Tall buildings and deep foundations – Middle East challenges

Hauts bâtiments et fondations profondes – challenges du Moyen Orient

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ABSTRACT
This paper will describe a foundation design process for high-rise buildings. The process will be illustrated via its application to three high-rise building projects in Dubai, the Emirates Twin Towers, the Burj Dubai, and the Nakheel Tall Tower. The Burj Dubai is now the world's tallest building and the Nakheel Tower will become the world's tallest when completed. The foundation system for each of the towers was a piled raft, founded on deep deposits of carbonate soils and rocks. For each case, an outline will be given of the geotechnical investigations undertaken, the field and laboratory testing programs, and the design process. Of particular concern in these cases was a potential issue of low skin friction and cyclic degradation of skin friction. A numerical computer analysis that was adopted for the design process, using a limit state approach, will be described. For the Emirates project, a comprehensive program of pile load testing was undertaken and "Class A" predictions of both axial and lateral load-deflection behaviour were in fair agreement with the load test results. Despite this agreement, the overall settlements of the towers observed during construction were significantly less than those predicted. The possible reasons for the discrepancy are discussed. For the Burj Dubai, load tests were also carried out, and "Class A" predictions were made, taking account of the lessons learned from the Emirates project. The measured and predicted building settlements will be presented. For the Nakheel Tower, no performance measurements are available as it is in the early stages of foundation construction.

RÉSUMÉ
Cet article décrit le procédé de conception de foundations pour hauts bâtiments. Ce procédé est illustré au moyen de trois examples d'application à Dubai, les Emirates Twin Towers, le Burj Dubai, et le Nakheel Tower. Le Burj Dubai est à l'heure actuelle le plus haut bâtiment du monde. Le système de foundation pour chacune des tours est une foundation mixte radier-pieux établie sur de profonds dépôts de sols et de roches calcaires. Pour chaque example, les grandes lignes des recherches géotechniques entreprises, des tests de terrain et en laboratoire et du procédé de conception sont presentées. Une attention particulière a été portée pour ces deux examples sur la possibilitée d'un faible frottement latéral et d'une degradation cyclique du frottement latérale. Cet article décrit l'analyse pour ce procédé de conception et qui utilise une approche de type état critique. Pour le projet des Emirates, un programme complet de tests de charge sur pieux a été entrepris et des prédictions "class A" pour le comportement charge-déflection axial et latéral sont en accord avec les résultants, le tassement général de ces tours observé pendant le construction a été notablement moindre que prévu. Les raisons pouvant expliquer ces differences sont présentées pour le bâtiment Burj Dubai, des tests de charge ont été effectués, et des prédictions "class A" faites, en tenant comptes des résultats précédents au projet des Emirates. Le tassement du bâtiment mesuré et sa prédiction sont décrits. Pour le Nakheel Tower, les pieux sont construit maintenant.

Keywords : Dubai ; dynamic response ; foundations ; Middle East ; piled raft ; predictions ; settlement. ; stability.

1 INTRODUCTION

The last two decades have seen a remarkable increase in the rate of construction of tall buildings in excess of 150m in height. Figure 1 shows the number of such tall buildings constructed per decade (CTBUH, 2008) and reveals an almost exponential rate of growth. A significant number of these buildings have been constructed in the Middle East, and many more are either planned or already under construction. Dubai has now the tallest building in the world, the Burj Dubai, which is estimated to exceed 800m in height when completed, but another taller tower, the Nakheel Tall Tower, is currently under construction and will eventually exceed 1000m in height.

"Super-tall" buildings in excess of 300m in height are presenting new challenges to engineers, particularly in relation to structural and geotechnical design. Many of the traditional design methods cannot be applied with any confidence since they require extrapolation well beyond the realms of prior experience, and accordingly, structural and geotechnical designers are being forced to utilize more sophisticated methods of analysis and design. In particular, geotechnical engineers involved in the design of foundations for super-tall buildings are leaving behind empirical methods and are employing state-of-the art methods increasingly.

This paper will review some of the challenges that face designers of very tall buildings in the Middle East, primarily from a geotechnical viewpoint. Some characteristic features of such buildings will be reviewed and then geological, geotechnical and seismic characteristics of the Middle East will be discussed. The process of foundation design and verification will be described for two projects in Dubai, the Emirates twin towers, and the Burj Dubai. Comparisons between measured and anticipated performance will be presented and it will be demonstrated that experience gained in undertaking such comparisons can be very valuable for future projects.

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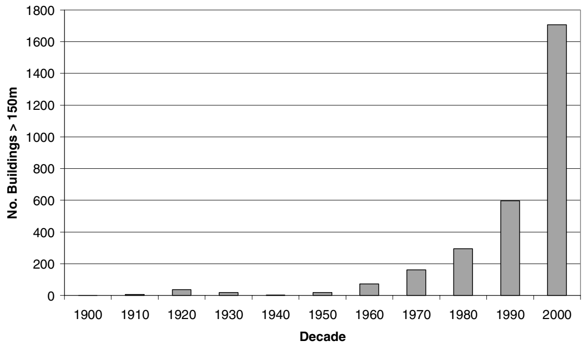


Figure 1 Number of tall building projects built per decade (CTBUH, 2008)

2 THE FOUNDATION DESIGN PROCESS

2.1 *Key Design Issues*

The following issues will generally need to be addressed in the design of foundations for high-rise buildings:

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, earthquakes and wave loadings (if appropriate) on foundation capacity and movements.
3. Overall settlements.
4. Differential settlements, both within the high-rise footprint, and between high-rise and low-rise areas.
5. Possible effects of externally-imposed ground movements on the foundation system, for example, movements arising from excavations for pile caps or adjacent facilities.
6. 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 (and, if appropriate, wave) forces.
8. Structural design of the foundation system; including the load-sharing among the various components of the system (for example, the piles and the supporting raft), and the distribution of loads within the piles. For this, and most other components of design, it is essential that there be close cooperation and interaction between the geotechnical designers and the structural designers.

This paper will address, directly or indirectly, most of the above issues, and will focus on combined pile and raft (piled raft) foundations.

2.2 *Steps in Foundation Design*

The process of foundation design is well-established, and generally involves the following aspects:

1. A desk study and a study of the geology and hydrogeology of the area in which the site is located.
2. Site investigation to assess site stratigraphy and variability.
3. In-situ testing to assess appropriate engineering properties of the key strata.
4. Laboratory testing to supplement the in-situ testing and to obtain more detailed information on the behaviour of the key strata than may be possible with in-situ testing.
5. The formulation of a geotechnical model for the site, incorporating the key strata and their engineering properties. In some cases where ground conditions are variable, a series of models may be necessary to allow proper consideration of the variability.
6. Preliminary assessment of foundation requirements, based upon a combination of experience and relatively simple methods of analysis and design. In this assessment, considerable simplification of both the geotechnical profile(s) and the structural loadings is necessary.
7. Refinement of the design, based on more accurate representations of the structural layout, the applied loadings, and the ground conditions. From this stage and beyond, close interaction with the structural designer is an important component of successful foundation design.
8. Detailed design, in conjunction with the structural designer. As the foundation system is modified, so too are the loads that are computed by the structural designer, and it is generally necessary to iterate towards a compatible set of loads and foundation deformations.
9. In-situ foundation testing at or before this stage is highly desirable, if not essential, in order to demonstrate that the actual foundation behaviour is consistent with the design assumptions. This usually takes the form of testing of prototype or near-prototype piles. If the behaviour deviates from that expected, then the foundation design may need to be revised to cater for the observed foundation behaviour. Such a revision may be either positive (a reduction in foundation requirements) or negative (an increase in foundation requirements). In making this decision, the foundation engineer must be aware that the foundation testing involves only individual elements of the foundation system, and that the piles and the raft within the system interact.
10. Monitoring of the performance of the building during and after construction. At the very least, settlements at a number of locations around the foundation should be monitored, and ideally, some of the piles and sections of the raft should also be monitored to measure the sharing of load among the foundation elements. Such monitoring is becoming more accepted as standard practice for high-rise buildings, but not always for more conventional structures. As with any application of the observational method, if the measured behaviour departs significantly from the design expectations, then a contingency plan should be implemented to address such departures. It should be pointed out that departures may involve not only settlements and differential settlements that are greater than expected, but also those that are smaller than expected.

2.3 *Basic Design Procedures And Design Criteria*

2.3.1 *Traditional Factor of Safety Approach*

In this approach, the geotechnical design criterion can be expressed as follows:

$$P_{all} = R_u / FS \qquad (1)$$

where P_{all} = allowable load (for the applied loading being considered)
R_u = ultimate load capacity (for the applied loading being considered)
FS = overall factor of safety.

In this method, the elements of uncertainty are lumped into a single factor of safety FS, which is typically between 2 and 3. Despite the many limitations of such an approach, it is still widely employed in engineering practice in many countries, and specific design values often appear in national codes or standards.

Equation (1) can also be applied to the structural design of the foundation, although the majority of structural design is now carried out using a limit state design approach.

2.3.2 *Limit State Design Approach – Ultimate State*

There is an increasing trend for limit state design principles to be adopted in foundation design, for example, in the Eurocode 7 requirements and those of the Australian Piling Code (1995). In terms of limit state design using a load and resistance factor design approach (LRFD), the design criteria for the ultimate limit state are as follows:

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

$$R_g^* \geq S^* \tag{3}$$

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

The above criteria in equations 2 and 3 are applied to the entire foundation system, while the structural strength criterion (equation 2) is also applied to each individual pile. However, it is not considered to be good practice to apply the geotechnical criterion (equation 3) to each individual pile within the group, as this can lead to considerable over-design (Poulos, 1999).

R_s^* and R_g^* can be obtained from the estimated ultimate structural and geotechnical capacities, multiplied by appropriate reduction factors. Values of the structural and geotechnical reduction factors are often specified in national codes or standards. The selection of suitable values of ϕ_g requires considerable judgement and should take into account a number of factors that may influence the foundation performance. As an example, the Australian Piling Code AS2159-1995 specifies values of ϕ_g between 0.4 and 0.9, the lower values being associated with greater levels of uncertainty and the higher values being relevant when a significant amount of load testing is carried out.

2.3.3 *Load Combinations*

The required load combinations for which the structure and foundation system have to be designed will usually be dictated by an appropriate structural loading code. In some cases, a large number of combinations may need to be considered. For example, for the Emirates Project described in this paper, a total of 18 load combinations was analyzed for each tower, these being 1 loading set for the ultimate dead and live loading only, 4 groups of 4 loading sets for various combinations of dead, live and wind loading for the ultimate limit state, and 1 set for the long-term serviceability limit state (dead plus live loading).

2.3.4 *Design for Cyclic Loading*

In addition to the normal design criteria, as expressed by equations 2 and 3, it is suggested that an additional criterion be imposed for the whole foundation of a tall building to cope with the effects of repetitive loading from wind and/or wave action, as follows:

$$\eta R_{gs}^* \geq S_c^* \tag{4}$$

where R_{gs}^* = design geotechnical shaft capacity of a pile in the group
S_c^* = maximum amplitude of wind loading
η = a factor assessed from geotechnical laboratory testing.

This criterion attempts to avoid the full mobilization of shaft friction along the piles, thus reducing the risk that cyclic loading will lead to a degradation of shaft capacity. For the Emirates project, η was selected as 0.5, based on laboratory data from laboratory constant normal stiffness (CNS) direct shear tests. S_c^* can be obtained from computer analyses which gave the cyclic component of load on each pile, for various wind loading cases.

2.3.5 *Soil-Structure Interaction Issues*

When considering soil-structure interaction for the geotechnical ultimate limit state (for example, the bending moments in the raft of a piled raft foundation system), the worst response may not occur when the pile and raft capacities are factored downwards. As a consequence, additional calculations may need to be carried out for geotechnical reduction factors both less than 1 and greater than 1. As an alternative to this duplication of analyses, it would seem reasonable to adopt a reduction factor of unity for the pile and raft resistances, and then factor up the computed moments and shears (for example, by a factor of 1.5) to allow for the geotechnical uncertainties. The structural design of the raft and the piles will also incorporate appropriate reduction factors.

2.3.6 *Serviceability Limit State*

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

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

$$\theta_{max} \leq \theta_{all} \tag{6}$$

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

Values of ρ_{all} and θ_{all} depend on the nature of the structure and the supporting soil. Table 1 sets out some suggested criteria from work reported by Zhang and Ng (2006). This table also includes values of intolerable settlements and angular distortions. The figures quoted in Table 1 are for deep foundations, but the authors also consider separately allowable settlements and angular distortions for shallow foundations, different types of structure, different soil types, and different building usage. Criteria specifically for very tall buildings do not appear to have been set, but it should be noted that it may be unrealistic to impose very stringent criteria on very tall buildings on clay deposits, as they may not be achievable. In addition, experience with tall buildings in Frankfurt Germany suggests that total settlements well in excess of 100mm can be tolerated without any apparent impairment of function.

It should also be noted that the allowable angular 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.

