With such increases in height, the foundations to support such mega-tall buildings have also had to become deeper and more robust, while at the same time, having to attempt to fulfil the contemporary requirements of sustainability.

Foundation and building behaviour is highly interactive, with the building loads influencing the foundation movements, which in turn influence the behaviour of the building. Foundation behaviour is mainly governed by the prevailing ground conditions, the foundation type, and the magnitude and distribution of the building loads. Foundation design should therefore be considered as a performance-based soil-structure interaction (SSI) issue and not limited to traditional empirically based design methods, such as a bearing capacity approach with an applied factor of safety.

The main elements in foundation design include the building loads, the ground conditions and the required building performance, as well as the other economic factors such as local construction conditions, cost and project program requirements

The critical factor in deep foundation design, especially for tall buildings, is often foundation settlement and lateral movement, rather than ultimate foundation stability. This paper will therefore be concentrated mainly on the prediction of foundation deformations, although some issues related to geotechnical and structural strength of the foundation system itself will also be addressed. Two examples of the application of the design approaches set out herein will be described, and finally, a further issue that may be of importance in congested urban environments, multiple building interaction, will be discussed briefly.

2. Foundation options

There are three main types of foundation that can be used to support tall buildings:

- Raft or mat foundations.
- Piled foundations.
- Piled raft foundations.

All three types are generally be employed with some degree of “compensation” arising from excavation for basements or underground parking and facilities.

Raft or compensated raft foundations are generally only feasible when rock is present at or near the ground surface. In most other cases, some form of deep foundation is required. While piled foundations have been widely used in the past, piled rafts or compensated piled rafts may have advantages from the point of view of cost and sustainability.

In a piled-raft foundation system, the piles provide most of the stiffness for controlling settlements at serviceability loads and the raft element provides additional capacity at ultimate loading. A geotechnical assessment for designing such a foundation system therefore needs to consider not only the capacity of the pile and raft elements, but their combined capacity and interaction under serviceability loading.

The most effective application of piled rafts occurs when the raft can provide adequate load capacity, but the

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Submitted on June 17, 2025; Final Acceptance on July 30, 2025; Discussion open until August 31, 2026.

Editor: Renato P. Cunha

https://doi.org/10.28927/SR.2026.009525

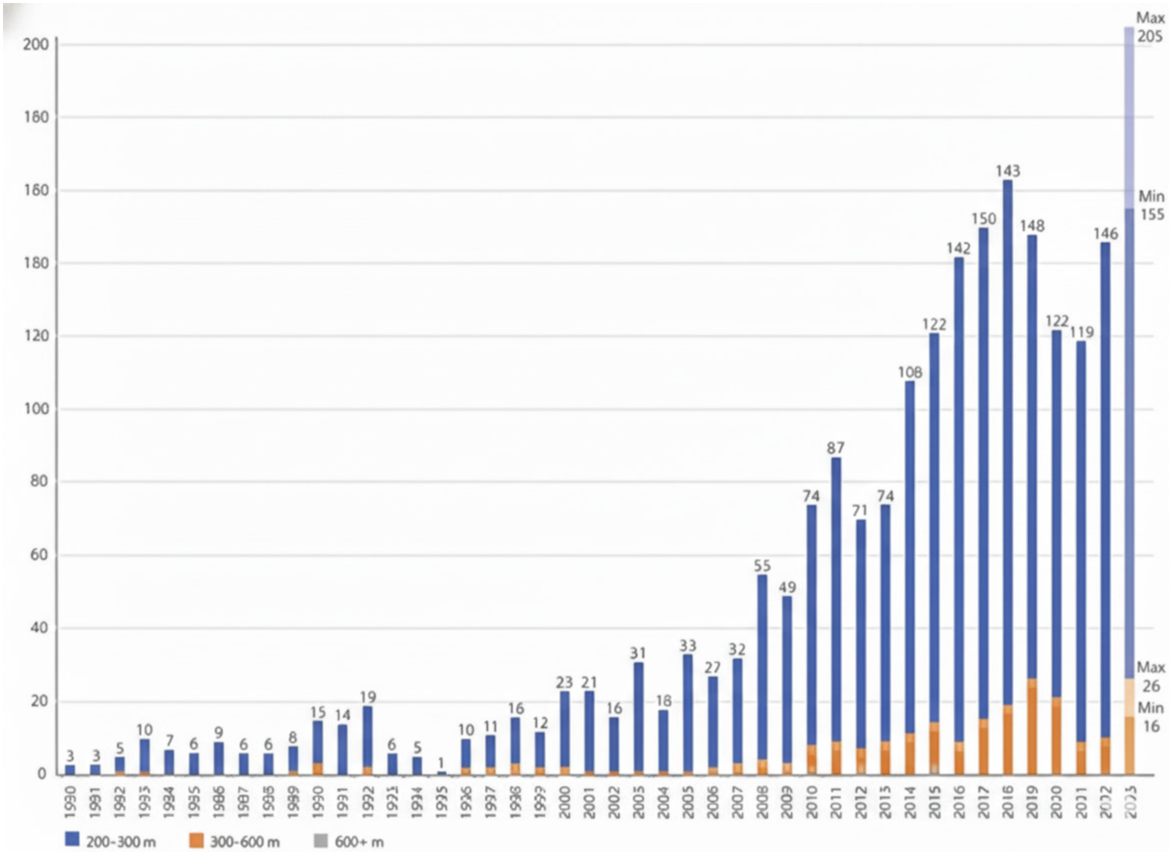

Figure 1. Number of buildings completed (CTBUH, 2024).

settlement and/or differential settlements of the raft alone exceed the allowable values. Poulos (2001) has examined a number of idealised soil profiles and found that the following situations may be favourable:

- Soil profiles consisting of relatively stiff clays.
- Soil profiles consisting of relatively dense sands.

It has been found that the performance of a piled-raft foundation can be optimised by selecting suitable locations for the piles below the raft. In general, the piles should be concentrated in the most heavily loaded areas, while the number of piles can be reduced, or even eliminated, in less heavily loaded areas (Horikoshi & Randolph, 1998).

3. Design issues

The following key issues need to be addressed in the design of deep foundations:

1. Ultimate capacity and global stability of the foundation system under vertical, lateral and moment loading combinations.
2. The influence of the cyclic nature of wind and earthquakes on foundation capacity and movements.
3. Overall foundation settlements.
4. Differential settlements, both within the structure footprint, and between high-rise and adjacent low-rise areas.
5. Possible effects of any externally-imposed ground movements on the foundation system, for example, movements arising from excavation and construction operations.
6. 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.
7. Dynamic response of the structure-foundation system to wind-induced forces.
8. 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.

4. Design criteria

Most modern methods of design now use limit state design concepts in which consideration is given to three main criteria:

- The ultimate limit state for geotechnical strength and stability;
- The ultimate limit state for structural strength and stability;
- The serviceability limit state.

The criteria for each of these aspects are discussed below.

4.1 Ultimate limit state for geotechnical strength and stability

In terms of limit state design using a load and resistance factor design approach (LRFD), the design criteria for the ultimate limit state for geotechnical design is as follows in:

$$R_{dg} \geq E_d \tag{1}$$

where R_{dg} is the design geotechnical strength ($R_{dg} = \phi_g . R_{ug}$), R_{ug} is the ultimate geotechnical capacity, ϕ_g is the geotechnical reduction factor, and E_d is the factored combination of loadings. This criterion is applied to the entire foundation system. It is not considered to be good practice to apply the geotechnical criterion to each individual pile within the group, as this can lead to considerable over-design (Poulos, 1999).

The ultimate geotechnical capacity R_{ug} can be obtained from conventional methods of design, depending on the available geotechnical data. For example, various methods are discussed by Randolph (2023) and Poulos (2017).

The selection of suitable values of ϕ_g requires considerable judgement and should take into account a number of factors that may influence the foundation performance.

The Australian standard for piling, AS 2159-2009, employs a risk assessment approach to arrive at an appropriate geotechnical reduction factor, depending on a number of issues, as follows (AS, 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.

Each of these factors is given a subjective risk rating, ranging between 1 for very low risk, to 5 for very high risk. The individual risk ratings are weighted via an importance factor for that factor, and then an average risk rating (again between 1 and 5) is computed from the sum of the individual weighted risk factors. The higher the average risk rating, the lower is the geotechnical reduction factor. Some benefit is derived by having a high redundancy foundation system, for example, a large group of piles, or a piled raft foundation. Load testing provides further benefits and leads to a higher ϕ_g value, i.e. a less conservative design.

ϕ_g can typically range between 0.4, for conservative designs involving little or no pile testing and where uncertain ground conditions prevail, 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.

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

In addition to the criterion in Equation 1, it is considered prudent that an additional criterion should be imposed for the piled foundation of a tall structure to cope with the effects of repetitive loading from wind and/or wave action, as follows in Equation 2:

$$.R_{gs} \geq E_c \tag{2}$$

where R_{gs} is the ultimate geotechnical shaft capacity, E_c is the maximum half-amplitude of cyclic wind loading, and η is the cyclic load ratio. 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. E_c can be obtained from computer analyses which gave the cyclic component of load on each pile, for various wind or seismic loading cases. For the Emirates project in Dubai, Poulos & Davids (2005), η was selected as 0.5, based on data from laboratory constant normal stiffness (CNS) tests.

4.2 Ultimate limit state for structural strength and stability

The design criterion for this limit state can be expressed as follows in Equation 3:

$$R_{ds} \geq E_d \tag{3}$$

where R_{ds} is the design structural strength ($R_{ds} = \phi_s . R_{us}$), R_{us} is the ultimate structural strength, and ϕ_s is the structural reduction factor, which is normally stipulated in standards. This criterion is applied to the entire foundation system, and also to each individual pile within the system. R_{us} can be obtained from the estimated ultimate structural capacity via an appropriate structural analysis. The strength reduction factor is usually derived from the relevant design standards or codes.

4.3 Serviceability limit state

The design criteria for the serviceability limit state can be stated as follows in Equations 4 and 5:

$$_{max} \leq \ _{all} \tag{4}$$

$$_{max} \leq \ _{all} \tag{5}$$

where ρ_{max} is the maximum computed settlement, ρ_{all} is the allowable foundation settlement, θ_{max} is the maximum local angular distortion, and θ_{all} is the allowable angular distortion. Values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Some suggested criteria have been reported by Zhang & Ng (2006) for deep foundations. Commonly specified values of maximum allowable settlement tend to be between 25 and 75 mm, depending on the nature of the building. 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 settlement criteria on very tall buildings on clay deposits, as they may not be achievable. For example, experience with tall buildings in Frankfurt Germany suggests that total settlements in excess of 100 mm can be tolerated without any apparent impairment of function.

A common criterion is θ_{all} = 1/500 (0.002), and the work of Juang et al. (2011) suggests that, for this value, there is a 20% possibility that damage could occur.

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

5. Design procedures

5.1 The design process

The following process can be employed for geotechnical assessment and deep foundation design:

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

It is sound practice 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.

5.2 Stages of design

The following design stages can be employed for foundation design:

- Concept Design;
- Detailed Design;
- Final Design.

These stages are described in more detail below, together with the activities that are required. The procedures employed for each stage should be consistent with the level of detail required. For example, sophisticated numerical analyses normally would not be appropriate for the concept design stage.

5.2.1 Concept design

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

In collaboration with the structural designers, a concept foundation layout is then developed and its performance under preliminary ultimate and serviceability loadings is assessed. Various foundation options are usually examined in this stage.

A Concept Design Stage report is prepared, summarizing the preliminary geotechnical model, the findings of the analyses undertaken, and details of the most feasible foundation options to be considered further.

5.2.2 Detailed and final phases of design

In the Detailed Design stage, pile geotechnical capacities are assessed for a range of pile diameters and preliminary pile layout options for various pile diameters. The foundation layout is adjusted and optimized to try and provide the most economical foundation system that satisfies the various design criteria.

The Final Design stage usually involves the use of a refined analysis to check the optimized solution developed in the Detailed Design phase. It provides the final values of predicted foundation performance and of pile stiffness characteristics that are then used by the structural designer.

5.3 Design analyses

A summary of the analyses that are recommended to be carried out for building foundation design are shown in Table 1. These analyses involve various combinations of factored/unfactored geotechnical strengths and Ultimate Limit State (ULS) or Serviceability Limit State (SLS) loadings.

It should be emphasized that when considering the structural design case, a geotechnical reduction factor should not be applied to the pile resistances, otherwise an unrealistic limit will be imposed on the computed forces and moments in the piles.

In addition, it should be recognized that the soil stiffness values used in the design analyses should be relevant to the loading condition being considered. Thus, for cases involving wind loading, short-term parameters should be used, whereas for long-term conditions under dead and live loading, long-term geotechnical parameters would be relevant.

Short-term soil stiffness parameters are generally larger than the corresponding long-term parameters, especially for fine-grained soils.

The above analyses should be applied to the entire foundation system, and will involve consideration of issues such as group efficiency and pile-soil-pile interaction. The criteria to be satisfied within each of the analyses are set out in Section 4 above.

5.4 Design inputs

The required inputs for a satisfactory design to be undertaken should include, but not necessarily be limited to, the following:

1. The design criteria that are being sought, such as allowable settlement, differential settlement and tilt;
2. The key geotechnical parameters: these are set out in Section 6.1 below.
3. The design loadings;
4. Details of the analysis methods to be employed and justification of their relevance.

5.5 Design outputs

The outcome of the geotechnical design process is usually a report and drawings that include, but are not limited to, the following items:

1. The interpretation of the geological and geotechnical characteristics of the site;
2. The geotechnical design parameters that have been adopted;
3. The loadings for which the design has been undertaken;
4. Details of the assessment of the Ultimate Limit State adequacy of the foundation system;
5. Details of the assessment of the Serviceability Limit State adequacy of the foundation system. Desirably, these should also include verification of the outcome via an independent analysis, albeit perhaps via a simplified method;
6. Values of the stiffness of the raft and of each pile within the foundation system. This is primarily required for the structural designer to input into the structural model to undertake a complete analysis of the structure-foundation system.

In providing equivalent spring stiffness values for the piles, an analysis of the pile group or piled raft system needs to be undertaken. In such an analysis, the following suggestions are offered:

1. For the vertical springs, it is preferable to consider an average "working" load acting on each pile, so that representative linear spring stiffness values can be obtained.
2. For the raft, to avoid undue complexity, an average spring stiffness (or modulus of subgrade reaction) can be computed on the basis of the ratio of average raft pressure to average raft settlement.
3. For the lateral and rotational springs, again it is preferable to apply an average "working" lateral load to each pile, and assume that the pile cap is able to rotate.

Table 1. Some computer programs for pile group analysis.

Program	Features
PIGLET (Randolph, 2004)	Simplified linear analysis with interaction factors
DEFPIG (Poulos, 1990)	Simplified boundary element analysis with interaction factors, non-linear capability
REPUTE (Geocentrix, 2013)	Non-linear boundary element analysis
GROUPv2022 (Ensoft (2022)	Hybrid: t-z and p-y analyses for single pile response, elastic theory for group interaction
RSPILE (Rocscience, 2025)	t-z and p-y analyses with group modifications
ELPLA (Geotec, 2025)	Elastic or Winkler-based analysis of vertically loaded group or piled raft

6. Ground characterization

The assessment of a geotechnical model and the associated parameters for foundation design should first involve a review of 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 is highly desirable, coupled with a detailed site investigation program.

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.

6.1 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.

6.2 Methods of parameter assessment

The following techniques are used for geotechnical parameter assessment:

- Empirical correlations – these are useful for preliminary design, and as a check on parameters assessed from other methods.
- Laboratory testing, including triaxial and stress path testing, resonant column testing, and constant normal stiffness (CNS) testing.
- In-situ testing, including various forms of penetration testing, pressuremeter testing, dilatometer testing, and geophysical testing.
- 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.

Detailed discussions of the methods of parameter assessment are available in several references, including Fleming et al. (2009), Tomlinson (2004), Poulos & Badelow (2015) and Poulos (2017).

6.3 Geophysical testing

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

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

From the measured shear wave velocity, v_s, the small-strain shear modulus, G_{max}, can be obtained as follows in Equation 6:

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

where ρ is the 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, Haberfield (2013) has suggested that for typical cases, the operative modulus can be approximated as 20% of the small-strain modulus value.

Poulos (2022) has also suggested some very approximate correlations between shear wave velocity and other pile design parameters such as pile skin friction and end-bearing resistance. These can provide a means of comparison with other methods of parameter assessment.

7. Design tools

7.1 Concept design

For the concept design phase, 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:

- The sum of the ultimate capacities of the piles plus the net area of the raft (if in contact with the soil);
- 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 that are ion contact with the ground.

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. Figure 2 shows the basis of the equivalent pier approximation.

Poulos (2023) has provided some approximate solutions for the settlement of a short pier within an upper layer and bearing on a lower layer of greater stiffness. A useful simplified expression for the vertical stiffness Kv of the pier is as follows in Equation 7:

$$Kv = Kv_1 + D\left(E_2 - E_1\right) / \left[1 -^2\right] \tag{7}$$

where Kv_1 is the vertical stiffness of pier wholly within material within upper layer, D is the pier diameter, E_1 is the Young's modulus of upper layer, E_2 is the Young's modulus of lower layer, and n is the Poisson's ratio of both layers. Kv_1 can be calculated from the following expression (Equation 8) provided by Bordon et al. (2021):

$$Kv_1 = E_1 . D . \ln\left(3 - 4v\right) . \left[1 + 1.08\left(1 - 0.76v\right)\left(\frac{L}{D}\right)^{0.82}\right] / \left[\left(1 - 2v\right).\left(1 + v\right)\right] \tag{8}$$

where L is the pier length.

The average settlement of the pier, S_{av}, under a vertical load V can then be expressed as in Equation 9:

$$S_{av} = V / \left[Kv_1 + \frac{D\left(E_2 - E_1\right)}{\left(1 -^2\right)}\right] \tag{9}$$

A comparison between the above approximate solutions and those obtained from finite element analyses indicates that the approximate solution is in good agreement with finite element solutions, with a tendency for the approximate solutions to be slightly conservative.

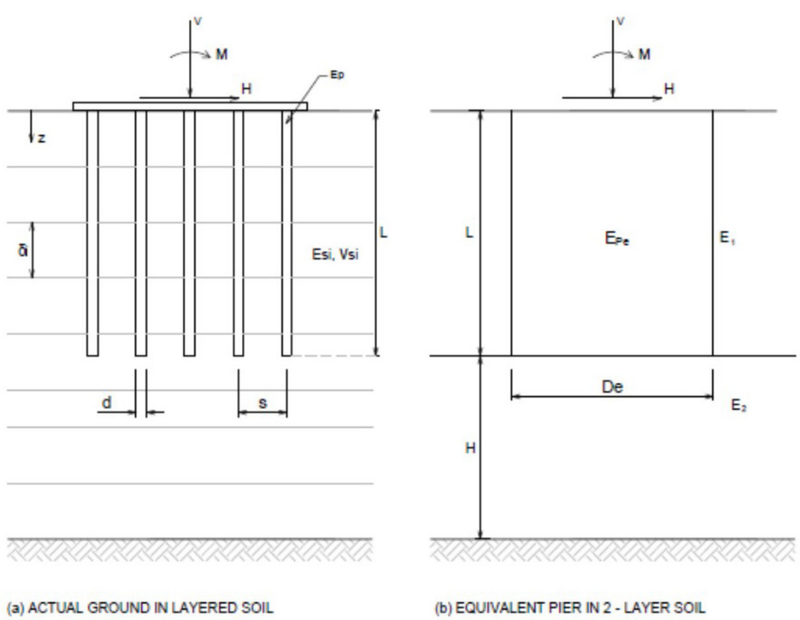

Figure 2. Equivalent pier approximation.

Corresponding solutions for the lateral response of a pier are given by Poulos (2025).

7.2 Detailed and final design

For the detailed and final design stages, more refined techniques are generally required than for preliminary design, and the programs used should ideally have a number of capabilities.

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.

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.

The pile group analysis programs REPUTE, PIGLET and DEFPIG have some of the above requirements, but fall short of a number of critical aspects, particularly in the inability of a number of programs to include raft-soil contact and raft flexibility. Table 1 summarizes some of the available pile group analysis programs.

None of the available pile group analysis software packages have all of the desirable characteristics listed above, and so increasing use is being made of three-dimensional finite element packages such as PLAXIS3D, RS3 or ABAQUS, or the finite difference program FLAC3D.