2.3.7 *Dynamic Loading*

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

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

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

Quantity	*Value*	*Comments*
Limiting Tolerable Settlement mm	106	Based on 52 cases of deep foundations. Std. Deviation = 55mm. Factor of safety of 1.5 recommended on this value
Observed Intolerable Settlement mm	349	Based on 52 cases of deep foundations. Std. Deviation = 218mm
Limiting Tolerable Angular Distortion rad	1/500 1/250 (H<24m) 1/330 (24<H<60m) 1/500 (60<H<100m) 1/1000 (H>100m)	Based on 57 cases of deep foundations. Std. Deviation = 1/500 rad From Chinese Code (MOC, 2002) H = building height
Observed Intolerable Angular Distortion rad	1/125	Based on 57 cases of deep foundations. Std. Deviation = 1/90 rad

It is of interest to have some idea of the acceptable levels of dynamic motion, which can be expressed in terms of dynamic amplitude of motion, or velocity or acceleration (e.g. Boggs, 1997). Table 2 reproduces guidelines for human perception levels of dynamic motion, expressed in terms of acceleration (Mendis et al, 2007). These are for vibration in the low frequency range of 0-1 Hz encountered in tall buildings, and incorporate such factors as the occupant's expectancy and experience, their activity, body posture and orientation, visual and acoustic cues. They apply to both the translational and rotational motions to which the occupant is subjected. The acceleration levels are a function of the frequency of vibration, and decrease as the frequency increases. For example, allowable vibration levels at a frequency of 1 Hz are typically only 40-50% of those acceptable at a frequency of 0.1 Hz. It is understood that, for a 10 year return period event, with a duration of 10 minutes, American practice typically allows accelerations of between 0.22 and 0.25m^2/s for office buildings, reducing to 0.10 to 0.15 m/s^2 for residential buildings.

Table 2
Human Perception Levels of Dynamic Motion (Mendis et al, 2007)

Level of Motion	*Acceleration m/s^2*	*Effect*
1	<0.05	Humans cannot perceive motion
2	0.05 - 0.1	Sensitive people can perceive motion. Objects may move slightly
3	0.1 – 0.25	Most people perceive motion. Level of motion may affect desk work. Long exposure may produce motion sickness.
4	0.25 – 0.4	Desk work difficult or impossible. Ambulation still possible.
5	0.4 – 0.5	People strongly perceive motion, and have difficulty in walking. Standing people may lose balance.
6	0.5 – 0.6	Most people cannot tolerate motion and are unable to walk naturally.
7	0.6 – 0.7	People cannot walk or tolerate motion.
8	> 0.85	Objects begin to fall and people may be injured.

2.3.8 *Design for Ground Movements*

Foundation design has traditionally focused on loads applied by the structure, but as pointed out by Poulos (2007), significant loads can also be applied to the foundation system because of ground movements. There are many sources of such movements, and the following are some sources that may be relevant to tall buildings:

1. Settlement of the ground due to site filling or dewatering. Such effects can persist for many years and may arise from activities that occurred decades ago and perhaps on sites adjacent to the present site of interest. Such vertical ground movements give rise to negative skin friction on the piles within the settling layers.
2. Heave of the ground due to excavation of the site for basement construction. Ground heave can induce tensile forces in piles located within the heaving ground. Excavation can also give rise to lateral ground movements, which can induce additional bending moments and shears in existing piles.
3. Lateral and vertical movements arising from the installation of piles near already-installed piles. These movements may induce additional axial and lateral forces and bending moment in the existing piles.
4. Dynamic ground motions arising from seismic activity. Such kinematic motions can induce additional moments and shears in the piles, in addition to the inertial forces applied by the structure to the foundation system.

Such ground movements do not reduce the geotechnical ultimate capacity of the piles, but have a two-fold influence:

- The foundations are subjected to additional movements which must be considered in relation to the serviceability requirements;

- The foundations are subjected to additional forces and moments, which must be allowed for in the structural design of the foundations.

Because the action of ground movements on piles is a soil-structure interaction problem, the most straight-forward approach to designing the piles for the additional forces and moments is to compute the best-estimate values, and then apply a factor on these computed values to obtain the design values, as suggested in the section above on soil-structure interaction.

2.4 *Design Methods and Tools*

The design process generally involves three key stages:

1. Preliminary analysis, assessment and design;
2. The main design process
3. Detailed analyses to check for complexities that may not be captured by the main design process.

The methods and tools that are employed need to be appropriate to the stage of design. Some typical design methods may include the following:

1. Preliminary analysis and design – these can make use of spreadsheets, MATHCAD sheets or simple hand or computer methods which are based on reliable but simplified methods.
2. Main design evaluation and sensitivity study – for this stage, it may be appropriate to use computer methods for pile and pile-raft analysis such as, DEFPIG (Poulos, 1980), PIGLET (Randolph, 2004), GROUP7 (Ensoft, 2007), REPUTE (Geocentrix, 2006) and NAPRA (Mandolini et al, 2005).
3. Detailed design and the final design check: Here, it may be feasible to use complex finite element and finite difference analyses, preferably three-dimensional, such as PLAXIS 3D and FLAC3D (for example, Katzenbach et al, 2000). Caution should be exercised in using two-dimensional analyses as they can often be misleading and can give settlements, differential settlements and pile loads which are inaccurate, for example, Prakoso & Kulhawy (2001).

A key element in undertaking each of the these stages of design is to try and employ geotechnical parameters that are consistent with the method being used. For example, it would be generally inappropriate to employ parameters based on a few SPT values in a three-dimensional finite element analysis carried out for the detailed design stage, although no doubt this does happen on occasions.

3 SOME PERTINENT CHARACTERISTICS OF TALL BUILDINGS

3.1 *General Characteristics*

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

1. The building weight, and thus the vertical load to be supported by the foundation, can be substantial. Moreover, the building weight increases non-linearly with height, as illustrated in Figure 2 (Moon, 2008). Thus, both ultimate bearing capacity and settlement need to be considered carefully.
2. High-rise buildings are often surrounded by low-rise podium structures which are subjected to much smaller loadings. Thus, differential settlements between the high- and low-rise portions need to be controlled.
3. The lateral forces imposed by wind loading, and the consequent moments on the foundation system, can be very high. These moments can impose increased vertical loads on the foundation, especially on the outer piles within the foundation system. The structural design of the piles needs to take account of these increased loads that act in conjunction with the lateral forces and moments.
4. The wind-induced lateral loads and moments are cyclic in nature. Thus, consideration needs to be given to the influence of cyclic vertical and lateral loading on the foundation system, as cyclic loading has the potential to degrade foundation capacity and cause increased settlements.
5. Seismic action will induce additional lateral forces in the structure and also induce lateral motions in the ground supporting the structure. Thus, additional lateral forces and moments can be induced in the foundation system via two mechanisms:
    a. Inertial forces and moments developed by the lateral excitation of the structure;
    b. Kinematic forces and moments induced in the foundation piles by the action of ground movements acting against the piles.
6. The wind-induced and seismically-induced loads are dynamic in nature, and as such, their potential to give rise to resonance within the structure needs to be assessed. The risk of dynamic resonance depends on a number of factors, including the predominant period of the dynamic loading, the natural period of the structure, and the stiffness and damping of the foundation system. Some relevant issues related to the natural period of high-rise structures are discussed below.

3.2 *Dynamic Characteristics*

The dynamic response of tall buildings poses some interesting structural and foundation design challenges. In particular, the fundamental period of vibration of a very tall structure can be very high, and conventional dynamic loading sources such as wind and earthquakes have a much lower predominant period and will generally not excite the structure via the fundamental mode of vibration. However, some of the higher modes of vibration will have significantly lower natural periods and may well be excited by wind or seismic action. These higher periods will depend primarily on the structural characteristics but may also be influenced by the foundation response characteristics. As an example, the case of a 1600m tall concrete tower will be considered. The tower is assumed to have a mass of 1.5 million tonnes, a base diameter of 120m and a top diameter of 30m. Figure 3 shows the natural frequencies computed from a finite element analysis (Irvine, 2008). The first mode has a natural period in excess of 20s, but higher modes have an increasingly small natural period, and the higher axial, lateral and torsional modes have natural frequencies of 1s or less. Such frequencies are not dissimilar to those induced by wind and seismic action.

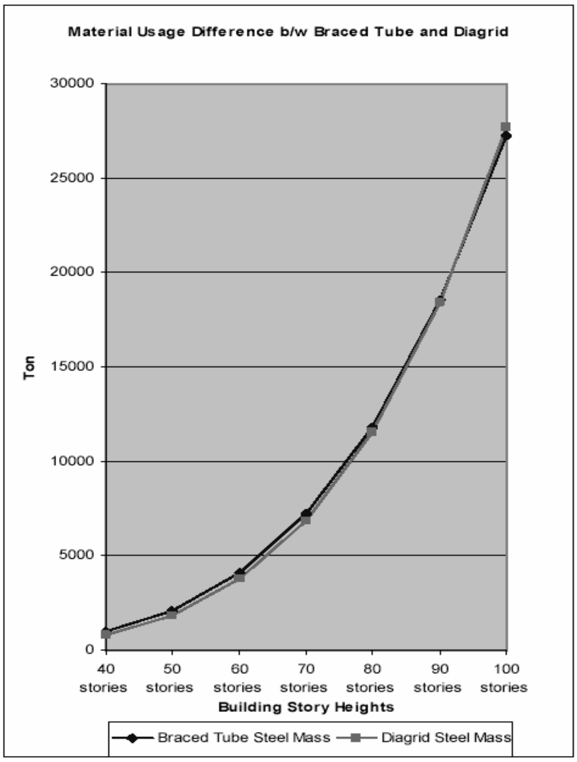


Figure 2 Example of Non-Linear Increase in Building Weight with Increasing Height (Moon, 2008)

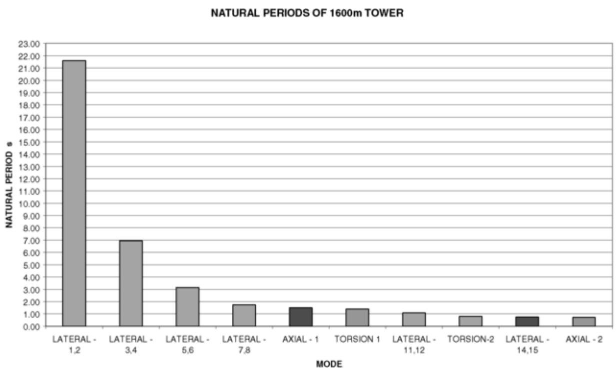


Figure 3 Natural Periods for Various Modes of Vibration

It is interesting to note that a tall building such as the one considered cannot accurately be considered as a flexural member or as a shear beam for the purposes of assessing natural frequencies. Figure 4 compares the ratio of the natural frequency to the fundamental frequency, and clearly demonstrates the substantial reduction in natural frequency for the higher modes. It also shows that the actual natural frequency lies between those for the flexural beam and the shear beam in this case.

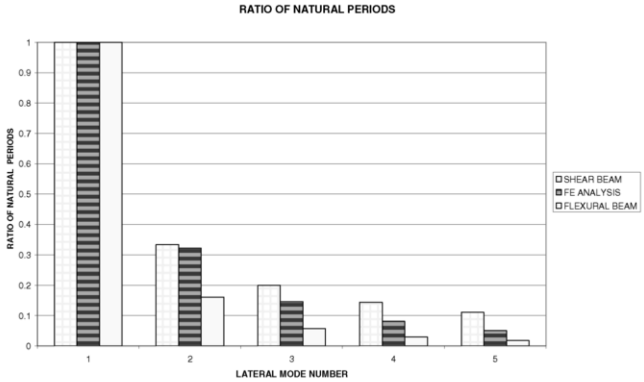


Figure 4 Ratio of Natural Period to Fundamental Period, from Various Methods

4 GEOLOGICAL AND GEOTECHNICAL FEATURES OF SOME MIDDLE EASTERN COUNTIES

4.1 *Introduction*

This section will present some of the available information on geological and geotechnical characteristics of the Arabian Peninsula (Figure 5), with particular emphasis on the Emirate of Dubai. Evans (1978) has provided a summary of the geology and the soil conditions for a number of countries in the Middle East, and some of the information below is taken from this source, although more recent published information is now available on some areas, particularly Kuwait and Saudi Arabia. The major elements of the structural geology of the Arabian Peninsula are the Arabian Shield, and the Arabian Shelf, and these, together with the interior platform and the basins, are summarized by Kent (1978). Kent provides a broad overview of the geology of the Middle East, and has identified some typical geological profiles that are reproduced in Figure 6.

4.2 *Geology of Dubai*

The geology of the Arabian Gulf area has been substantially influenced by the deposition of marine sediments resulting from a number of changes in sea level during relatively recent geological time. The area is generally relatively low-lying (with the exception of the mountainous regions in the north-east), with near-surface geology dominated by deposits of Quaternary to late Pleistocene age, including mobile Aeolian dune sands, evaporite deposits and marine sands.

The geology of the United Arab Emirates (UAE), and the Arabian Gulf Area, has been substantially influenced by the deposition of marine sediments associated with numerous sea level changes during relatively recent geological time. With the exception of mountainous regions shared with Oman in the north-east, the country is relatively low-lying, with near-surface geology dominated by deposits of Quaternary to late Pleistocene age, including mobile aeolian dune sands, sabkha/ evaporite deposits and marine sands.

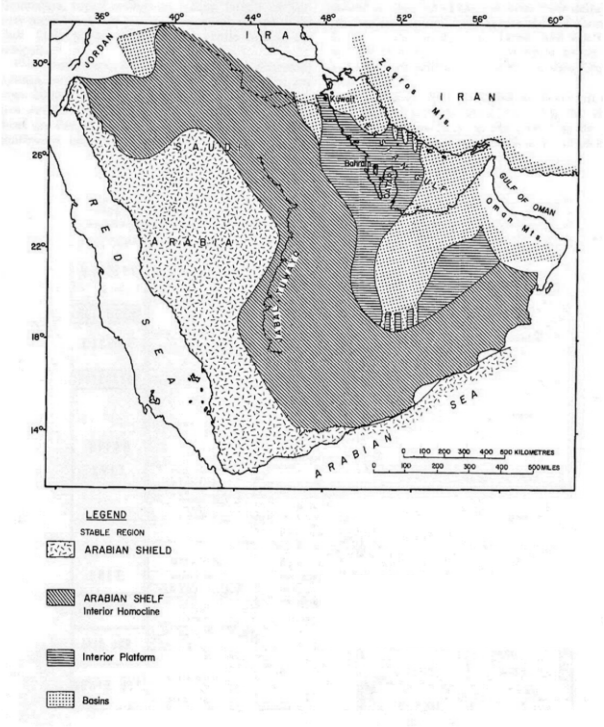


Figure 5 Summary of Structural Geology of the Arabian Peninsula (Evans, 1978)

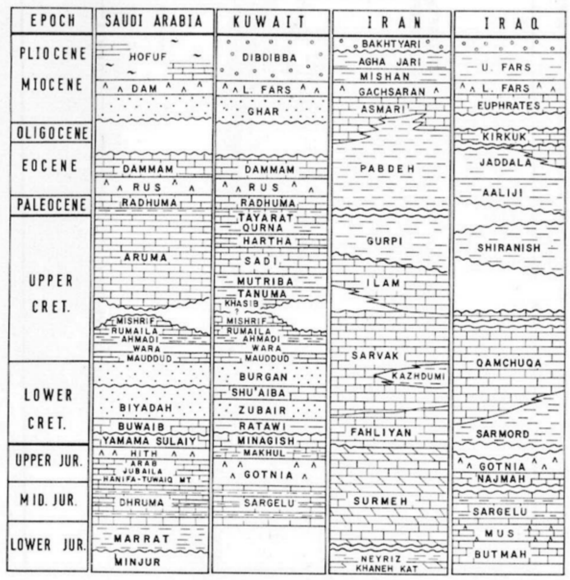


Figure 6 Some typical Middle East geological profiles (Kent, 1978).

Dubai is situated towards the eastern extremity of the geologically stable Arabian Plate and is separated from the unstable Iranian Fold Belt to the north by the Arabian Gulf. It is believed that a tilting of the entire Arabian Plate occurred during the early Permian period, resulting in uplift in southern Yemen and depression to the north east. Tectonic movements peripheral to folding of the Iranian Zagros Range during the Plio- Pleistocene epoch probably contributed to the formation of both the Arabian Gulf depression and the mountainous regions in the north east of the UAE and Oman.

4.3 *Main Stratigraphic Units in Dubai*

The main stratigraphic units encountered in Dubai are described briefly below, and a typical geotechnical profile is illustrated in Figure 7.

Marine Deposits: The Marine Deposits generally occur in two or three layers of medium dense and very loose to loose brown grey silty to very silty sand, with occasional cemented lumps and shell fragments.

Calcarenite/ Calcareous Sandstone: This stratum typically comprises weak to moderately weak fine grained Calcarenite, interbedded with cemented sand and with frequent shell fragments. The Calcarenite is generally underlain by very weak to weak, thinly to thickly laminated, grey brown, fine grained calcareous Sandstone.

Calcareous Sandstone/ Calcarenite/ Sandstone/ Sand: The stratum typically comprises very weak to weak, fine grained Calcarenite/ calcareous Sandstone/ Sandstone, interbedded with cemented sand. Bands of <1m up to approximately 5m of medium dense to very dense, cemented sand with sandstone bands may occur within this stratum.

Gypsiferous Sandstone/ Sand: This stratum typically comprises very weak to weak, fine-grained gypsiferous sandstone, interbedded with cemented sand.

Calcisiltite/ Conglomeritic Calcisiltite : This formation typically comprises very weak to weak calcisiltite (occasionally conglomeritic) and is encountered at levels ranging between –28m and –72m DMD.

Calcareous/ Conglomeritic Stratum: This unit typically comprises very weak to weak calcareous siltstone/ calcareous conglomerate/ conglomeritic sandstone/ limestone.

Claystone/ Siltstone Strata: This stratum comprises very weak to moderately weak grey claystone interbedded with reddish brown siltstone. Between levels of about –110 and –130 m DMD occasional thin bands of gypsum may be encountered. Below approximately –130m DMD the stratum may be encountered as weak to moderately weak siltstone with medium to widely spaced fractures.

The groundwater table is typically 1-3m below the ground surface.

4.4 *Foundation Design Parameters*

Alrifai (2007) presents some data on unconfined compressive strength (UCS) for relatively shallow strata, and has found that the UCS values are low, generally between 1 and 3 MPa, with a considerable scatter in the data.

There is relatively little published information on foundation design parameters for buildings in Dubai. Poulos and Davids (2005) have presented some information on pile design parameters, and these data will be considered in more detail later in the paper. Alrifai (2007) presents some data from a series of five load tests on bored piles with diameters ranging between 0.6 and 1.0m, and length between about 12 and 18m. There were 4 tests in compression and one in tension, and on the basis of these tests, Alrifai offered the following recommendations:

1. For design purposes, the ultimate skin friction values in Table 3 can be used for compression piles.
2. The ultimate skin friction for piles in tension is about 0.73 times that for compression.
3. For lateral loading, Young's modulus E_{sh} of the upper strata can be estimated from the following empirical

correlation: E_{sh} = 2.5N MPa, where N = SPT-N value.

4. Only a small amount of load is transferred to the pile base, and it was recommended that end bearing be ignored for design.

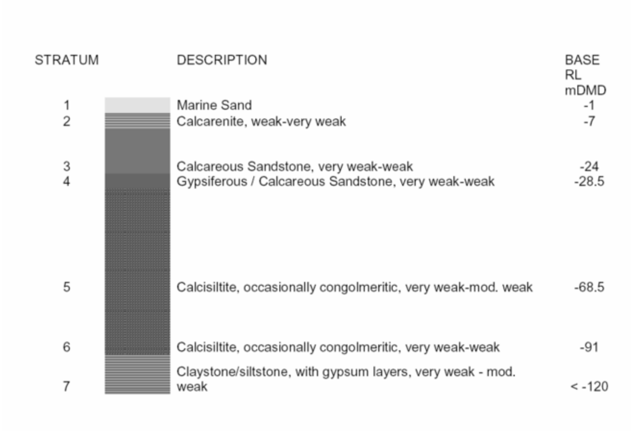


Figure 7 Typical Dubai Stratigraphy

Table 3 Summary of Recommended Ultimate Skin Friction Values for Dubai Deposits (Alrifai, 2007)

Stratum	*Elevation m DMD*	*Ultimate Skin Friction kPa*
Very dense/dense sands above rockhead (Stratum 1)	Transition zone above rockhead	100 (maximum)
Upper sandstone (Stratum 2)	Rockhead to -10	280
Conglomerate (Stratum 3/4)	-10 to -18	440

4.5 *Some important aspects of deposits in the Middle East*

It is possible to identify a number of factors which are present in Middle Eastern countries and which may be significant in designing foundations, especially for high-rise buildings. Among these factors are the following:

- Very weak rock with variable cementation. If subjected to high stresses and the cementation breaks down, these rocks may become very compressible and result in troublesome long-term settlements.
- Interbedded layers with variable properties, or deposits containing gypsum and so may be highly heterogeneous. In such cases, relatively small variations in foundation toe level may lead to considerable differences in pile performance characteristics.
- Deposits which are loose in their natural state, and rich in carbonates. They may be susceptible to degradation during cyclic loading.
- Limestone deposits with possible karstic features. The end bearing capacity of foundations in such conditions may be very small or absent, and there is also a risk that the ground support conditions may deteriorate with time if a solution cavity is formed.
- Ground conditions that do not necessarily improve with depth, at least within the feasible foundation depths. The conditions in Doha, Qatar, are an example of this phenomenon. In such cases, it may not be feasible or economical to achieve design objectives by increasing the length of the piles, and alternative strategies then need to be explored.

It is critical that such factors be identified during the ground investigation phase, and that appropriate in-situ, laboratory and field testing be undertaken to assess the extent to which such factors, if present, may influence the foundation performance.

Another issue that may be important for foundation performance relates to the chemically aggressive ground conditions that often prevail, and that may cause accelerated deterioration of foundation materials such as steel and concrete. Fookes (1978) and Fookes et al (1985) describe some of the possible consequences of such deterioration and point out that, without adequate care being taken in design and during construction, reinforced concrete in coastal areas of the Middle East may have only half the life expectancy of the same concrete in more temperate conditions.

5 SEISMICITY

Some earlier information on the seismicity of the Eastern Mediterranean and the Middle East has been summarized by Ambraseys (1978). On the basis of somewhat limited information, the following relationships were suggested for the maximum acceleration (a_{max}) and velocity (v_{max}), in terms of the earthquake magnitude M and the focal distance from the source to the site, R:

$$\log (a_{max}) = 0.46 + 0.63M - 1.10 \log(R) \qquad (7)$$

$$\log (v_{max}) = -1.36 + 0.76M - 1.22 \log(R) \qquad (8)$$

The above relationships were considered to be applicable for an earthquake magnitude M no greater than 6.

Site-specific assessments made for the Emirates towers in Dubai, carried out in 1996, indicated that the peak ground acceleration (PGA) for the horizontal component of motion was 0.072 for a 475 year return period and 0.12 g for a 2000 year return period. The corresponding PGA values for vertical components were suggested to be 0.043 and 0.073 g.

More recently, the United States Geological Survey (USGS) has published a seismic risk map which is reproduced in Figure 8. This map indicates that most of the Arabian Peninsula is relatively benign from a seismic viewpoint, but in the vicinity of Dubai, a peak bedrock acceleration of the order of 0.2g may occur with a 10% probability in 50 years, i.e. with a return period of 475 years.

Abdalla and Al-Homoud (2004) have presented the results of a seismic hazard assessment of the United Arab Emirates (UAE) based on a probabilistic approach. They have concluded that the most seismically active region in the UAE is the northern section, which includes Dubai. For this area, the PGA on bedrock was found to range between about 0.22g for a return period of 475 years to 0.38g for a return period of 1900 years. The former value is consistent with that for the area around Dubai from the USGS map in Figure 8, but they are significantly larger than the values assessed for the Emirates Towers. It would therefore appear desirable for careful site-specific studies to be made for future developments in the UAE, rather than adopting a more “broad-brush” approach for the region. It is relevant to note that there was a significant “shake” in Dubai in September 2008 that caused the evacuation of a number of high-rise buildings. The epicentre of the earthquake was in southern Iran, about 400km from Dubai, and measured 6.2 on the Richter scale.

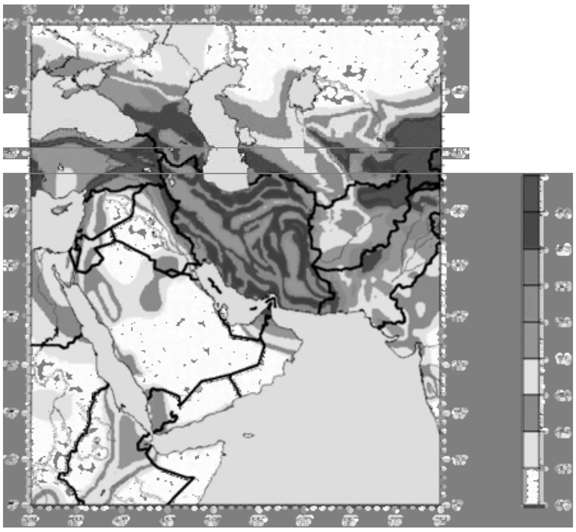


Figure 8 Peak ground acceleration (m/s^2) with 10% probability of exceedance in 50 years (USGS).

6 THE EMIRATES PROJECT

6.1 *Introduction*

The Emirates Project is a twin tower development in Dubai, one of the United Arab Emirates. The towers are triangular in plan form with a face dimension of approximately 50 m to 54 m. The taller Office Tower has 52 floors and rises 355 m above ground level, while the shorter Hotel Tower is 305 m tall. These towers are more than double the height of the nearby World Trade Centre, which was the previous tallest building in Dubai. The Office Tower when completed was the 8th tallest building in the world, while the Hotel Tower was the 17th tallest. The twin towers are located on a site of approximately 200,000 m^2, which also incorporates low level retail and parking podium areas.

As described by Davids (2008), the Office Tower was constructed using conventional slip forming techniques for the central and corner cores. A series of concrete-filled steel tube columns around the perimeter supported the edge beams. Building stability was provided by coupling the central concrete core using outrigger and belt trusses at plantroons which ran around the perimeter of the building and connected back to the core. The Hotel Tower was constructed using conventional jump-forming techniques for the central and corner cores, which supported flat plate reinforced concrete floor slabs. A cast in-situ reinforced concrete perimeter frame provided stability to the completed tower.

Figure 9 shows a photograph of the towers just after the completion of construction.

The foundation system for both towers involved the use of large diameter piles in conjunction with a raft. The geotechnical investigation undertaken for the project and the process employed for the foundation design will be described below, together with the results of a major program of pile testing. Comparisons will be made between predicted and observed test pile behaviour, and then some limited data on settlements during construction of the towers will be presented, together with the predicted values. Lessons learned from these comparisons will be summarized.