7.3 Some risk factors

There are a number of risk factors of which the foundation designer needs to be aware, and these include:

- Inappropriate design approach;
- Inappropriate design parameters;
- Inadequate investigation and consequently, inappropriate ground model;
- Geological "imperfections" such as soft or compressible layers below the pile tips, sloping bedrock of uneven layer thicknesses, and the presence of boulders;
- Construction deficiencies and imperfections.

There are measures that can be taken to reduce these risks, including the following:

- Independent peer review of the ground investigation report;
- Independent peer review of the foundation design;
- Independent supervision of the construction process;
- Foundation element testing, e.g. load tests on pile (or barrettes) proposed to be used;
- Monitoring of the settlements during construction, and comparison with predicted settlements.

Adoption of these measures should be treated as insurance investments, rather than simply as additional costs.

8. Examples of tall building design

This Section will discuss two examples where the general design principles set out above been applied. Both cases have been documented previously, but will be treated in more detail than in some earlier sources, and are considered to be useful in illustrating the application of many of the principles of design described above.

8.1 Burj Khalifa

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 client for the project was Emaar, a leading developer based in Dubai. The Burj Khalifa Tower (originally denoted as the Burj Dubai prior to completion and opening) is currently the world's tallest building at 828 m. It is founded on a 3.7 m thick raft supported on bored piles, 1.5 m in diameter, extending approximately 50 m below the base of the raft. Figure 3 shows the completed tower. The site is generally level and site levels are related to Dubai Municipality Datum (DMD).

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. A detailed description of this case is given by Poulos & Bunce (2008).

The building was 'Y' shaped in plan, to reduce the wind forces on the tower and to keep the structure relatively simple and aid constructability. Baker et al. (2007) describe the structural system as a "buttressed core". Each wing had its own high-performance concrete corridor walls and perimeter columns, and buttressed the others via a six-sided central core or hexagonal hub. As a consequence, the tower was very stiff laterally and torsionally.

8.1.1 Geotechnical investigation and testing program

The geotechnical investigation was carried out in four phases as follows:

- *Phase 1 (main investigation):* 23 boreholes, in situ SPT's, 40 pressuremeter tests in 3 boreholes, installation of 4 standpipe piezometers, laboratory testing, specialist laboratory testing and contamination testing – 1st June to 23rd July 2003;

Figure 3. Completed Burj Khalifa tower.

- *Phase 2 (main investigation):* 3 geophysical boreholes with cross-hole and tomography geophysical surveys carried out between 3 new boreholes and 1 existing borehole – 7th to 25th August, 2003;
- *Phase 3:* 6 boreholes, in situ SPT's, 20 pressuremeter tests, installation of 2 standpipe piezometers and laboratory testing – 16th September to 10th October 2003;
- *Phase 4:* 1 borehole, in situ SPT's, cross-hole geophysical testing in 3 boreholes and down-hole geophysical testing in 1 borehole and laboratory testing.

The drilling was carried out using cable percussion techniques with follow-on rotary drilling methods to depths between 30 m and 140 m below ground level. The quality of core recovered in some of the earlier boreholes was somewhat poorer than that recovered in later boreholes, and therefore the defects noted in the earlier rock cores may not have been representative of the actual defects present in the rock mass. Phase 4 of the investigation was targeted to assess the difference in core quality and this indicated that the differences were probably related to the drilling fluid used and the overall quality of drilling.

Disturbed and undisturbed samples and split spoon samples were obtained from the boreholes. Undisturbed samples were obtained using double tube core barrels (with Coreliner) and wire line core barrels producing core varying in diameter between 57 mm and 108.6 mm. Standard Penetration Tests (SPTs) were carried out at various depths in the boreholes and were generally carried out in the overburden soils, in weak rock or soil bands encountered in the rock strata.

Pressuremeter testing, using an OYO Elastmeter, was carried out in 5 boreholes between depths of about 4 m to 60 m below ground level typically below the Tower footprint.

The geophysical survey comprised cross-hole seismic survey, cross-hole tomography and downhole geophysical survey. The main purpose of the geophysical survey was to complement the borehole data and provide a check on the results obtained from borehole drilling, in situ testing and laboratory testing.

The cross-hole seismic survey was used to assess compression (*P*) and shear (*S*) wave velocities through the ground profile. Cross-hole tomography was used to develop a detailed distribution of P-wave velocity in the form of a vertical seismic profile of P-wave with depth, and to highlight any variations in the nature of the strata between boreholes. Down-hole seismic testing was used to determine shear (*S*) wave velocities through the ground profile.

The geotechnical laboratory testing program consisted of two broad classes of test:

- Conventional tests, including moisture content, Atterberg limits, particle size distribution, specific gravity, unconfined compressive strength, point load index, direct shear tests, and carbonate content tests.
- Sophisticated tests, including stress path triaxial, resonant column, cyclic undrained triaxial, cyclic simple shear and constant normal stiffness (CNS) direct shear tests. These tests were undertaken by a variety of commercial, research and university laboratories in the UK, Denmark and Australia.

8.1.2 Geotechnical conditions

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 +0.0 m DMD (approximately 2.5 m below ground level). The ground conditions encountered in the investigation were consistent with the available geological information.

Table 2. Geotechnical profile and parameters.

Stratum	Thickness (m)	E_u (MPa)	E' (MPa)	f_s (kPa)
1: (Silty sand)	3.7	34-12	30-10	-
2: (v. weak-weak calcarenite)	6.1	500	400	-
3: Calcareous sand & sandstone, v. weak-weak	16.7	50-250-140	40-200-110	250
4: Gypsiferous/calcareous sandstone, v. weak-weak	4.5	140	110	250
5: Calcisiltite, v. weak-weak	40	310-405	250-325	285-325
6: Calcareous conglomerate, v. weak-weak	22.5	560	450	400
7: Claystone/siltstone, v. weak-weak, with gypsum layers	>47	405	325	325

The ground profile and derived geotechnical design parameters assessed from the investigation data and adopted by the foundation designer (Hyder Consulting Ltd. UK) are summarized in Table 2. In this table, E_u is the undrained Young's modulus, E' is the drained Young's modulus, and f_s is the ultimate shaft friction. E_u and E' values relate to relatively large strain levels in the strata below the structure

Non-linear stress-strain responses were derived for each strata type using the results from the SPT values, the pressuremeter, the geophysics and the standard and specialist laboratory testing. An allowance for degradation of the mass stiffness of the materials was incorporated in the derivation of the non-linear stress-strain curves used in the numerical design analyses.

An assessment of the potential for degradation of the stiffness of the strata under cyclic loading was carried out through a review of the CNS (Constant Normal Stiffness) and cyclic triaxial specialist test results, and also using the computer program SHAKE91 (Idriss & Sun, 1992) for potential degradation under earthquake loading. The results indicated that there was a potential for degradation of the mass stiffness of the materials, but limited potential for degradation at the pile-soil interface.

8.1.3 Foundation design

An assessment of the foundations for the structure was carried out and it was clear that piled foundations would be appropriate for both the Tower and Podium construction. An initial assessment of the pile capacity was carried out using the following design recommendations given by Horvath et al. (1983), as presented by Burland & Mitchell (1989) in Equation 10:

$$f_s = 0.25\left(q_u\right)^{0.5} \tag{10}$$

where f_s is the ultimate unit shaft resistance in kPa, and q_u is the uniaxial compressive strength in MN/m^2. The adopted ultimate compressive unit shaft friction values for the various site rock strata adopted by the foundation designer are tabulated in Table 2. The ultimate unit pile skin friction of a pile loaded in tension was taken, conservatively, as half the ultimate unit shaft resistance of a pile loaded in compression. The initial ABAQUS runs indicated that the strains in the strata were within the initial small strain region of the non-linear stress strain curves developed for the materials. The secant elastic modulus values at small strain levels were therefore adopted for the validation and sensitivity analyses carried out using PIGLET and REPUTE. A non-linear analysis was carried out in VDISP using the non-linear stress strain curves developed for the materials.

Table 3. Computed settlements.

Analysis method	Settlement mm Rigid	Settlement (mm)
FEA	56	66
REPUTE	45	-
PIGLET	62	-
VDISP	46	72

Linear and non-linear analyses were carried out to obtain predictions for the load distribution in the piles and for the settlement of the raft and podium. The assessed pile capacities were provided to the structural designers and they then supplied details on the layout, number and diameter of the piles. Tower piles were 1.5m diameter and 47.45 m long with the tower raft founded at -7.55 m DMD. The podium piles were 0.9 m diameter and 30 m long with the podium raft being founded at -4.85 m DMD. The thickness of the raft was 3.7 m. Loading was provided by the structural designer (SOM) and comprised 8 load cases including four load cases for wind and three for seismic conditions.

The settlements from the FE Analysis (FEA) model and from VDISP were converted from those for a flexible pile cap to those for a rigid pile cap (δ_{rigid}) for comparison with the REPUTE and PIGLET models using the following approximate Equation 11:

$$\delta_{rigid} = \frac{1}{2}\left(\delta_{centre} + \delta_{edge}\right)_{flexible} \tag{11}$$

The computed settlements are shown in Table 3, for the case of the tower only (dead + live load). The settlements from the FEA model correlated acceptably well with the results obtained from REPUTE, PIGLET and VDISP.

A sensitivity analysis was carried out using the FE analysis model and applying the maximum design soil strata non-linear stress-strain relationships. The results from the stiffer soil strata response gave a 28% reduction in Tower settlement for the combined Dead load, live load and wind load case analyzed, from 85 mm to 61 mm.

The maximum and minimum pile loadings were obtained from the FE analysis for all loading combinations. The maximum loads were at the corners of the three "wings" and were of the order of 35 MN, while the minimum loads were within the center of the group and were of the order of 12-13 MN.

The impact of cyclic loading on the pile was an important consideration and in order to address this, the load variation above or below the dead load plus live load cases was determined. The maximum load variation was found to be less than 10 MN.

8.1.4 Overall stability assessment

The minimum centre-to-centre spacing of the piles for the tower was 2.5 times the pile diameter. A check was therefore carried out to ensure that the Tower foundation was stable both vertically and laterally, assuming that the foundation acted as a block comprising the piles and soil/rock. A factor of safety of slightly less than 2 was assessed for vertical block movement, excluding base resistance of the block while a factor of safety of greater than 2 was determined for lateral block movement excluding passive resistance. A factor of safety of approximately 5 was obtained against overturning of the block.