6.2 *Ground Investigation and Site Characterization*

Preliminary geotechnical data was available from earlier investigations at the site, via a series of boreholes drilled to about 15 m depth. These revealed layers of sand or silty sand, overlying very weak to weak sandstone which was in turn underlain by weak to moderately weak calcisiltite. For the twin tower development, it was clear that this preliminary information was inadequate, and hence a comprehensive additional investigation was carried out. This investigation involved the drilling of 23 boreholes, to a maximum depth of about 80m. The deepest boreholes were located below the tower footprints, while boreholes below the low-rise areas tended to be considerably shallower. Standard Penetration Tests (SPT) were carried out at nominal 1 m depths in the upper 6m of each borehole, and then at 1.5m intervals until an SPT value of 60 was achieved. SPT values generally ranged between 5 and 20 in the upper 4m, increasing to 60 at depths of 8 to 10m. Rotary coring was carried out thereafter. Core recoveries were typically 60-100% and RQD values were also between about 60 and 100%.

Figure 10 shows the borehole information along a section which passes through the two towers. It was found that the stratigraphy was relatively uniform across the whole site, so that it was considered adequate to characterize the site with a single geotechnical model. The ground surface was typically at a level of +1 to +3 m DMD, while the groundwater level was relatively close to the surface, typically between 0 m DMD and –0.6 m DMD (DMD = Dubai Municipality datum). The investigation revealed seven main strata which are summarized in Table 4, using material descriptions commonly adopted in Dubai.

Fig. 9 Emirates towers soon after completion

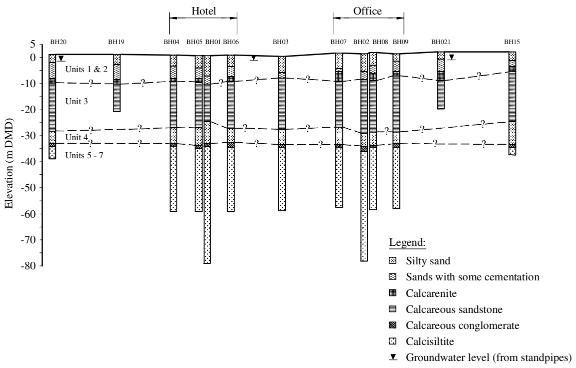


Figure 10 Geotechnical conditions at Emirates Site

6.3 *Foundation Parameter Assessment and the Geotechnical Model*

6.3.1 *In-situ and Laboratory Testing*

Because of the relatively good ground conditions near the surface, it was assessed that a piled raft system would be appropriate for the foundation of each of the towers. The design of such a foundation system requires information on both the strength and stiffness of the ground. As a consequence, a comprehensive series of in-situ tests was carried out. In addition to standard SPT tests and permeability tests, pressuremeter tests, vertical seismic shear wave testing, and site uniformity borehole seismic testing were carried out.

Conventional laboratory testing was undertaken, consisting of conventional testing, including classification tests, chemical tests, unconfined compressive strength tests, point load index tests, drained direct shear tests, and oedometer consolidation tests. In addition, a considerable amount of more advanced laboratory testing was undertaken, including stress path triaxial tests for settlement analysis of the deeper layers, constant normal stiffness direct shear tests for pile skin friction under both static and cyclic loading, resonant column testing for small-strain shear modulus and damping of the foundation materials, and undrained static and cyclic triaxial shear tests to assess the possible influence of cyclic loading on strength, and to investigate the variation of soil stiffness and damping with axial strain

.

6.3.2 *Test Results*

From the viewpoint of the foundation design, some of the relevant findings from the in-situ and laboratory testing were as follows:

- The site uniformity borehole seismic testing did not reveal any significant variations in seismic velocity, thus indicating that it was unlikely that major fracturing or voids would be present in the areas tested.
- The cemented materials were generally very weak to weak, with UCS values ranging between about 0.2 MPa and 4 MPa, with most values lying within the range of 0.5 to 1.5 MPa.
- The average angle of internal friction of the near-surface soils was about 31 degrees.
- The oedometer data for compressibility were considered to be unreliable because of the compressibility of the apparatus being of a similar order to that of some of the samples.
- The cyclic triaxial tests indicated that the Unit 4 sand deposit had the potential to generate significant excess pore pressures under cyclic loading, and to accumulate permanent deformations under repeated one-way loading. It could therefore be susceptible to earthquake-induced settlements.
- The constant normal stiffness (CNS) shear tests (Lam and Johnston, 1982) indicated that cyclic loading had the potential to significantly reduce or degrade the skin friction after initial static failure, and that a cyclic stress of 50% of the initial static resistance could cause failure during cyclic loading, resulting in a very low post-cyclic residual strength.

Table 4 Main Strata at Emirates Site

Unit No.	Designation	Material Description	Av. Elevn. of Base of Unit (m DMD)
1	Silty Sand	Uncemented calcareous silty sand; loose to medium dense	-3.3
2	Silty Sand	Variably and weakly cemented calcareous silty sand	-8.1
3	Sandstone	Calcareous sandstone, slightly to highly weathered, well cemented	-26.8
4	Silty Sand	Calcareous silty sand, variably cemented with localized well-cemented bands	-33.1
5	Calcisiltite	Variably weathered, very weakly to moderately well cemented	-53.5
6	Calcisiltite	As for Unit 5	-68.5
7	Calcisiltite	As for Unit 5	-79

Figure 11 summarizes the values of Young's modulus obtained from the following tests:

- seismic data (reduced by a factor of 0.2 to account for a strain level appropriate to the overall behaviour of the pile foundation);
- resonant column tests (at a strain level of 0.1%);
- laboratory stress path tests, designed to simulate the initial and incremental stress state along and below the foundation system;
- unconfined compression tests (at 50% of ultimate stress).

Figure 12 shows the ultimate static shear resistance derived from the CNS test data, as a function of depth below the surface. With the exception of one sample, all tests showed a maximum shear resistance of at least 500 kPa. The measured values from the CNS tests were within and beyond the range of design values of static skin friction of piles in cemented soils suggested tentatively by Poulos (1988) of between 100 and 500 kPa, depending on the degree of cementation.

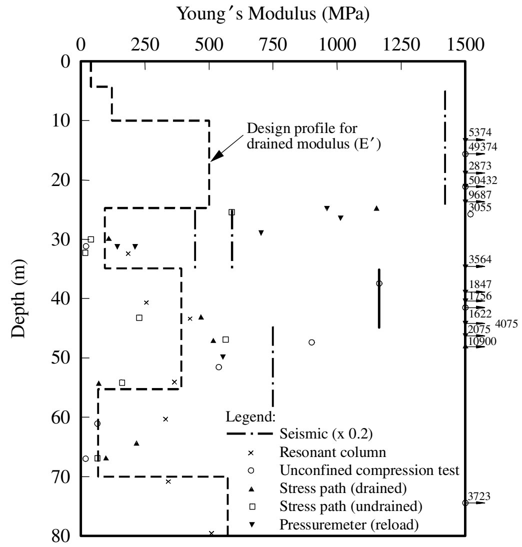


Figure 11 Summary of Young's modulus values

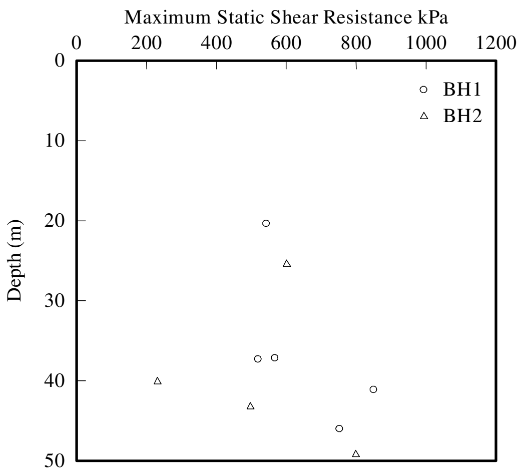


Figure 12 Ultimate skin friction values from CNS tests

6.3.3 *Geotechnical Model*

The key design parameters for the foundation system were the ultimate skin friction of the piles, the ultimate end bearing resistance of the piles, the ultimate bearing capacity of the raft, and the Young's modulus of the soils for both the raft and the pile behaviour under static loading. For the assessment of dynamic response under wind and seismic loading conditions, Young's modulus values for rapid loading conditions were also required, together with internal damping values for the various strata.

The geotechnical model for foundation design under static loading conditions was based on the relevant available in-situ and laboratory test data, and is shown in Figure 13. The ultimate skin friction values were based largely on the CNS data, while the ultimate end bearing values for the piles were assessed on the basis of correlations with UCS data (Reese and O'Neill, 1988) and also previous experience with similar cemented carbonate deposits (Poulos, 1988). The values of Young's modulus were derived from the data summarized in Figure 11. Considerable emphasis was placed on the laboratory stress path tests, which should have reflected realistic stress and strain levels within the various units. The values for the upper two units were obtained from correlations with SPT data.

The bearing capacity of the various layers for shallow foundation loading, p_u, was estimated from bearing capacity theory for the inferred friction angles, the tangent of which was reduced by a factor of 2/3 to allow for the effects of soil compressibility, as suggested by Poulos and Chua (1985).

	E_u MPa	E' MPa	ν'	f_s kPa	f_b MPa	p_u MPa	Unit
SILTY SAND, some calcarenite bands	40	30	0.2	18	0.15	0.1	1
As above	125	100	0.2	73	1.5	1.5	2
CALCAREOUS SANDSTONE	700	500	0.1	200	2.3	2.3	3
SILTY SAND	125	100	0.2	150	1.9	1.9	4
CALCISILTITE	500	400	0.2	450	2.7	2.7	5
As above	90	80	0.3	200	2.0	2.0	6
As above	700	600	0.3	450	2.7	2.7	7

Figure 13 Geotechnical model adopted for design

6.4 *Analysis Methods*

Conventional pile capacity analyses were used to assess the ultimate geotechnical capacity of the piles and raft. For the piles, this capacity was taken as the sum of the shaft and base capacities. For the raft, account was taken of the layering of the geotechnical profile, and the large size of the foundation, and a value of 2.0 MPa was adopted for the ultimate bearing capacity. In these conventional analyses, it was assumed that the portion of the raft effective in providing additional bearing capacity had a diameter of 3.6m (3 pile diameters) around each pile.

In addition to the conventional analyses, more complete analyses of the foundation system were undertaken with the computer program GARP (Poulos, 1994). GARP (Geotechnical Analysis of Raft with Piles) utilizes a simplified boundary element analysis to compute the behaviour of a rectangular piled raft when subjected to applied vertical loading, moment loading, and free-field vertical soil movements. The raft is represented by an elastic plate, the soil is modelled as a layered elastic continuum, and the piles are represented by elastic-plastic or hyperbolic springs which can interact with each other and with the raft. Pile-pile interactions are incorporated via interaction factors. Beneath the raft, limiting values of contact pressure in compression and tension can be specified, so that some allowance can be made for non-linear raft behaviour. The output of GARP includes the settlement at all nodes of the raft, the transverse, longitudinal and torsional bending moments at each node in the raft, the contact pressures below the raft, and the vertical loads in each pile. In addition to GARP, the simplified boundary element program DEFPIG (Poulos and Davis, 1980) was used to obtain the required input values of the pile stiffness and pile-pile interaction factors for GARP, and also for computing the overall lateral response of the foundation system (ignoring the effect of the raft in this case).

GARP and DEFPIG were used for the ultimate limit state, using undrained soil parameters for the wind loading cases, while drained soil parameters were used for the cases where only dead and live loading were acting. The pile and raft capacities were factored, as discussed in Section 2.3.2 above. They were also used for the serviceability limit state, but the pile and raft resistances were unfactored in this case.

6.5 *FOUNDATION DESIGN*

6.5.1 *Pile layout*

The number, depth, diameter and locations of the foundation piles were altered several times during the design process. There was close interaction between the geotechnical and structural designers in executing an iterative process of computing structural loads and foundation response. In the final design, the piles were primarily 1.2 m diameter, and extended 40 or 45 m below the base of the raft. In general, the piles were located directly below 4.5 m deep walls which spanned between the raft and the first level floor slab. These walls acted as “webs” which forced the raft and the slab to act as the flanges of a deep box structure. This deep box structure created a relatively stiff base to the tower superstructure, although the raft itself was only 1.5 m thick. Figure 14 shows the foundation layout for the hotel tower, with the piles being generally located beneath the load bearing walls. Also shown in this figure are the contours of predicted final settlement, which will be discussed later.

6.5.2 *Ultimate limit state – overall foundation*

Table 5 summarizes the maximum computed settlement and angular rotation for each tower, from the GARP analyses; for the cases where the pile and raft capacities have been reduced by a geotechnical reduction factor of 0.6. While the calculated values may not be meaningful, they do indicate that the main geotechnical design criterion in equation 2 is satisfied, in that the reduced foundation resistance clearly exceeds the worst design action effects. For both foundation systems, the average ratio of the cyclic component of load to the design shaft resistance was found to be less than 0.5, thus satisfying the requirements of the criterion in equation 3 for cyclic loading.

Table 5
Computed Maximum Settlement and Angular Rotation
Ultimate Limit State

Tower	*Max. Settlement mm*	*Max. Angular Rotation*
Office	185	1/273
Hotel	181	1/256

6.5.3 *Serviceability limit state – overall foundation*

Table 6 summarizes the computed maximum settlement and angular rotation under serviceability loading conditions, from the GARP analyses. While the computed values are relatively large, they nevertheless were considered to be tolerable and thus the foundation systems were assessed to be satisfactory from the viewpoint of serviceability.