8.1.5 Liquefaction assessment

An assessment of the potential for liquefaction during a seismic event at the Burj Dubai site was carried out using the Japanese Road Association Method and the conventional SPT-based method. Both approaches gave similar results and indicated that the Marine Deposits and sand to 3.5m below ground level (from +2.5 m DMD to –1.0 m DMD) could potentially liquefy. However, the foundations of the Podium and Tower structures were below this level. Consideration was however required in the design and location of buried services and shallow foundations which were within the top 3.5 m of the ground. Occasional layers within the sandstone layer between – 7.3 m DMD and –11.75 m DMD could potentially liquefy. However, taking into account the imposed confining stresses at the foundation level of the Tower this potential liquefaction was considered to have a negligible effect on the design of the Tower foundations. The assessed reduction factor to be applied to the soil strength parameters, in most cases, was found to be equal to 1.0 and hence liquefaction would have a minimal effect upon the design of the Podium foundations. However, consideration was given in design for potential downdrag loads on pile foundations constructed through the liquefiable strata.

8.1.6 Independent verification analyses

The author was involved in the independent verification analyses. The parameters were assessed independently on the basis of the available information and experience gained from the nearby Emirates project (Poulos & Davids, 2005). In general, this model was rather more conservative than the original model employed by the foundation designer for the design. In particular, the ultimate end bearing capacity was reduced together with the Young's modulus in several of the upper layers, and the presence was assumed of a stiffer layer, with a modulus of 1200 MPa below RL –70 m DMD, to allow for the fact that the strain levels in the ground decrease with increasing depth.

The following three-stage approach was employed for the independent verification process:

- The commercially available computer program FLAC was used to carry out an axisymmetric analysis of the foundation system for the tower. The foundation plan was represented by a circle of equal area, and the piles were represented by a solid block containing piles and soil. The axial stiffness of the block was taken to be the same as that of the piles and the soil between them. The total dead plus live loading was assumed to be uniformly distributed. The soil layers were assumed to be Mohr-Coulomb materials, with the modulus values as presented in Poulos (2016), and values of cohesion taken as 0.5 times the estimated unconfined compressive strength. The main purpose of this analysis was to calibrate and check the second, and more detailed, analysis, using the computer program for pile group analysis, PIGS (Poulos, 2008).
- An analysis using PIGS was carried out for the tower alone, to check the settlement with that obtained by FLAC. In this analysis, the piles were modeled individually, and it was assumed that each pile was subjected to its nominal working load of 30 MN. The stiffness of each pile was computed via the program DEFPIG (Poulos, 1990), allowing for contact between the raft section above the pile and the underlying soil. The pile stiffness values were assumed to vary hyperbolically with increasing load level, using a hyperbolic factor (R_f) of 0.4.
- Finally, an analysis of the complete tower-podium foundation system was cried out using the in-house program PIGS, and considering all 926 piles in the system. Again, each of the piles was subjected to its nominal working load.

Because of the difference in shape between the actual foundation and the equivalent circular foundation, only the maximum settlement was considered for comparison purposes. The following results were obtained for the central settlement:

- FLAC analysis, using an equivalent block to represent the piles: 72.9 mm
- PIGS analysis, modeling all 196 piles: 74.3 mm

Thus, despite the quite different approaches adopted, the computed settlements were in remarkably good agreement. It should be noted that, as found with the Emirates project, the computed settlement is influenced by the assumptions made regarding the ground properties below the pile tips. For example, if in the PIGS analysis the modulus of the ground below RL70 m DMD was taken as 400 MPa (rather than 1200 MPa), the computed settlement at the centre of the tower would increase to about 96 mm.

8.1.7 Cyclic loading effects

The possible effects of cyclic loading were investigated via the following means:

- Cyclic triaxial laboratory tests;
- Cyclic direct shear tests;
- Cyclic Constant Normal Stiffness (CNS) laboratory tests;
- Via an independent theoretical analysis carried out by the independent verifier.

The cyclic triaxial tests indicated that there was some potential for degradation of stiffness and accumulation of excess pore pressure, while the direct shear tests indicated a reduction in residual shear strength, although these were carried out using large strain levels which were not representative of the likely field conditions.

The CNS tests indicated that there is not a significant potential for cyclic degradation of skin friction, provided that the cyclic shear stress remains within the anticipated range.

The independent analysis of cyclic loading effects was undertaken using the approach described by Poulos (1988), and implemented via the computer program SCARP (Static and Cyclic Axial Response of Piles). This analysis involved a number of simplifying assumptions, together with parameters that were not easily measured or estimated from available data. As a consequence, the analysis was indicative only. Since the analysis of the entire foundation system was not feasible with SCARP, only a typical pile (assumed to be a single isolated pile) with a diameter of 1.5m and a length of 48m was considered. The results were used to explore the relative effects of the cyclic loading, with respect to the case of static loading.

It was found that a loss of capacity would be experienced when the cyclic load exceeded about ± 10 MN. The maximum loss of capacity (due to degradation of the skin friction) was of the order of 15-20%. The capacity loss was relatively insensitive to the mean load level, except when the mean load exceeded about 30 MN. It was predicted that, at a mean load equal to the working load and under a cyclic load of about 25% of the working load, the relative increase in settlement for 10 cycles of load would be about 27%.

The indicative pile forces calculated from the ABAQUS finite element analysis of the structure suggested that cyclic loading of the Burj Tower foundation would not exceed ± 10 MN. Thus, it seemed reasonable to assume that the effects of cyclic loading would not significantly degrade the axial capacity of the piles, and that the effects of cyclic loading on both capacity and settlement were unlikely to be significant.

8.1.8 Pile load testing

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 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. Attention here is focused on the static load tests.

The details of the piles tested within the preliminary test program are summarized in Table 4. The main purpose of the tests was to assess the general load-settlement behaviour of piles of the anticipated length below the tower, and to verify the design assumptions. Each of the test piles was different, allowing various factors to be investigated, as follows:

- The effects of increasing the pile shaft length;
- The effects of shaft grouting;

Table 4. Summary of pile load tests – preliminary pile testing.

Pile No.	Diameter (m)	Length (m)	Side grouted?	Test Type
TP1	1.5	45.15	No	C
TP2	1.5	55.15	No	C
TP3	1.5	35.15	Yes	C
TP4	0.9	47.10	No	C-Cyclic
TP5	0.9	47.05	Yes	C
TP6	0.9	36.51	No	T
TP7A	0.9	37.51	No	LAT

Note: C = compression; T = tension; LAT = lateral.

- The effects of reducing the shaft diameter;
- The effects of uplift (tension) loading;
- The effects of lateral loading;
- The effect of cyclic loading.

The piles were constructed using polymer drilling fluid, rather than the more conventional bentonite drilling fluid. The use of the polymer appears to have led to piles whose performance exceeded expectations. Strain gauges were installed along each of the piles, enabling detailed evaluation of the load transfer along the pile shaft, and the assessment of the distribution of mobilized skin friction with depth along the shaft. The reaction system provided for the axial load tests consisted of four or six adjacent reaction piles (depending on the pile tested), and these reaction piles had the potential to influence the results of the pile load tests via interaction with the test pile through the soil. The possible consequences of this are discussed subsequently.

None of the 6 axial pile load tests appears to have reached its ultimate axial capacity, at least with respect to geotechnical resistance. The 1.5 m diameter piles (TP1, TP2 and TP3) were loaded to twice the working load, while the 0.9 m diameter test piles TP4 and TP6 were loaded to 3.5 times the working load, and TP5 was loaded to 4 times working load. With the exception of TP5, none of the other piles showed any strong indication of imminent geotechnical failure. Pile TP5 showed a rapid increase in settlement at the maximum load, but this was attributed to structural failure of the pile itself. From a design viewpoint, the significant finding was that, at the working load, the factor of safety against geotechnical failure appeared to be in excess of 3, thus giving a comfortable margin of safety against failure, especially as the raft would also provide additional resistance to supplement that of the piles.

From the strain gauge readings along the test piles, the mobilized skin friction distribution along each pile was evaluated. In general, in those regions in which the available shaft friction appeared to be fully mobilized (typically down to about RL – 30 m DMD), the values were in excess of those adopted for the initial design. In the lower part of the profile, the shaft friction did not appear to have been fully mobilized. Moreover, shaft grouting appeared to enhance the skin friction developed along the pile, but it was assessed that adequate shaft resistance could be developed without the need for shaft grouting.

Because the skin friction in the lower part of the ground profile did not appear to have been fully mobilized, it was recommended that the original values (termed the "theoretical ultimate unit skin friction") be used in the lower strata. It was also recommended that the "theoretical" values in the top layers (Strata 2 and 3a) be used because of the presence of the casing in the tests would probably have given skin friction values that may have been too low. For Strata 3b, 3c and 4, the minimum measured skin friction values were used for the final design.

None of the load tests was able to mobilize any significant end bearing resistance, because the skin friction appeared to be more than adequate to resist loads well in excess of the working load. Therefore, no conclusions could be reached about the accuracy of the estimated end bearing component of pile capacity. For the final design, the length of the piles was increased where the proposed pile toe levels were close to or within the gypsiferous sandstone layer (Stratum 4).

This was the case for the 0.9 m diameter podium piles. It was considered prudent to have the pile toes founded below this stratum, to allow for any potential long-term degradation of engineering properties of this layer (e.g. via solution of the gypsum) that could reduce the capacity of the piles.

8.1.9 Load-settlement behaviour

Table 5 summarizes the measured pile settlements at the working load, and the corresponding values of pile head stiffness (load/settlement). The following observations are made:

- The measured stiffness values were relatively large, and were considerably in excess of those anticipated;
- As expected, the stiffness was greater for the larger diameter piles;
- The stiffness of the shaft grouted piles (TP3 and TP5) was greater than that of the corresponding ungrouted piles.

On the basis of experience gained in the nearby Emirates Project site (Poulos & Davids, 2005), it had been expected that the pile head stiffness values for the Burj Dubai piles would be somewhat less than those for the Emirates Towers, in view of the apparently inferior quality of rock at the Burj Dubai site.

Table 5. Summary of axial pile load test results.

Pile No.	Working Load (MN)	Settlement (mm)	Stiffness (MN/m)
TP1	30.13	7.89	3819
TP2	30.13	5.55	5429
TP3	30.13	5.78	5213
TP4	10.1	4.47	2260
TP5	10.1	3.64	2775
TP6	-1.0	-0.65	1536

This expectation was not realized, and it is possible that the improved performance of the piles in the present project may be attributable, at least in part, to the use of polymer drilling fluid, rather than bentonite, in the construction process. However, it was also possible that at least part of the reason for the high stiffness values was related to the interaction effects of the reaction piles. When applying a compressive load to the test pile, the reaction piles experience a tension and a consequent uplift, which tends to reduce the settlement of the test pile. Thus, the apparent high stiffness of the pile may not reflect the true stiffness of the pile beneath the structure. The mechanisms of such interaction are discussed by Poulos (2000).

"Class A" predictions of the anticipated load-settlement behaviour were made prior to the construction of the preliminary test piles. The designer used the finite element program ABAQUS, while the independent verifier used the computer program PIES (Poulos, 1989). No allowance was made for the effects of interaction from the reaction piles. There was close agreement between the predicted curves for the 1.5 m diameter piles extending to RL-50 m, but for the 0.9 m diameter piles extending to RL-40 m, the agreement was less close, with the designer predicting a somewhat softer behaviour than the independent verifier.

The measured load-settlement behaviour was considerably stiffer than either of the predictions. This is shown in Figure 4, which compares the measured stiffness values with the predicted values, at the working load. As mentioned above, the high measured stiffness may be, at least partly, a consequence of the effects of the adjacent reaction piles. An analysis of the effects of these reaction piles on the settlement of pile TP1 revealed that the presence of the reaction piles could reduce the settlement at the working load of 30 MN by 30%. In other words, the real stiffness of the piles might be only about 70% of the values measured from the load test. This would then reduce the stiffness to a value which is more in line with the stiffness values experienced in the Emirates project, where the reaction was provided by a series of inclined anchors that would have had a very small degree of interaction with the test piles.

In the axial load tests, a relatively small number of cycles of loading was applied to the pile after the working load was reached. The settlement after cycling was related to the settlement for the first cycle, both settlements being at the maximum load of the cycling process. There was an accumulation of settlements under the action of the cyclic loading, but that this accumulation was relatively modest (10-30%), given the relatively high levels of mean and cyclic stress that were applied to the pile (in all cases, the maximum load reached was 1.5 times the working load). These results were consistent with the assessments made during design that cyclic loading effects would be unlikely to be significant for this building.

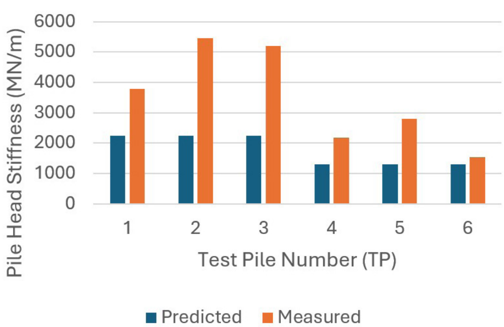

Figure 4. Measured and predicted pile head stiffness values.

8.1.10 Uplift versus compression loading

On the basis of the tension test on pile TP6, the ultimate skin friction in tension was taken as 0.5 times that for compression. It is customary to allow for a reduction in skin friction for piles in granular soils or rocks subjected to uplift. De Nicola and Randolph (1993) have developed a theoretical relationship between the tensile and compressive skin friction values, and have shown that this relationship depends on the Poisson's ratio of the pile, the relative stiffness of the pile to the soil, the interface friction characteristics and the pile length to diameter ratio. This theoretical relationship was applied to the Burj Khalifa case, and the calculated ratio of tension to compression skin friction was about 0.6, which was reasonably consistent with the initial assumption of 0.5 made in the design.

8.1.11 Lateral loading

One lateral load test was carried out, on pile TP7A, with the pile being loaded to twice the working load (50 t). At the working lateral load of 25 t, the lateral deflection was about 0.47 mm, giving a lateral stiffness of about 530 MN/m, a value which was consistent with the designer's predictions. An analysis of lateral deflection was also carried out by the independent verifier using the program DEFPIG. In this latter analysis, the Young's modulus values for lateral loading were assumed to be 30% less than the values for axial loading, while the ultimate lateral pile-soil pressure was assumed to be similar to the end bearing capacity of the pile, with allowances being made for near-surface effects. These calculations indicated a lateral movement of about 0.7 mm at 25 t load, which was larger than the measured deflection, but of a similar order.

Thus, pile TP7A appeared to perform better than anticipated under the action of lateral loading, mirroring the better-than-expected performance of the test piles under axial load. However, there may again have been some effect of the reaction system used for the test, as the reaction block developed a surface shear which would tend to oppose the lateral deflection of the test pile.

8.1.12 Works pile testing program

A total of eight works pile tests were carried, including two 1.5 m diameter piles and six 0.9 m diameter piles. All pile tests were carried out in compression, and each pile was tested approximately 4 weeks after construction. The piles were tested to a maximum load of 1.5 times the working load. The following observations were made from the test results:

- The pile head stiffness of the works piles was generally larger than for the trial piles.
- None of the works piles reached failure, and indeed, the load-settlement behaviour up to 1.5 times the working load was essentially linear, as evident from the relatively small difference in stiffness between the stiffness values at the working load and 1.5 times the working load. In contrast, the relative difference between the two stiffnesses was considerably greater for the preliminary trial piles.

At least three possible explanations could be offered for the greater stiffness and improved load-settlement performance of the trial piles:

1. The level of the bottom of the casing was higher for the works piles than for the trial piles (about 3.5-3.6 m higher), thus leading to a higher skin friction along the upper portion of the shaft;
2. A longer period between the end of construction and testing of the works piles (about 4 weeks, versus about 3 weeks for the trial piles);
3. Natural variability of the strata.

Cyclic loading was undertaken on two of the works piles, and it was observed that there was a relatively small amount of settlement accumulation due to the cyclic loading, and certainly less than that observed on TP1 or the other trial piles. The smaller amount of settlement accumulation could be attributed to the lower levels of mean and cyclic loading applied to the works piles (which were considered to be more representative of the design condition) and also to the greater capacity that the works piles seem to possess. Thus, the results of these tests reinforced the previous indications that the cyclic degradation of capacity and stiffness at the pile – soil interface appeared to be negligible.

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. The interaction effects between the test piles and the reaction piles may have contributed to the higher apparent pile head stiffnesses, but the piles nevertheless exceeded expectations. The capacity of the piles also appeared to be in excess of the predicted values, although none of the tests 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.

Shaft grouting appeared to have enhanced the load-settlement response of the piles, but it was assessed that shaft grouting would not need to be carried out for this project, given the very good performance of the ungrouted piles.

The inferences from the pile load test data were that the design estimates of capacity and settlement may be conservative, although it was recognized that the overall settlement behaviour (and perhaps the overall load capacity) would be dependent not only on the individual pile characteristics, but also on the characteristics of the ground within the zone of influence of the structure.

8.1.13 Settlement performance during construction

The settlement of the Tower raft was monitored after completion of concreting. A summary of the settlements to February 2008 in Wing C is shown on Figure 5 which also shows the final predicted settlement profile from the design.

At that time, the majority of the dead loading would have been applied to the foundation, and the maximum settlement measured was about 43 mm. It will be seen that the measured settlements are less than those predicted during the design process However, there remained some dead and live load to be applied to the foundation system, and it was noted that the monitored figures do not include the impact of the raft, cladding and live loading which would be in excess of 20% of the overall mass. Extrapolating for the full dead plus live load, it was anticipated that the final settlement would be of the order of 55-60 mm, which was comfortably less than the predicted final settlement of about 70-75 mm.

Russo et al. (2013) have carried out a careful re-assessment of the settlement analyses, taking into account such factors as the structure stiffness, the interpretation of the preliminary pile tests, and the effects of the reaction piles in the load tests. They found that the total predicted maximum settlement could then be reduced to about 52 mm.

Figure 6 shows contours of measured settlement. The general distribution is similar to that predicted by the various analyses.

To put the foundation settlements into perspective, the computed shortening of the structure after 30 years was estimated to be about 300 mm (Baker et al., 2007), which is substantially greater than the foundation settlements.

8.2 Incheon tower

A 151 storey super high-rise building project was designed for a location on reclaimed land in Songdo, Korea. The foundation system considered comprised 172 No. 2.5 m diameter bored piles, socketed into the soft rock layer and connected to a 5.5 m thick raft. This building is illustrated in Figure 7 and is described in detail by Badelow et al. (2009) and Abdelrazaq et al. (2011).

8.2.1 Ground conditions and geotechnical model

The Incheon area has extensive sand/mud flats and near-shore intertidal areas. The site lies entirely within an area of reclamation, comprises approximately 8m of loose

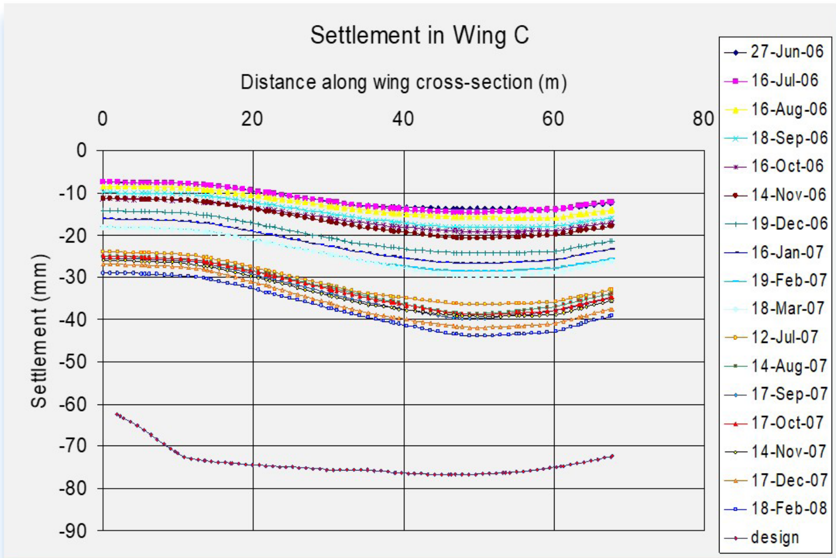

Figure 5. Measured and predicted settlement profiles along Wing C.