Figure 14 shows the computed contours of settlement from the GARP analyses for the hotel tower. Similar settlement contours were developed for the office tower, which had a somewhat different pile layout. It can be observed that, for both towers, the predicted settlements showed a “dishing” pattern, with the settlements near the centre being significantly greater than those near the edge of the foundation.

Table 6
Computed Maximum Settlement and Angular Rotation
Serviceability Limit State

Tower	*Max. Settlement mm*	*Max. Angular Rotation*
Office	134	1/384
Hotel	138	1/378

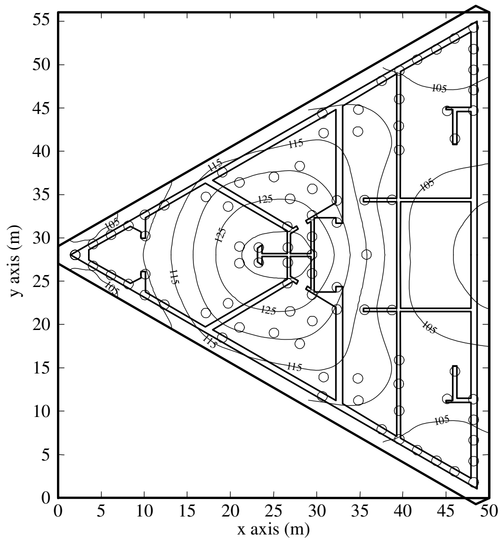


Figure 14 Computed final settlement contours for Hotel Tower

6.5.4 *Raft design*

During the design process, the effects of raft thickness were studied, but it was found that the performance of the foundation was not greatly affected by raft thickness, within the range of feasible thicknesses considered. It was therefore decided to use a raft 1.5 m thick for the final design.

Initially, GARP was used to obtain estimates of the largest bending moments and shears in the raft, for any of the combinations of ultimate limit state loadings. Subsequently, it was realized that the moments thus computed were likely to be greater than the actual moments, because no account was taken of the effects of the stiffness of the structure itself in these calculations. Therefore, for the final assessment of raft moments and shears, the computed pile stiffness for each pile in the system, and the raft contact pressures, were provided to the structural engineer who used them in a program for the complete analysis of the structure and foundation. While the settlements were generally similar to those computed from GARP, the resulting values of moment and shear from the structural analysis were significantly smaller, presumably due to the over-simplistic modeling of the raft as a uniform flat plate in the GARP analysis.

6.5.5 *Pile design*

To enable assessment of the piles from the standpoint of structural design, the maximum axial force, lateral force and bending moment in each pile were computed by the following process:

1. The maximum axial force was computed from the GARP analyses for the various loading combinations;
2. The maximum lateral shear force and bending moment were computed via the program DEFPIG, allowing for interaction effects among the piles, but ignoring any contribution of the raft to the lateral resistance. The overall

group was analyzed under the action of the various wind loadings.

It was found that the largest axial forces were developed in the piles near the corners, and in two of the core piles. A number of the piles reached their full geotechnical design resistance, but the foundation as a whole still could support the imposed ultimate design loads, and therefore satisfied the design criterion in equation 2. In this case, the structural reduction factor ϕ_s was taken as 0.6.

Combined with the moments developed by the lateral loading, the load on some of the piles fell outside the original design envelope for a 1.2 m diameter pile with 4% reinforcement, as supplied by the structural engineer. A number of options were considered to address the problem of overstressing of the piles, including increasing the reinforcement in the 1.2 m diameter piles, increasing the number of 1.2 m diameter piles in the problem areas, and increasing the diameter of the "problem" piles to 1.5 m. The second option was adopted, and for the Office Tower, the total number of piles was increased from the original 91 to 102, while for the Hotel Tower, the number of piles increased from 68 to 92.

6.5.6 *Dynamic response*

The structural design required information on the vertical and lateral stiffness of the individual piles in the two tower blocks in order to carry out a dynamic response analysis of the entire structure-foundation system. The program DEFPIG was used for the pile stiffness calculations, making the following simplifying assumptions:

- each pile carried an equal share of the vertical and lateral load;
- the loadings from the most severe case of wind loading were considered;
- the loading was very rapid so that undrained conditions prevailed in the soil profile;
- the pile heads were fixed against rotation, to simulate the effect of the restraint provided by the raft;
- the dynamic stiffness of the piles in the group environment is equal to the static stiffness.

To check the latter assumption, approximate dynamic analyses were also undertaken, using the approach outlined by Gazetas (1991), incorporating dynamic interaction factors and dynamic pile stiffnesses. It was found that, in the frequency range of interest (up to about 0.2Hz), dynamic effects on stiffness were minor, and in general, the static stiffness values provided an adequate approximation to the dynamic foundation stiffness.

The relatively low range of frequencies was assessed to be lower than the natural frequency of the soil profile, which was of the order of 0.7 to 0.8 Hz. As a consequence, little or no radiation damping could be relied upon from the piles, so that all the damping would be derived from internal damping of the soil. From the resonant column laboratory test data, the average value of internal damping ratio was found to be about 0.05. Following the recommendations of Gazetas (1991), the foundation damping ratio was taken to be 0.05 for vertical and rocking motions, and 0.04 for lateral and torsional motions.

6.5.7 *Seismic hazard assessment*

A seismic hazard assessment was carried out by a specialist consultant, and for a 500 year return period, the peak ground acceleration was assessed to be 0.075g. Assessments were then made of the potential for ground motion amplification and for liquefaction at the site. Because of the lack of detailed information on likely earthquake time histories, the potential for site amplification was estimated simply on the basis of the site geology, related to the shear wave velocity within the upper 30m of the geotechnical profile (Joyner and Fumal, 1984). On this basis, the site was assessed to have a relatively low potential for amplification.

The presence of uncemented sands near the ground surface, and below the water table, suggested that there could be a possibility of liquefaction during a strong seismic event. The grading curves for these soils indicated that they might fall into the range commonly considered to be very easily liquefied. The procedure described by Seed and de Alba (1986) was used as a basis for assessing liquefaction resistance, using SPT data. Because of the greater propensity of the calcareous sand to generate excess pore pressures under cyclic loading, a conservative approach was adopted, in which only a small amount of fines was considered, while the design earthquake magnitude was assumed to be 7.5. The overall risk of liquefaction was assessed on the basis of the liquefaction potential index defined by Iwasaki et al (1984). This index considers the factor of safety against liquefaction within the upper 20m of the soil profile. On this basis, the risk of liquefaction was judged to be low to very low, depending on the borehole considered. Consequently, there appeared to be no need to consider special measures to mitigate possible effects of liquefaction within the upper uncemented soil layers.

6.6 *Site Settlement Study*

To assist in the design of the structural interfaces between the various structures on the site to resist differential settlements, an assessment was made of the settlement over the entire site at various times after the commencement of construction. The methodology employed involved the integration of the effect of each of the towers and distributed loadings at defined points across the site. For each of these loadings, the relationship between settlement and distance was obtained. the following procedure was developed:

1. For the towers themselves, the settlements were available from the GARP analyses for the serviceability loadings.
2. The settlements of points outside the towers, due to the tower loadings, were computed using a computer program PIGS (Pile Group Settlement). This program uses a simplified approach to compute the settlements both within and outside pile groups subjected to vertical loading. PIGS employs the equations of Randolph and Wroth (1978) to compute the single pile stiffness values, while the approximate approach described in Fleming et al (1992) is used to compute pile interaction factors. The Mindlin equations are then used to compute ground settlements outside the loaded area.
3. The loads acting on the low-rise areas were modelled as a series of uniformly loaded circular areas. The computer program FLEA (Small, 1984) was used to compute the variation of surface settlement with distance from each loaded area.
4. The time rate of settlement for both the tower foundations and distributed loads was calculated on the basis of two- and three-dimensional consolidation theory, allowing for the gradual increase of load with time (Taylor, 1948). For these calculations, a coefficient of consolidation of 800 m^2 / year was assumed, based on the results of field permeability tests and the assessed Young's modulus values for the various layers. The rate of settlement of the towers was based on the solution for an equivalent isolated surface circular load, 40m in diameter, located above a compressible material 60 m deep with an impermeable base layer and a free-draining surface layer (Davis and Poulos, 1972). For the low rise areas, the solutions for the

rate of settlement of a strip foundation were used, to allow for the continuity of loading.

For both the PIGS and FLEA analyses, linear soil behaviour was assumed. A large EXCEL spreadsheet was developed to allow the summation of the effects of all 188 circular loads and two towers assumed in the model. The settlement at a total of 289 points over the site was computed for 6-monthly intervals after the commencement of construction. A typical contour plot for 24 months is shown in Figure 15.

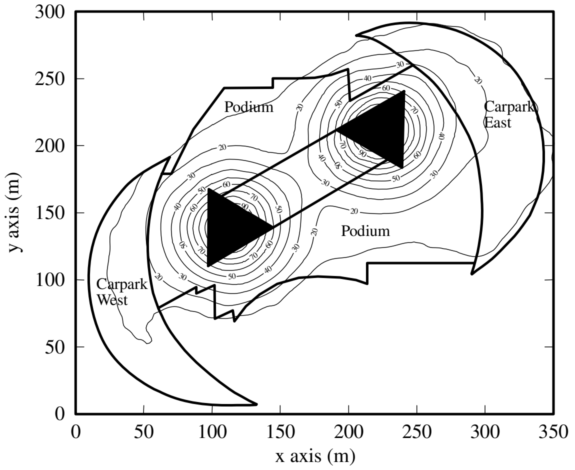


Figure 15 Computed contours of final settlement around site

6.7 *PILE LOAD TEST PROGRAM*

6.7.1 *Introduction*

As part of the foundation design process, a program of pile load testing was undertaken, the main purpose being to assess the validity of the design assumptions and parameters. The test program involved the installation of three test piles at or near the location of each of the two towers. Table 7 summarizes the tests carried out. All piles were drilled under bentonite slurry support, with steel casing being provided in the upper 3-4m of each shaft. Because of the very large design loads on the piles, it was not considered feasible to test full-size piles in compression, and as a consequence, the maximum pile diameter for the pile load tests was 0.9m. Nevertheless, it will be observed from Table 7 that the two compression tests on the 0.9 m diameter piles involved a very high maximum test load of 30 MN.

Table 7
Summary of Pile Load Tests

Tower	*Test Pile No.*	*Dia. m*	*Length m*	*Test Type*	*Max. Test Load MN*
Hotel	P3(H)	0.9	40	Comprn.	30
“	P1(H)	0.6	25	Static Tens.	6.5*
“	P2(H)	0.6	25	Cyclic Tens.	3.25**
“	P2(H)	0.6	25	Lateral	0.2
Office	P3(O)	0.9	40	Comprn.	30
“	P1(O)	0.7	25	Static Tens.	6.5*
“	P2(O)	0.7	25	Cyclic Tens.	3.25**
“	P2(O)	0.7	25	Lateral	0.2

* Initial estimated value – actual value was different
** Max. load in cyclic test = 0.5* max. load in static tension test.

6.7.2 *Test details*

Figure 16 shows the test setup for the 0.9 m diameter test piles. For the compression tests, the loading was supplied by a series of jacks, while the reaction was provided by 22 anchors drilled into the underlying Unit 4 calcisiltite. Each anchor had a total length of between 40 and 45m. The anchors were connected to the test pile via two crowns (a larger one above a smaller unit) located above the jacks and load cells. For the tension tests, the reaction was supplied by a pair of reaction piles 12 m long, with a cross-beam connecting the heads of the test and reaction piles. In the lateral load tests, the test pile was jacked against the adjacent 0.9m diameter compression test pile, the center-to-centre spacing between the piles being 4.5m.

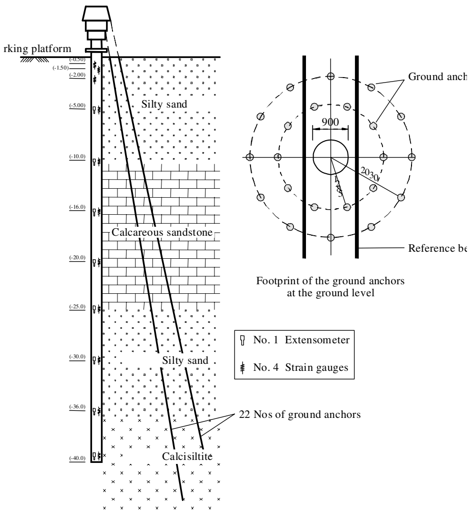


Figure 16 Setup for axial pile load tests

For piles P2(O) and P(2)H, the cyclic tension tests were carried out prior to the lateral loading test. In each of the cyclic tension tests, 4 parcels of uniform one-way cyclic load were applied.

Four main types of instrumentation were used in the test piles:

- Strain gauges (concrete embedment vibrating wire type) to allow measurement of strains along the pile shafts, and hence estimation of the axial load distribution.
- Rod extensometers, to provide additional information on axial load distribution with depth.
- Inclinometers – the piles for the lateral load tests had a pair of inclinometers, at 180 degrees, to enable measurement of rotation with depth, and hence assessment of lateral displacement with depth.
- Displacement transducers, to measure vertical and lateral displacements.

For the two P3 piles, a total of 44 strain gauges were used, 4 at each of 11 levels, while extensometers were installed at 8 levels. For the P1 and P2 piles, there were 32 strain gauges, 4 at each of 8 levels, and extensometers at 5 levels. In general, the strain gauges performed reasonably reliably. For the office piles, only 3 of the strain gauges (all on P3(O)) did not function properly, while for the hotel piles, a total of 13 strain gauges (out of a total of 108) did not function properly, 1 on P3(H), 4 on P2(H) and 8 on P1(H). The strain gauge readings were generally consistent with the extensometer readings.