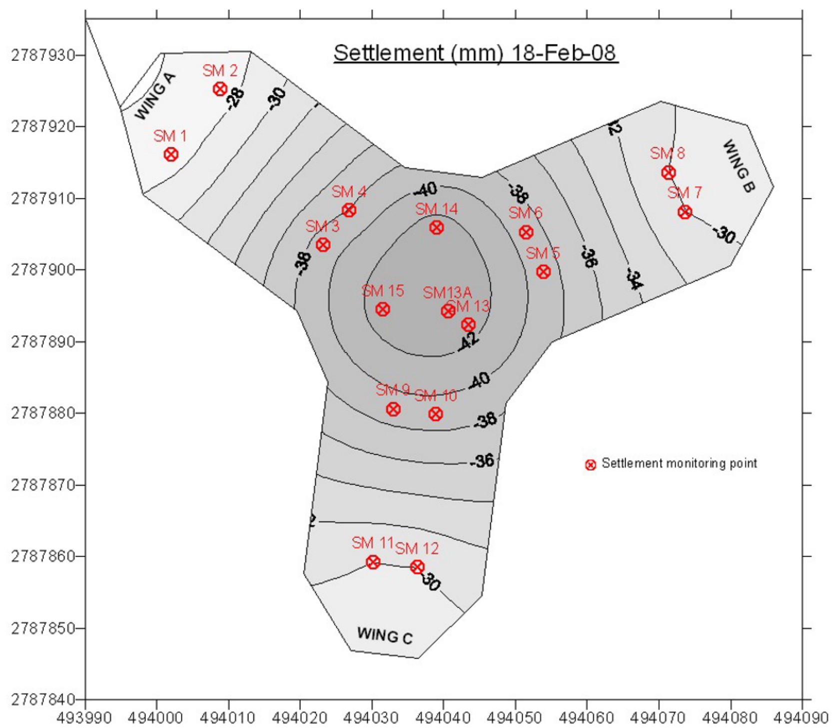

Figure 6. Contours of measured settlement as at February 2008.

sand and sandy silt, constructed over approximately 20 m of soft to firm marine silty clay, referred to as the Upper Marine Deposits (UMD). These deposits are underlain by approximately 2 m of medium dense to dense silty sand, referred to as the Lower Marine Deposits (LMD), which overlie residual soil and a profile of weathered rock.

The lithological rock units present under the site are referred to as "soft rock", and comprise granite, granodiorite, gneiss (interpreted as possible roof pendant metamorphic rocks) and aplite. The rock materials within about 50 meters from the surface have been affected by weathering which has reduced their strength to a very weak rock or a soil-like

Figure 7. Incheon 151 tower (artist's impression).

material. This depth increases where the bedrock is intersected by closely spaced joints, and sheared and crushed zones that are often related to the existence of the roof pendant sedimentary / metamorphic rocks. The geological structures at the site are complex and comprise geological boundaries, sheared and crushed seams - possibly related to faulting movements, and jointing.

From the available borehole data for the site, inferred contours were developed for the surface of the "soft rock" founding stratum within the tower foundation footprint. These are reproduced in Figure 8. It can be seen that there is a potential variation in level of the top of the soft rock (the pile founding stratum) of up to 40 m across the foundation.

The footprint of the tower was divided into eight zones which were considered to be representative of the variation of ground conditions and geotechnical models were developed for each zone. Appropriate geotechnical parameters were selected for the various strata based on the available field and laboratory test data, together with experience of similar soils on adjacent sites. One of the critical design issues for the tower foundation was the performance of the soft UMD under lateral and vertical loading, and hence careful consideration was given to the selection of parameters for this stratum. Typical parameters adopted for the initial foundation design are presented in Table 6.

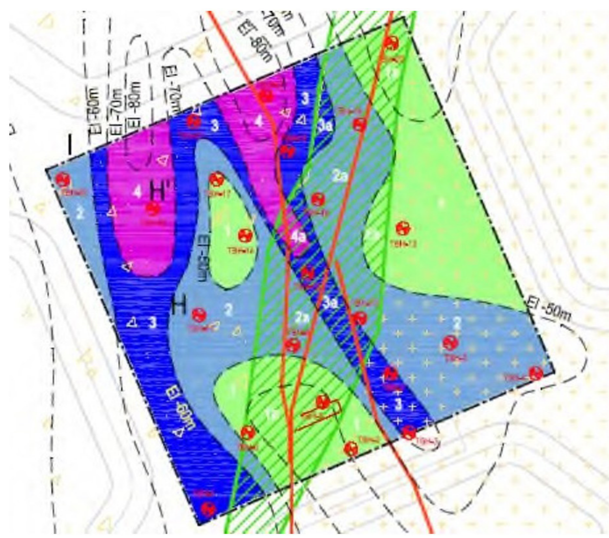

Figure 8. Inferred contours of top of soft rock – Incheon tower.

8.2.2 Foundation layout

The foundation comprises a mat and piles supporting columns and core walls. The numbers and layout of piles and the pile size were obtained from a series of trial analyses through collaboration between the geotechnical engineer and the structural designer. The pile depth was determined by the geotechnical engineer, considering the performance and capacity of piles. The pile layout was selected from the various options considered and is presented in Figure 9.

8.2.3 Loadings

Typical loads acting on the tower were as follows in Table 7. The vertical loads (*DL+LL*) and overturning moments (*Mx, My*) were represented as vertical load components at column and core locations. The load combinations, as provided by the structural designer were adopted throughout the geotechnical analysis, and 24 wind load combinations were considered.

8.2.4 Assessment of pile capacities

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

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

The pile depths required to control settlement of the tower foundation were greater than those required to provide the geotechnical capacity required. The pile design parameters for the weathered/soft rock layer are shown in Table 6 and were estimated on the basis of the pile test results in the adjacent site and the ground investigation data such as pressuremeter tests and rock core strength tests.

8.2.5 Assessment of vertical pile behavior

The vertical pile head stiffness values for each of the 172 foundation piles under serviceability loading conditions (*DL + LL*) were assessed using the in-house computer programs CLAP and GARP. CLAP was used to assess the geotechnical capacities, interaction factors and stiffness values for each pile type under serviceability loading for input into the group assessment. CLAP computed the distributions of

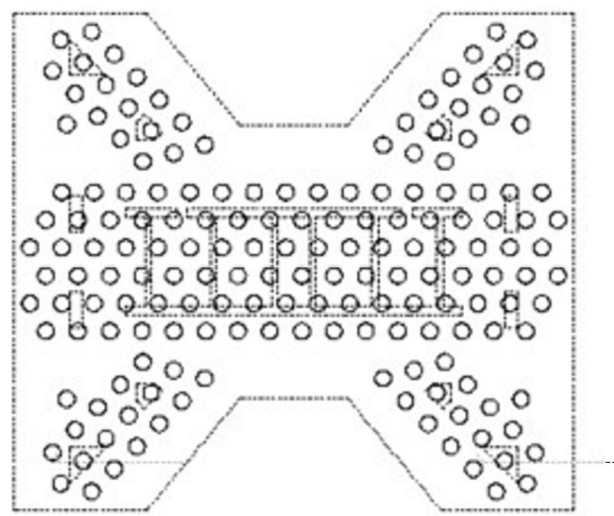

Figure 9. Pile layout plan.

Table 6. Summary of geotechnical parameters.

Strata	E_v (MPa)	E_h (MPa)	f_s (kPa)	f_b (MPa)
UMD	7-15	5-11	29-48	-
LMD	30	21	50	-
Weathered soil	60	42	75	-
Weathered rock	200	140	500	5
Soft rock (above El -50m)	300	210	750	12
Soft rock (below El -50m)	1700	1190	750	12

Note: E_v = vertical modulus; E_h = horizontal modulus; f_s = ult. shaft friction; f_b = ult. end bearing.

Table 7. Typical loads acting on the tower.

Vertical dead plus live load (MN)	Horizontal wind loads (MN)		Horizontal earthquake loads (MN)		Wind load moments (MN.m)		Wind load torsional load (MN.m)
	x axis	*y* axis	*x* axis	*y* axis	*x* axis	*y* axis	
6622	149	115	105	105	12578	21173	1957

Table 8. Summary of predicted settlements.

Design Stage	Method	Predicted Settlement (mm)	Remarks
Preliminary	Equivalent pier	75	Average settlement
Detailed	Program GARP	67	Maximum, with all 8 ground profiles
Final	PLAXIS3D	56	Maximum, with one representative profile

axial and lateral deflections, rotations and axial and lateral loads and moments, at the top of a group of piles, subjected to a combination of vertical loads, lateral loads, moments, and torsion. GARP was used to assess the group foundation behavior of the Tower.

Individual pile vertical stiffness values were computed, and it was found that the outer piles were stiffer. The analysis was non-linear, and therefore the higher stiffness values for the outer piles degraded more rapidly under loading than the central piles. The concentration of loads on outer piles within a group is a real phenomenon that has been measured in the field. Therefore, it was considered that foundation behavior can be simulated more realistically by using the individual pile stiffness values, rather than an average value for all piles within the group. Lower and upper bound estimates of pile stiffness values were provided to the structural engineers to include in their analysis, in order to capture the upper and lower bound behavior of the raft foundation and the potential impact on the tower superstructure.

8.2.6 Predicted settlements

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. Table 8 summarizes the predicted maximum settlements and indicates that the very simple equivalent pier estimate during the first stage was conservative, but of a similar order to that predicted from more refined estimates carried out during the later stages of design.

8.2.7 Assessment of lateral pile behavior

One of the critical design issues for the tower foundation was the performance of the pile group under lateral loading. Therefore, several numerical analysis programs were used in order to validate the predictions of lateral behavior obtained. The numerical modeling packages used in the analyses were:

- Computer program DEFPIG (Poulos, 1990).
- In-house computer program CLAP.
- 3D finite Element Structural Analysis Programs (Midas Set, Etabs, Safe) that included the effect of soil structure interaction.
- Finite element computer program PLAXIS 3D.

PLAXIS 3D provided an assessment of the overall lateral stiffness of the foundation. The programs DEFPIG and CLAP were used to assess the lateral stiffness provided by the pile group assuming that the raft is not in contact with the underlying soil and a separate calculation was carried out to assess the lateral stiffness of the raft and basement. Table 9 presents the computed lateral stiffness for the piled mat foundation obtained from the analyses.

Table 9. Summary of lateral stiffness of pile group and raft.

Horizontal Load (MN)	Pile Group Displ. (mm)	Lateral Pile Stiffness (MN/m)
149 (x dirn.)	17	8760
115 (y dirn,)	14	8210

8.2.8 Assessment of pile group rotational stiffness and torsional stiffness

An assessment of the rotational spring stiffness values at selected pile locations within the foundation was undertaken using an in-house computer program CLAP. To assess the rotational spring constant at each pile location, the average dead load, horizontal load (x and y direction) and moment (about the x, y and z axes) were applied to each pile head. The passive resistance of the soil surrounding the raft, and the friction between the soil and the raft, were not included in the analysis as it was assessed that the base friction of the raft footing and the passive resistance of the soil on the raft would be relatively small when compared to lateral resistance of the piles. The assessed rotational spring stiffness values obtained from the analysis for four piles considered to represent the range of values for different piles within the pile foundation, and the computed values ranged between 700 and 2680 MNm.rad.

The overall torsional stiffness of the piled mat was assessed using the computer program PLAXIS 3D. The overall torsional stiffness of the piled mat estimated using PLAXIS was 10,750,000 MNm/radian, which is approximately equivalent to 16mm displacement at the edge of the raft for the applied torsional moment of 1956MN-m applied at the centre of the raft.

8.2.9 Cyclic loading due to wind action

Wind loading for the tower structure was quite severe, therefore in order to assess the effect of low frequency cyclic wind loading, an assessment based on the method suggested in Equation 2. The factor η was selected to be 0.5, based on experience with similar projects. To assess the half amplitude of cyclic axial wind induced load, the difference in pile load between the following load cases was computed. The following cases were considered:

$$CASE\ A: 0.75(DL + LL)$$

$$CASE\ B: 0.75(DL + LL + WLx + WLy)$$

where DL is the dead load, LL is the live load, WLx is the vertical load from x-component of wind load, and WLy is the vertical load from y-component of wind load.

The difference in axial load between the two load cases was the half-amplitude of the cyclic load (S_c*). Table 10 summarizes the results of the cyclic loading assessment.

8.2.10 Pile load tests

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 10.

Table 10. Summary of cyclic loading assessment.

Quantity	Value
Maximum half-amplitude cyclic axial wind load Sc* (MN)	29.2
Maximum ratio η = Sc*/Design shaft capacity	0.43
Cyclic load criterion satisfied? ($\eta < 0.5$)	Yes

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 11. Stage 1 test was focused on the friction capacity of weathered rock and the movement of 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 150 MN 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.

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 5-6 m 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 11, which is based on the pile test analysis performed by the Load Test Corporation.

Test Pile 3 (TP3) results are not shown herein due to construction defects identified in the pile (Poulos et al., 2013); thus, the test results were ignored in obtaining the average results. While the overall performance of the test

Table 11. Summary of vertical pile test results (allowable pile bearing 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.

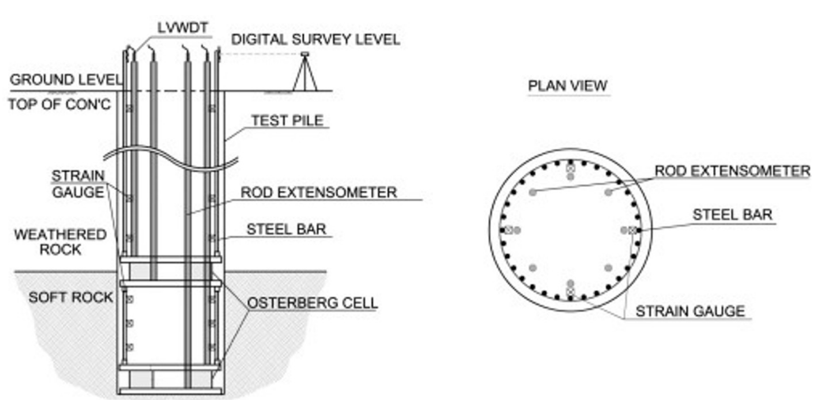

Figure 10. Schematic of monitoring for vertical pile load test.

piles exceeded expectations, Test Pile 3 highlighted that the possibility that variability in rock elevation within a short distance could affect the overall pile quality of the pile and may require careful assessment, during construction, of the pile excavation and the quality of the rock at all levels. The pile testing program also demonstrated that the foundation system could still be optimized, given the higher than anticipated shaft and base resistances that were obtained in the other four pile tests.

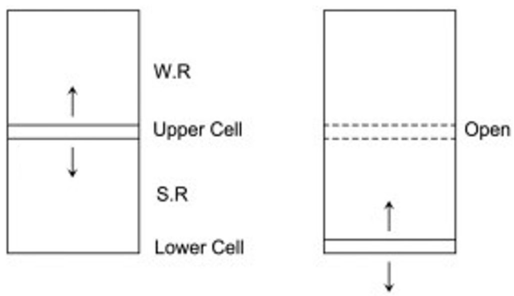

Figure 11. Typical procedure of O-cell test (Stage 1 left; Stage 2 right).

A lateral pile load test was also performed after excavation of about 8 m 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 12.

The lateral test pile was subjected to a maximum lateral load of 2.7 MN. The dynamic load-unloading test was carried out at 900 kN, 1350 kN and 1800 kN by applying 20 cycles to obtain the lateral dynamic performance of the pile, especially within the marine clay layer. The load versus pile head displacement relationship from the lateral pile test is shown in the Figure 13. 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 12. This stiffer pile behavior will be also

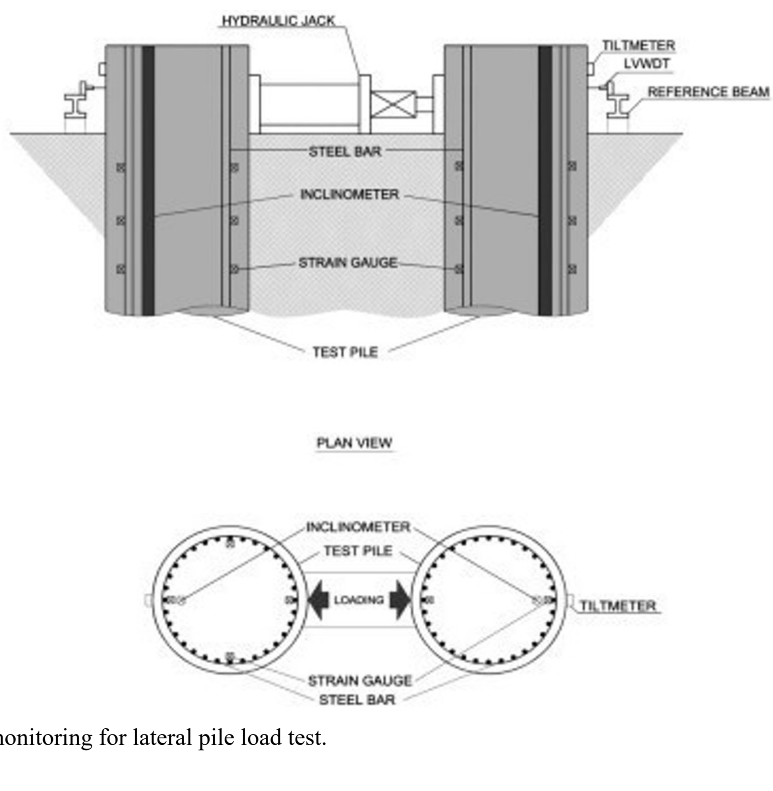

Figure 12. Schematic of monitoring for lateral pile load test.

Table 12. Lateral stiffness of the test pile.

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

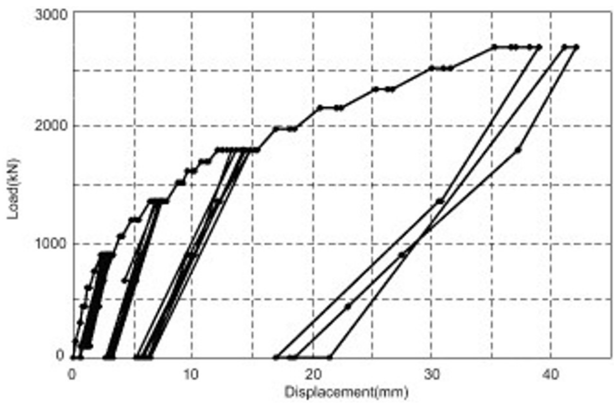

Figure 13. Load vs. displacement curve TP5.

considered in the final structural design of the tower foundation system, as well as the predicted pile group movement.

8.3 Summary

This case involved the design and testing process of a pile raft foundation system for a super high rise building to be located within the reclaimed area in Songdo, Korea. The design process involved four principal phases, namely concept design, the main design phase, the post design/study phase, and the vertical and lateral load testing programs.

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. An independent finite element analysis using readily available commercial programs was used to include the effect of soil-structure interaction and to include the impact of the foundation system on the overall behavior of the tower.

The post-design process was extended in order to obtain the actual response of the ground and the piles due to various loadings. From the results of pile load tests carried out in the post-design period, the prediction of pile behavior can be refined and the pile capacities can be updated which may result in confirmation or modification of the design, which may lead to a more cost-effective design.

An extensive high quality vertical and lateral pile testing program was developed and performed for the project and it has been shown that the pile behavior and capacities are higher than expected, so that it may be beneficial to revise some of the more conservative assumptions made in the design.

Presently the project is on hold.

9. Interaction of multiple buildings

The development of congested urban areas to accommodate burgeoning populations has led to the construction of high-rise buildings that are concentrated in relatively close proximity. This trend appears to be accelerating, and may result in the formation of what may be termed "high-rise urban forests", consisting of a group of closely-spaced buildings that are tall and slender (Cardno, 2022). It is well-recognized that wind loadings on such buildings are influenced significantly by the proximity and orientation of surrounding buildings (Blessmann & Riera, 1985), as is the seismic response (Kato & Wang, 2022), but when designing the foundations for such buildings, there has been a tendency to focus on each building as an individual isolated structure. However, there is anecdotal evidence to suggest that the foundation systems of closely-spaced tall buildings can influence each other via interaction through the soil.

Poulos (2023) developed a simplified analysis to enable a rapid estimation to be made of these inter-building interaction effects. First, an idealized axi-symmetric finite element analysis was undertaken to try and understand some of the interaction characteristics. Then, a simplified approach will be described that relied on the concept of interaction factors among foundation systems represented by equivalent piers. It was considered that this approach could provide an adequate and convenient means of assessing whether interaction effects are likely to be important, without having to do a full three-dimensional finite element analysis. It could also enable examination of the effects of progressive construction around a structure, and the evolution of differential settlements as the construction of the surrounding buildings proceeds.

In the proposed simplified analysis, the following procedure is followed:

1. The foundation system of each tower is simplified and represented as an equivalent pier within a two-layer system. The average settlement of a single pier can then be approximated via Equation 9.
2. The interaction between pairs of piers is considered to estimate the settlement of a pier due to loading on adjacent piers.
3. Superposition is applied in an approximate manner to consider the settlement distribution within a multiple high-rise development area.