6.7.3 *Class A predictions*

In order to provide some guidance on the expected behaviour of the piles during the test pile program, "Class A" predictions of the load-deflection response of the test piles were carried out and communicated to the main consultant prior to the commencement of testing. The geotechnical model was similar to that used for design, with some minor modifications to allow for the specific conditions at the test pile locations, as revealed during installation of the test piles. The following programs were used to make the predictions:

1. Static compression and tension tests – PIES (Poulos, 1989);
2. Cyclic tension test – SCARP (Poulos, 1990).
3. Lateral load test – ERCAP (CPI, 1992).

All three programs were based on simplified boundary element analyses which represented the soil as a layered continuum, and were capable of incorporating non-linear pile-soil response, and of considering the effects of the reaction piles. Young's modulus of the piles was assumed to be 30000MPa.The input geotechnical parameters for the predictions were those used for the design, as shown in Figure 13. SCARP however required additional data on cyclic degradation characteristics for skin friction and end bearing. Some indication on skin friction degradation was available from the CNS test data, but some of the parameters relating to displacement accumulation had to be assessed via judgement and previous experience with similar deposits (Poulos, 1988). It was therefore expected that the predictions for the cyclic tension test would be less accurate than for the static tests.

6.7.4 *Predicted and measured test pile behaviour*

(a) Compression Tests

Comparisons between predicted and measured test pile behaviour were made after the results of the tests were made available. Figure 17 compares the measured and predicted load-settlement curves for Test P3(H), and reveals a fair measure of agreement in the early stages. The predicted settlements however exceed the measured values, and the maximum applied load of 30 MN exceeded the estimated ultimate load capacity of about 23 MN. The corresponding comparison for the Office Tower test pile P3(O), also revealed good agreement in the early stages, but again, the predicted ultimate load capacity of 23 MN was exceeded. Indeed, it is clear from Figure 17 that the actual ultimate load capacity is likely to be well in excess of the maximum applied load of 30 MN.

The fact that the actual capacity exceeded the predicted value was significant because the values of ultimate skin friction used for the predictions were well in excess of values commonly used for bored pile design at that time in Dubai.

Figure 18 shows the measured and predicted distributions of axial load with depth, for two applied load levels. The agreement at 15 MN load is reasonable, but at 23 MN, the measured loads at depth are less than those predicted, indicating that the actual load transfer to the soil (i.e. the ultimate shaft friction) was greater than predicted.

(b) Static Tension Tests

Figure 19 compares the measured and predicted load-displacement curves for the static tension test on Pile P1(H), and indicates good agreement up to about 2 MN load. At higher loads, the actual displacement exceeded the predicted value, but the maximum applied load of 5.5 MN exceeded the predicted ultimate value of about 4.7 MN. For the Office Tower test pile, a similar measure of agreement was obtained, although the maximum load in that case was about 7.5 MN, because the test pile had a larger diameter (700mm) than the originally planned 600mm upon which the predictions were based.

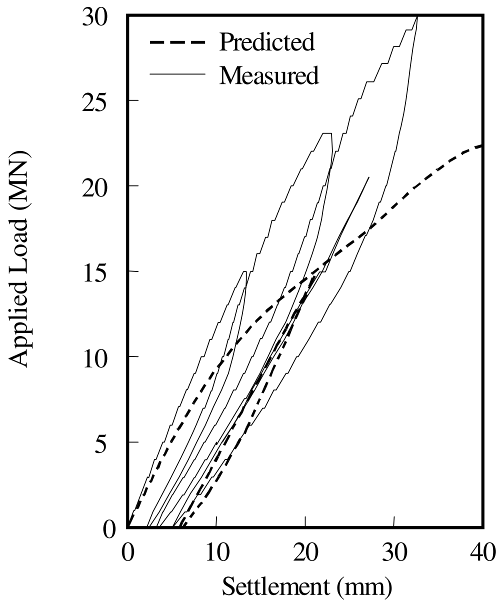


Figure 17 Predicted and measured load-settlement behaviour for Pile P3(H)

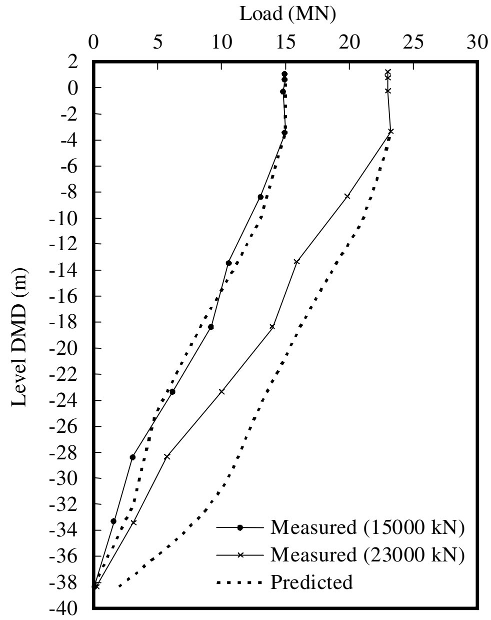
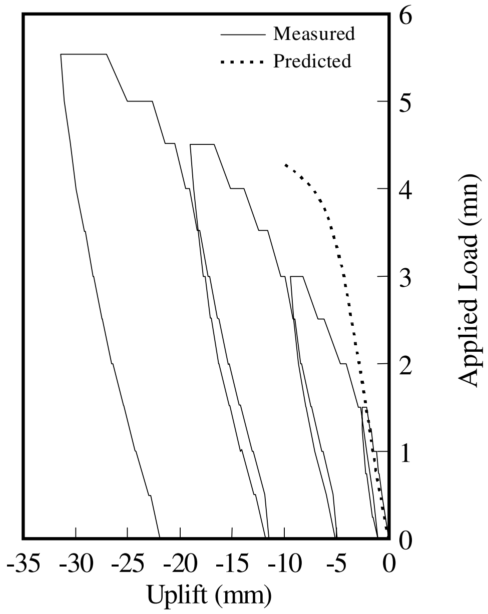


Figure 18 Predicted and measured axial load distribution for Pile P3(H)

Figure 20 shows the values of ultimate skin friction inferred from the axial load distribution measurements, for both the compression and tension tests. These values are derived for the maximum applied test loads, and are likely to be less than the actual ultimate values. Also shown are the ultimate values adopted for the design process, and these are in reasonable agreement with the measured values; indeed, the values used for design appear to be comfortably conservative. It is interesting to note that the design values were substantially larger (by about a factor of 2) than the design values commonly used in the UAE prior to the project. It appears that the CNS tests, which were



used as the primary basis for selecting the design values of skin friction, hold significant promise as a means of measuring relevant pile skin friction characteristics in the laboratory.

Figure 19 Predicted and measured load-uplift behaviour for tension test on Pile P1(H)

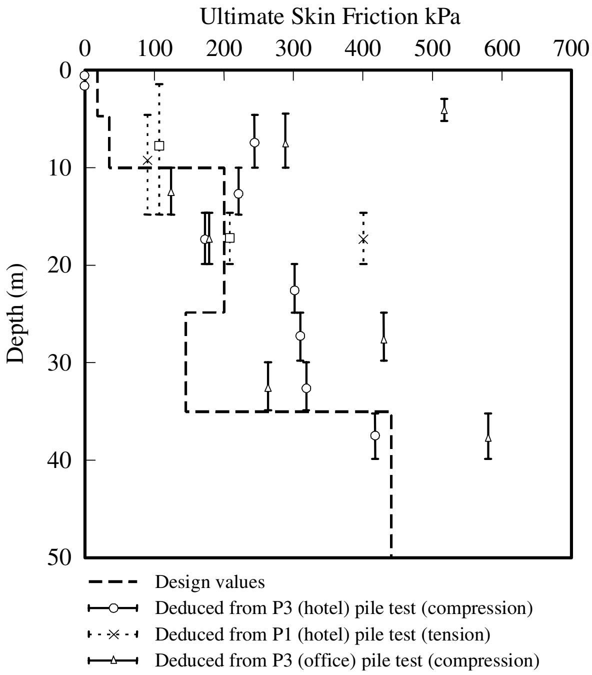
(c) Cyclic Tension Tests

Figure 21 shows the results of the cyclic tension test for the Hotel Tower pile (P2(H)). Four parcels of one-way cyclic load were applied, and for each parcel there was an accumulation of displacement with increasing number of cycles, this accumulation being more pronounced at higher load levels. The predictions from the SCARP analysis are also shown in Figure 21, and while the predictions at loads less than 1 MN are reasonable, the theory significantly under-estimates the accumulation of displacement at higher load levels. A similar (and limited) level of agreement was obtained for the test on the Office Tower test pile (P2(O)). It had been anticipated that predictions of cyclic response may not be accurate, and this expectation was borne out by the comparisons.

Clearly, the extent of displacement accumulation under cyclic loading was under-estimated in the SCARP analysis. Nevertheless, from a practical viewpoint, the important feature of the cyclic tension tests was that a load of about 50% of the static ultimate load could be applied without the pile failing (i.e. reaching an upward displacement of the order of 1-2% of diameter). However, the tests indicated that there could be a possibility of the foundation rotations under repeated wind loading being larger than predicted if the piles were to be subjected to a cyclic tension in excess of about 25% of the ultimate static uplift load capacity.

(d) Lateral Load Tests

Figure 22 shows the predicted and measured load-deflection curves for the Hotel Tower test pile. Both the test pile and the reaction pile responses are plotted. The agreement in both cases is reasonably good, although there is a tendency for the predicted deflections to be smaller than the measured values as the load level increases. A similar measure of agreement was found for the Office Tower pile, although the initial prediction had to be modified to allow for the larger as-constructed diameter of the test pile. It should be noted that the predictions took account of the interaction between the test pile and the reaction pile. Had this interaction not been taken into account, the predicted deflections would have been considerably larger than those measured.



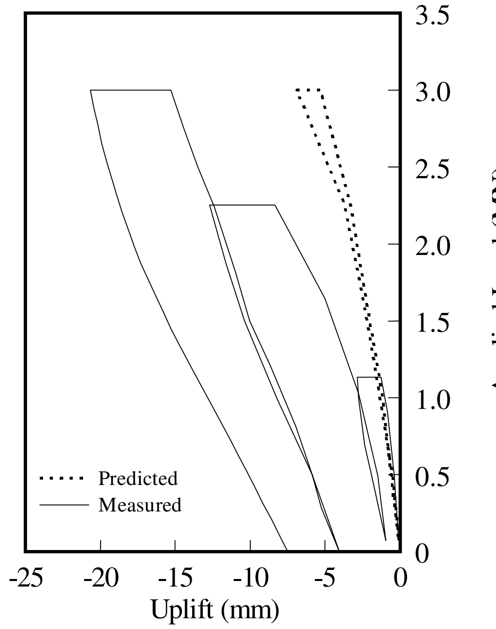


Figure 21 Measured and predicted load-uplift behaviour for cyclic uplift test- Pile P2(H)

Figure 23 shows the predicted and measured deflection profiles along the Hotel test pile, at an applied load of 150 kN. The agreement is generally good, although the measurements indicate a reversal of direction of deflection at about 3.5 m depth, a characteristic which was not predicted. This characteristic may reflect some inaccuracy in the inclinometer readings, or alternatively, the fact that the stiffness of the ground beyond about RL-4m was greater than assumed in the analysis. The sharp “kink” in the measured deflection profile may also reflect the effect of the change in stiffness due to the transition from a cased to an uncased pile.

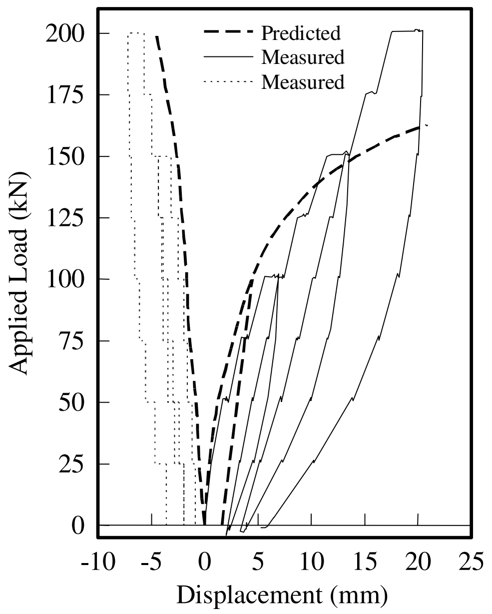


Figure 22 Measured and predicted lateral load versus deflection – Pile P2(H)

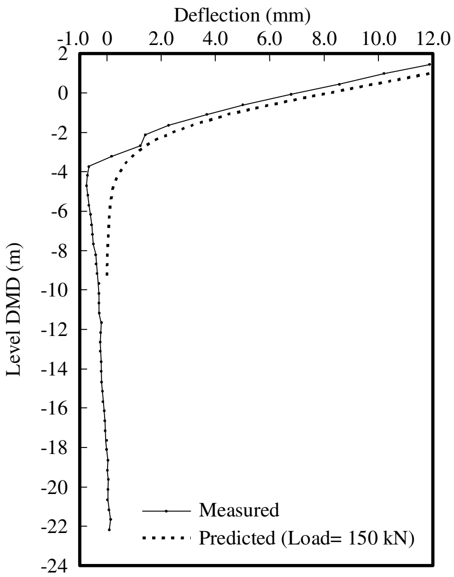


Figure 23 Measured and predicted deflection distributions – Pile P2(H)

6.8 *Measured and predicted building settlements*

6.8.1 *Comparisons during construction*

The generally good agreement between measured and predicted performance of the test piles gave rise to expectations of similar levels of agreement for the entire tower structure foundations. Unfortunately, this was not the case. Measurements were available only for a limited period during the construction process, and these are compared with the predicted time-settlement relationships in Figure 24, for two typical points within the Hotel Tower. The time-settlement predictions were based on the predicted distribution of final settlement, an assumed rate of construction, and a rate of settlement computed from three-dimensional consolidation theory. At the time of the last available measurements, the tower had reached about 70% of its final height (i.e. a height of about 215m). Figure 24 shows that the actual measured settlements were significantly smaller than those predicted, being only about 25% of the predicted values after 10-12 months. A similar level of disagreement was found for the office tower.