The settlement interaction factor α between two piers can be computed via finite element analyses. For the case of two identical piers, it was found via curve fitting that the following approximation could be used for α (Equations 12-15):

$$\alpha = \alpha_0 . F_1 . F_2 \tag{12}$$

$$\alpha_0 = 1.681 exp\left(-1.222\, r / D\right) + 0.038 \tag{13}$$

$$F_1 = 0.835 exp\left(0.237\, L / D\right) \tag{14}$$

$$F_2 = 2.337 exp\left(-1.055\, E_b / E_s\right) + 0.718 \tag{15}$$

where r is the centre-to-centre distance between the influencing pier j and the point on pier i at which the settlement is computed, L is the pier length, D is the pier diameter, E_b is the Young's modulus of lower stratum, and E_s is the Young's modulus of upper stratum.

The settlement S_i of a point on pier i within a group of piers can then be calculated as follows in Equation 16:

$$S_i = S_0 + \sum P_j \cdot \frac{\alpha_{ij}}{K_j} \tag{16}$$

where S_0 is the settlement of pier under its own load, P_j is the load on pier j, α_{ij} is the interaction factor between the point on pier i and pier j, and K_j is the vertical stiffness of pier j (note that the summation in Equation 13 does not include the case $i = j$). For non-identical piers, it appears preferable to use the larger of the stiffness values for the two piers, together with the average interaction factors for the influencing and influenced piers.

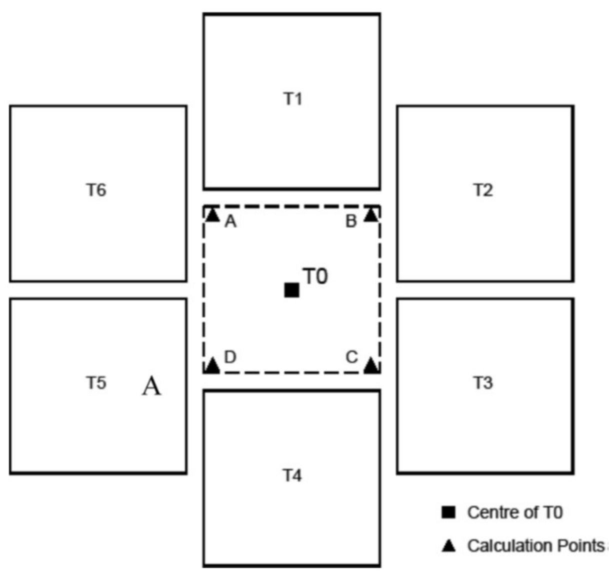

Figure 14. Configuration of tower cluster.

To illustrate the application of the proposed approach to a somewhat more realistic case, the example illustrated in Figure 14 has been considered. In this example, a cluster of 7 identical towers are to be constructed, with the central tower (T0) being constructed first, and then the remaining towers (T1 to T6) being constructed in turn. Each tower has an average service- ability loading of 0.3 MPa (equivalent to about a 30-storey building), and occupies a square footprint of 50 m by 50 m. The buildings are in close proximity, being spaced 5 m apart.

The ground conditions consist of a 20 m deep layer of medium clay with an average long-term Young's modulus of 20 MPa, overlying a 180 m deep layer of stiffer residual clay with a long-term Young's modulus of 100 MPa. The foundation system of each tower consists of a series of bored piles with a total length of 40 m, i.e. founded 20 m into the residual clay layer.

The evolution of settlement is calculated at the centre and at each of the corners of the central tower T0. Because of the simplified nature of the analysis, the settlement of T0 under its own loading is (approximately) uniform, but settlements due to the adjacent towers will be dependent on the distance between the tower centre and the point in question. Figure 15 shows the evolution of settlements of each of the 5 points (A, B, C, D and the centre) on the central tower (T0).

The following points can be seen from 15:

- Significant additional settlements are induced below T0 due to loading on the adjacent towers. The final settlement is almost 4 times the settlement of T0 under its own loading.
- The settlements below T0 are uniform at the start and finish of the loading sequence, but not at intermediate stages.
- Significant differential settlements are induced below T0 during the loading process of the adjacent towers. In this example, the largest differential settlement

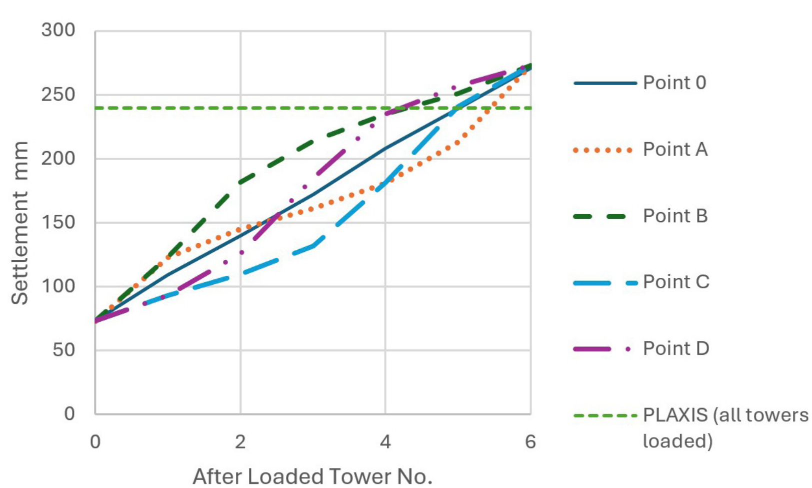

Figure 15. Evolution of computed settlements at Various points below T0.

(82 mm) occurs between points D and A after Tower T3 has been constructed and loaded, and is in excess of the initial uniform settlement of T0.
- In this case where the configuration of the towers is symmetrical, the differential settlements will eventually become zero or near-zero. However, in cases where the tower configuration is asymmetric, or where there is a marked difference between the loadings on adjacent towers, there will be a residual differential settlement of the original tower.

Clearly, the stiffer the bearing stratum on which the groups are founded, the less serious will be the settlements and differential settlements within the building cluster. Nevertheless, this simplified analysis highlights the need to consider he interactive effects of a cluster of buildings, rather than just each building on its own.

10. Conclusions

A three-phase process for the design of deep foundations has been outlined. The design issues that need to be considered have been discussed and then some of the available design tools are summarized for both the preliminary and detailed stages. The key to successful design is however related more to the appropriate assessment of geotechnical design parameters than to the specific design software adopted. In addition, it has been emphasized that simple methods should be used to check the results of complex computer analyses to ensure that the latter "make sense".

Some of the risk factors associated with foundation design have been listed, together with measures that can be employed to reduce these risks.

Two examples of foundation design for tall buildings are presented, the Burj Khalifa in Dubai and the Incheon Tower in Incheon, South Korea. Both cases have been documented previously but are useful in illustrating the various aspects of the design process.

Another issue that may be of increasing relevance to tall buildings in an urban environment are discussed, the interaction among groups of tall buildings that are close to each other. In this case, the interaction among the cluster of buildings may have a significant influence on the behaviour of the foundation system of the building in question.

With the design tools that are now available, the various aspects of tall building foundation design can be addressed satisfactorily. The main challenges that remain in relation to foundation design are the recognition and modelling of the factors that can influence deep foundation behaviour, and the ever-present challenge of appropriate assessment of the relevant geotechnical parameters.

Acknowledgements

The author is grateful to Tetra Tech Coffey for providing the resources and encouragement to prepare this paper, and to ABMS for inviting him to deliver the article at COBRAMSEG 2024 on which this paper is based.

Declaration of interest

The author has no conflicts of interest to declare.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the author, who takes full responsibility for the content of this publication.

List of symbols and abbreviations

f_b	Ultimate end bearing
f_s	Horvath et al. (1983), as presented by Burland & Mitchell (1989); Ultimate unit shaft resistance
n	Poisson's ratio of both layers
q_u	Horvath et al. (1983), as presented by Burland & Mitchell (1989); Uniaxial compressive strength
r	Centre-to-centre distance between the influencing pier j and the point on pier i at which the settlement is computed
v_s	Shear wave velocity
C	Compression
CNS	Constant normal stiffness
D	Pier diameter
DL	Dead load
DMD	Dubai Municipality Datum
E'	Drained Young's modulus
E_1	Young's modulus of upper layer
E_2	Young's modulus of lower layer
E_b	Young's modulus of lower stratum
E_c	Maximum half-amplitude of cyclic wind loading
E_d	Factored combination of loadings
E_h	Horizontal modulus
E_s	Young's modulus of upper stratum
E_u	Undrained Young's modulus
E_v	Vertical modulus
F_1	Computed value used in the approximation of the interaction factor α
F_2	Computed value used in the approximation of the interaction factor α
FEA	Finite Element Analysis

G_{max}	Small-strain shear modulus
Kj	Vertical stiffness of pier j
Kv	Vertical stiffness of the pier
Kv_1	Bordon et al. (2021) expression; Vertical stiffness of pier wholly within material within upper layer
L	Pier length
LL	Live load
LMD	Lower Marine Deposits
LRFD	Load and resistance factor design
Mx	Overturning moment about the x-axis
My	Overturning moment about the y-axis
P	Compression wave velocity
Pj	Load on pier j
R_{dg}	Design geotechnical strength
R_{ds}	Design structural strength
R_f	Hyperbolic factor
R_{gs}	Ultimate geotechnical shaft capacity
R_{ug}	Ultimate geotechnical capacity.
R_{us}	Ultimate structural strength
S	Shear wave velocity
S_0	Settlement of pier under its own load
S_{av}	Average settlement of the pier
S_c^*	Half-amplitude of the cyclic load
Si	Settlement of a point on pier i within a group of piers
SLS	Serviceability limit state
SOM	Structural designer
SPT	Standard Penetration Test
SSI	Soil-structure interaction
T	Tension
ULS	Ultimate limit state
UMD	Upper marine deposits
V	Vertical load
WLx	Vertical load from x-component of wind load
WLy	Vertical load from y-component of wind load
α	Settlement interaction factor between two piers
α_0	Computed value used in the approximation of the interaction factor α
αij	Interaction factor between the point on pier i and pier j
$\delta rigid$	Settlement of a rigid footing
η	Cyclic load ratio
θall	Allowable angular distortion
θmax	Maximum local angular distortion
ρ	Mass density of soil
ρall	Allowable foundation settlement
ρmax	Maximum computed settlement
ϕ_g	Geotechnical reduction factor
ϕ_s	Structural reduction factor

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