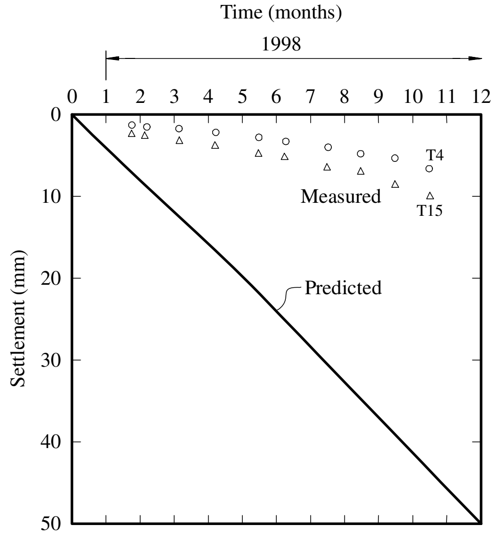


Figure 24 Measured and predicted time-settlement behaviour for Hotel Tower

Figure 25 shows the contours of measured settlement at a particular time during construction, for the hotel tower. Although the magnitude of the measured settlements is far smaller than predicted, the distribution bears some similarity to that predicted. The predicted ratio of final settlement at T4 to that at T15 is about 0.7, which is a similar order to that measured. Thus, despite the considerable thickness of the raft and the apparent stiffness of the structure, the foundation experienced a "dishing" distribution of settlement, which is similar to that measured on some other high-rise structures on piled raft foundations, particularly the Messe Turm Tower in Frankfurt, Germany (Sommer 1993; Franke et al, 1994).

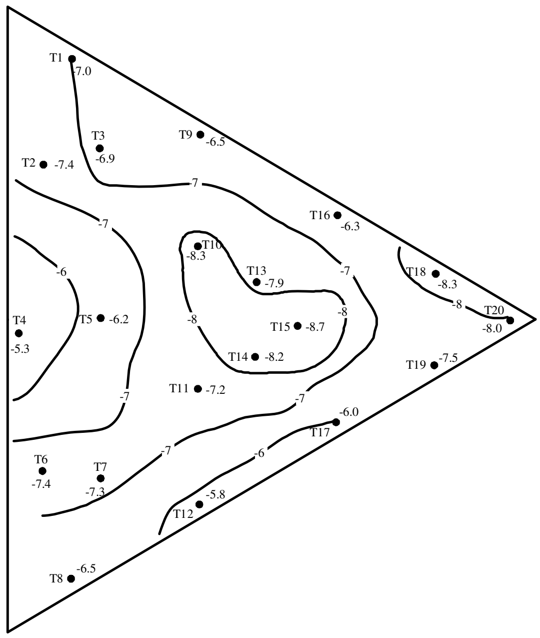


Figure 25 Measured settlement contours – Hotel Tower

6.8.2 *Possible reasons for discrepancies*

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

1. Some settlements may have occurred prior to the commencement of measurements;
2. The assumed time-load pattern may have differed from that assumed;
3. The rate of consolidation may have been much slower than predicted;
4. The interaction effects among the piles within the piled raft foundation may have been over-estimated;
5. The stiffness of the ground below RL-53 m may have been under-estimated.

Of these, based on the information available during construction, the first two did not seem to be likely, and the last two were considered to be the most likely causes. Calculations were therefore carried out to assess the sensitivity of the predicted settlements to the assumptions made in deriving interaction factors for the piled raft analysis with GARP. In using the program DEFPIG to derive the interaction factors originally used, it had been assumed that the soil or rock between the piles had the same stiffness as that around the pile, and that the rock below the pile tips had a constant stiffness for a considerable depth. In reality, the ground between the piles is likely to be stiffer than near the piles, because of the lower levels of strain, and the rock below the pile tips is also likely to increase significantly with depth, both because of the increasing level of overburden stress and the decreasing level of strain. DEFPIG was therefore used to compute the interaction factors for a series of alternative (but credible) assumptions regarding the distribution of stiffness both radially and with depth. The ratio of the soil modulus between the piles to that near the piles was increased to 5, while the modulus of the material below the pile tips was increased from the original 80 MPa to 600 MPa (the value assessed for the rock at depth). The various cases are summarized in Table 8.

Figure 26 shows the computed relationships between interaction factor and spacing for a variety of parameter assumptions. It can be seen that the original interaction curve used for the predictions lies considerably above those for what are considered (in retrospect) more realistic assumptions. Since the foundations analyzed contained many piles, the potential for over-prediction of settlements is considerable, since small inaccuracies in the interaction factors can translate to large errors in the predicted group settlement (for example, Poulos, 1993). In addition, Al-Douri and Poulos (1994) indicate that the interaction between piles in calcareous deposits may be much lower than those for a laterally and vertically homogeneous soil. Unfortunately, this experience was not incorporated in the Class A pile group settlement predictions for the towers.

Revised settlement calculations, on the basis of these interaction factors, gave the results shown in Table 8. The interaction factors used clearly have a great influence on the predicted foundation settlements, although they have almost no effect on the load sharing between the raft and the piles. The maximum settlement for Case 4 is reduced to 29% of the value originally predicted, while the minimum settlement is about 25% of the original value. If this case was used for the calculation of the settlements during construction, the settlement at Point T15 would be about 12 mm after 11 months, which is in much closer agreement with the measured value of about 10 mm than the original predictions.

The importance of proper assessment of the geotechnical model in order to compute the effects of group interaction has again been emphasized by this case history.

Curve No.	Modulus of Layer below MPa	Modulus of Soil between Piles to Near-Pile Values
1	80	1.0
2	80	5.0
3	200	5.0
4	600	5.0
5	600	1.0

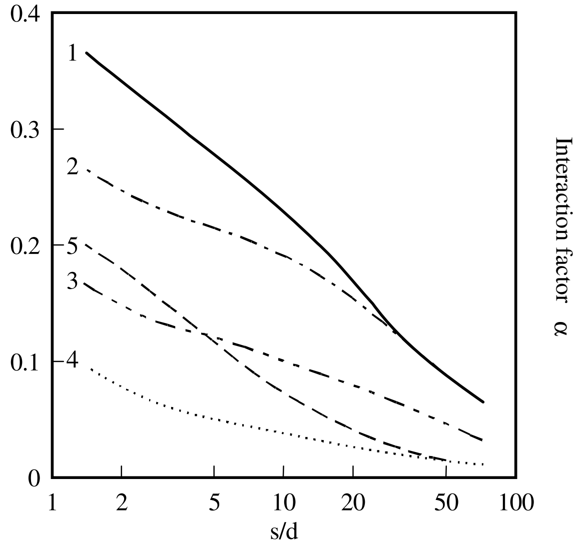


Figure 26 Sensitivity of computed interaction factors to analysis assumptions

Table 8
Summary of Revised Calculations for Hotel Tower

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

7 THE BURJ DUBAI

7.1 *Introduction*

The Burj Dubai project in Dubai comprises 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 is Emaar, a leading developer based in Dubai. Once completed, the Burj Dubai Tower will be the world's tallest building. It is founded on a 3.7m thick raft supported on bored piles, 1.5 m in diameter, extending approximately 50m below the base of the raft. Figure 27 shows an artist's impression of the completed tower. The site is generally level and site levels are related to Dubai Municipality Datum (DMD).

The Architects and Structural Engineers for the project were Skidmore Owings and Merrill LLP (SOM) in Chicago. Hyder Consulting (UK) Ltd (HCL) were appointed geotechnical consultant for the works by Emaar and carried out the design of the foundation system and an independent peer review was undertaken by Coffey Geosciences (Coffey). The process of foundation design and verification process is described below, together with the results of the pile load testing programs. The predicted settlements are then with those measured during construction.

The final height of the Burj Dubai remained a closely guarded secret, but it had reached a height of over 800m as at the February 2009. The 280,000 m^2 reinforced concrete tower will be utilized for multiple purposes, including retail, hotel, residential, and office spaces. The building is '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 (2008) describe the structural system as a "buttressed core". Each wing has its own high-performance concrete corridor walls and perimeter columns, and buttresses the others via a six-sided central core or hexagonal hub. As a consequence, the tower is very stiff laterally and torsionally. The structural aspects are described by Baker et al (2008), while Smith (2008) provides an architectural perspective of the building. The structural design has involved a three-dimensional model consisting of the reinforced concrete walls, link beams, slabs, raft and piles, together with the structural steel system. Gravity, wind and seismic loadings have been considered. According to Baker et al (2008), under lateral wind loading, the building deflections are assessed to be well below commonly used criteria. Dynamic analyses have indicated a period of 11.3 s for the first lateral mode of vibration, a period of 10.2s for the second mode, with the fifth mode (torsional motion) having a period of 4.3s.

The construction of the Burj Dubai has utilized the latest advancements in construction techniques and material technology. 80 MPa and 60 MPa concrete, with flyash, the higher strength being used for the lower portion of the structure. The walls have been formed using an automatic self-climbing formwork system, and the circular nose columns have been formed with steel forms, while the floor slabs have been poured on to special formwork. The wall reinforcement has been fabricated on the ground in 8m sections to allow for rapid placement. The central core and slabs have been cast first, in three sections: the wing walls and slabs then have followed, and after them, the wing nose and slabs have followed. Concrete has been pumped by specially designed pumps, capable of pumping to heights of 600m in a single stage. A special GPS system was developed to monitor the verticality of the structure during construction.

7.2 *GEOTECHNICAL INVESTIGATION & TESTING PROGRAM*

As described by Poulos and Bunce (2008), 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;

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 in 2 boreholes, 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.

Figure 27 Impression of Burj Dubai when Complete

The drilling was carried out using cable percussion techniques with follow-on rotary drilling methods to depths between 30m and 140m 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 57mm and 108.6mm.

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 4m to 60m below ground level typically below the Tower footprint.

The geophysical survey comprised cross-hole seismic survey, cross-hole tomography and down-hole 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 velocity with depth, and 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.

7.2.1 *Laboratory Testing*

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

1. 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.
2. 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.

Table 9 Geotechnical Model and Design Parameters for Burj Dubai Site

Strata	*Sub-Strata*	*Subsurface Material*	*Level at top of stratum (m DMD)*	*Thickness (m)*	*UCS (MPa)*	*Undrained Modulus* E_u (MPa)*	*Drained Modulus* E' (MPa)*	*Ult. Comp. Shaft Friction f_s (kPa)*
1	1a	Medium dense silty Sand	+2.50	1.50	-	34.5	30	-
	1b	Loose to very loose silty Sand	+1.00	2.20	-	11.5	10	-
2	2	Very weak to moderately weak Calcarenite	-1.20	6.10	2.0	500	400	350
3	3a	Medium dense to very dense Sand/ Silt with frequent sandstone bands	-7.30	6.20	-	50	40	250
	3b	Very weak to weak Calcareous Sandstone	-13.50	7.50	1.0	250	200	250
	3c	Very weak to weak Calcareous Sandstone	-21.00	3.00	1.0	140	110	250
4	4	Very weak to weak gypsiferous Sandstone/ calcareous Sandstone	-24.00	4.50	2.0	140	110	250
5	5a	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-28.50	21.50	1.3	310	250	285
	5b	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-50.00	18.50	1.7	405	325	325
6	6	Very weak to weak Calcareous/ Conglomerate strata	-68.50	22.50	2.5	560	450	400
7	7	Weak to moderately weak Claystone/ Siltstone interbedded with gypsum layers	-91.00	>46.79	1.7	405	325	325

Note: E_u and E' values relate to relatively large strain levels in the strata below the structure

7.3 *Geotechnical conditions*

The ground conditions comprise a horizontally stratified subsurface profile which is 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) are 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 are generally high across the site and excavations were likely to encounter groundwater at approximately +0.0m DMD (approximately 2.5m below ground level).

The ground conditions encountered in the investigation were consistent with the available geological information.

The ground profile and derived geotechnical design parameters assessed from the investigation data are summarized in Table 9. Values of Young's modulus derived by various means are plotted in Figure 28.

Non-linear stress-strain responses were derived for each strata type using the results from the SPT's, the pressuremeter, the geophysics and the standard and specialist laboratory testing. Best estimate and maximum design curves were generated and the best estimate curves are presented in Figure 29. 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 and cyclic triaxial specialist test results, and also using the computer program SHAKE91 (Idriss and 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 of the pile-soil interface. An allowance for degradation of the mass stiffness of the materials has been incorporated in the derivation of the non-linear curves in Figure 29.

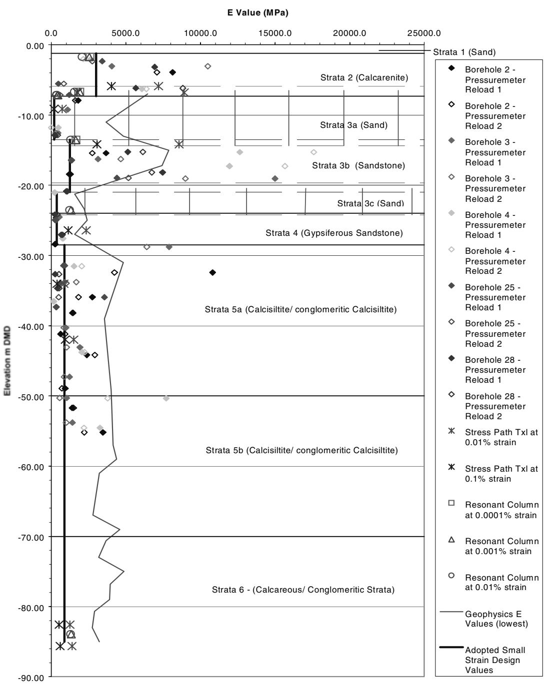


Figure 28 Modulus Values vs Elevation

7.4 *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 and Kenney (1979), as presented by Burland and Mitchell (1989):

$$\text{Ultimate unit shaft resistance } f_s = 0.25\,(q_u)^{0.5} \qquad (9)$$

where f_s is in kPa, and q_u = uniaxial compressive strength in MN/m^2

The adopted ultimate compressive unit shaft friction values for the various site rock strata are tabulated in Table 9. The ultimate unit skin friction for 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.

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 SOM and they then supplied details on the layout, number and diameter of the piles. Tower piles were 1.5m diameter and 47.45m long with the tower raft founded at -7.55mDMD. The podium piles were 0.9m diameter and 30m long with the podium raft being founded at -4.85mDMD. The thickness of the raft was 3.7m. Loading was provided by SOM and comprised 8 load cases including four load cases for wind and three for seismic conditions.

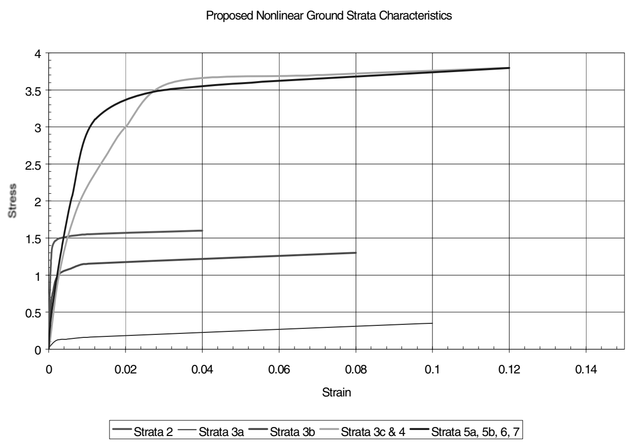


Figure 29 Non-linear Stress-strain Curves

The settlements from the FE analysis model and from VDISP have been converted from those for a flexible pile cap to those for a rigid pile cap for comparison with the REPUTE and PIGLET models using the following general equation:

$$\delta_{rigid} = 1/2\,(\delta_{\,centre} + \delta_{\,edge})_{flexible} \qquad (10)$$

The computed settlements are shown in Table 10, and the settlements from the FEA model correlated acceptably well with the results obtained from REPUTE, PIGLET and VDISP.

Table 10. Computed Settlements from Analyses

Analysis Method	*Loadcase*	*Settlement mm*	
		Rigid	*Flexible*
FEA	Tower Only (DL+LL)	56	66
REPUTE	Tower Only (DL+LL)	45	-
PIGLET	Tower Only (DL+LL)	62	-
VDISP	Tower Only (DL+LL)	46	72

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 85mm to 61mm.

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. Figure 30 shows contours of the computed maximum axial load. 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.

SOM carried out an analysis of the pile loads and a comparison on the results indicates that although the maximum pile loads are similar, the distribution is different. The SOM calculations indicated that the largest pile loads are in the central region of the Tower piled raft and decreasing towards the edges. However, the FE analyses indicated the opposite where the largest pile loads are concentrated towards the edges of the pile group reducing towards the centre of the group. Similarly, the PIGLET and REPUTE standard pile group analyses carried out indicated that the largest pile loads are concentrated towards the edge of the pile cap.

The difference between the pile load distributions could be attributed to a number of reasons including:

- The FE, REPUTE and PIGLET models take account of the pile-soil-pile interaction whereas SOM modelled the soil as springs connected to the raft and piles using an S-Frame analysis.
- The HCL FE analysis modelled the soil/rock using non-linear responses compared to the linear spring stiffnesses assumed in the SOM analysis.
- The specified/assumed superstructure stiffening effects on the foundation response were modelled more accurately in the SOM analysis.

The actual pile load distribution was expected to be somewhere between the two models, depending on the impact of the different modeling approaches.

7.5 *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 acts 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. A factor of safety of approximately 5 was obtained against overturning of the block.

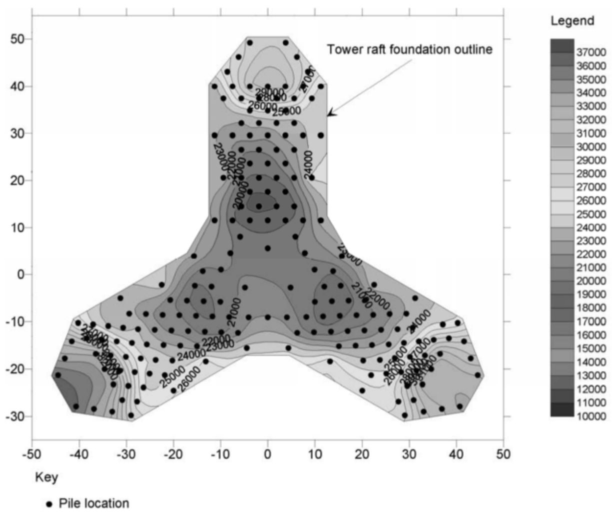


Figure 30 Contours of Maximum Axial Load (kN)

7.6 *Liquefaction assessment*

An assessment of the potential for liquefaction during a seismic event at the Burj Dubai site has been carried out using the Japanese Road Association Method and the method of Seed et al (1984). 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 3.5m 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.

7.7 *Independent verification analyses*

The geotechnical model used in the verification analyses is summarized in Table 11. The parameters were assessed independently on the basis of the available information and experience gained from the nearby Emirates project (Poulos and Davids, 2005). In general, this model was rather more conservative than the original model employed by HCL 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 –70m 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:

1. 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 shown in Table 11, 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).
2. 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 30MN. 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.
3. Finally, an analysis of the complete tower-podium foundation system was carried out using the program PIGS, and considering all 926 piles in the system. Again, each of the piles was subjected to its nominal working load.

<u>FLAC & PIGS Results For The Tower Alone</u>

Table 11. Summary of Geotechnical Model for Independent Verification Analyses

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 RL-70m 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.

<u>PIGS Results For Tower & Podium</u>

Figure 31 shows the contours of computed settlement for the entire area. It can be seen that the maximum settlements are concentrated in the central area of the tower.

Figure 32 shows the settlement profile across a section through the centre of the tower. The notable feature of this figure is that the settlements reduce rapidly outside the tower area, and become of the order of 10-12 mm for much of the podium area.

Overall, the settlements of the tower computed from the independent verification process agreed reasonably well with those obtained for the original design.

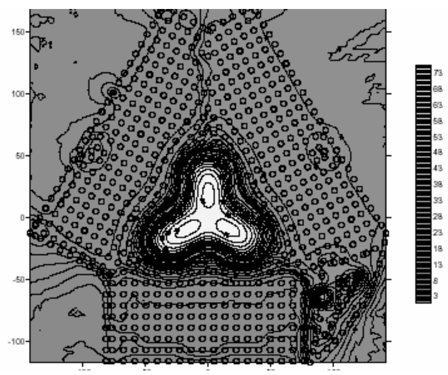


Figure 31 Computed Settlement Contours for Tower and Podium

Stratum Number	*Description*	*RL Range DMD*	*Undrained Modulus E_u MPa*	*Drained Modulus E' MPa*	*Ultimate Skin Friction kPa*	*Ultimate End Bearing MPa*
1a	Med. dense silty sand	+2.5 to +1.0	30	25	-	-
1b	Loose-very loose silty sand	_1.0 to –1.2	12.5	10	-	-
2	Weak-mod. weak calcarenite	-1.2 to –7.3	400	325	400	4.0
3	V. weak calc. Sandstone	-7.3 to -24	190	150	300	3.0
4	V. weak-weak sandstone/calc. Sandstone	-24 to –28.5	220	175	360	3.6
5A	V. weak-weak-mod. Weak calcisiltite/conglomerate.	-28.5 to –50	250	200	250	2.5
5B	V. weak-weak-mod. Weak calcisiltite/conglomerate	-50 to –70	275	225	275	2.75
6	Calcareous siltstone	-70 &below	500	400	375	3.75

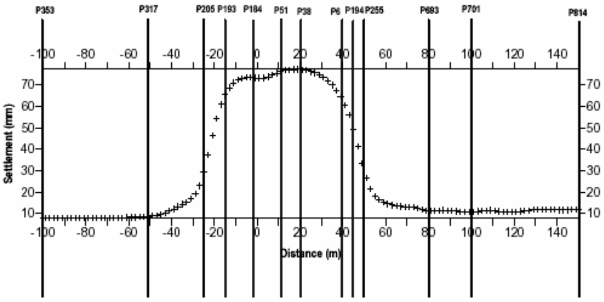


Figure 32 Computed Settlement Across Section Through Centre of Tower

7.8 *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 have 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 ± 10MN. 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 ± 10MN. 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.

7.9 *Pile load testing*

Two programs of static load testing were undertaken for the Burj Dubai 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.

7.9.1 *Preliminary Pile Testing Program*

The details of the piles tested within this program are summarized in Table 12. 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;
- 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. As will be shown below, 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.

Table 12. Summary of Pile Load Tests – Preliminary Pile Testing

Pile No.	*Pile Diam. m*	*Pile Length m*	*Side Grouted ?*	*Test Type*
TP1	1.5	45.15	No	Compression
TP2	1.5	55.15	No	Compression
TP3	1.5	35.15	Yes	Compression
TP4	0.9	47.10	No	Compression (cyclic)
TP5	0.9	47.05	Yes	Compression
TP6	0.9	36.51	No	Tension
TP7A	0.9	37.51	No	Lateral

<u>Ultimate Axial Load Capacity</u>

None of the 6 axial pile load tests appears to have reached its ultimate axial capacity, at least with respect to geotechnical resistance. The 1.5m diameter piles (TP1, TP2 and TP3) were loaded to twice the working load, while the 0.9m 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.

<u>Ultimate Shaft Friction</u>

From the strain gauge readings along the test piles, the mobilized skin friction distribution along each pile was evaluated. Figure 33 summarizes the ranges of skin friction deduced from the measurements, together with the original design assumptions and the modified design recommendations made after the preliminary test results were evaluated. The following comments can be made:

1. The skin friction values down to about RL-30m DMD appear to be ultimate values, i.e. the available skin friction has been fully mobilized.
2. The skin friction values below about RL-30m DMD do not appear to have been fully mobilized, and thus were assessed to be below the ultimate values.
3. The original assumptions appear to be comfortably conservative within the upper part of the ground profile.
4. Shaft grouting appeared to enhance the skin friction developed along the pile.

Because the skin friction in the lower part of the ground profile does 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.

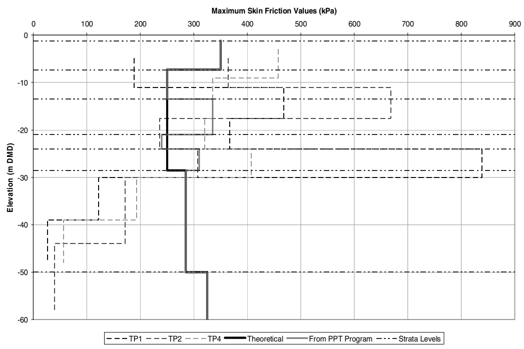


Figure 33 Measured and Design Values of Shaft Friction

<u>Ultimate End Bearing Capacity</u>

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 0.9m 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.

Load-Settlement Behaviour

Table 13 summarises the measured pile settlements at the working load and at the maximum test load, and the corresponding values of pile head stiffness (load/settlement). The following observations are made:

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

Table 13. Summary of Pile Load Test Results – Axial Loading

Pile Number	*Working Load MN*	*Max. Load MN*	*Settlement at W. Load mm*	*Settlement at Max. Load mm*	*Stiffness at W. Load MN/m*	*Stiffness at Max. Load MN/m*
TP1	30.13	60.26	7.89	21.26	3819	2834
TP2	30.13	60.26	5.55	16.85	5429	3576
TP3	30.13	60.26	5.78	20.24	5213	2977
TP4	10.1	35.07	4.47	26.62	2260	1317
TP5	10.1	40.16	3.64	27.45	2775	1463
TP6	-1.0	-3.5	-0.65	-4.88	1536	717

Effect of Reaction Piles

On the basis of the experience gained in the nearby Emirates Project site (Poulos and 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.

This expectation was certainly 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 is related to the interaction effects of the reaction piles. When applying a compressive load to the test pile, the reaction piles will experience a tension and a consequent uplift, which will tend 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).

Pile Axial Stiffness Predictions

"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 1.5m diameter piles extending to RL-50m, but for the 0.9m diameter piles extending to RL-40m, 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. Figure 34 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 30MN 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.

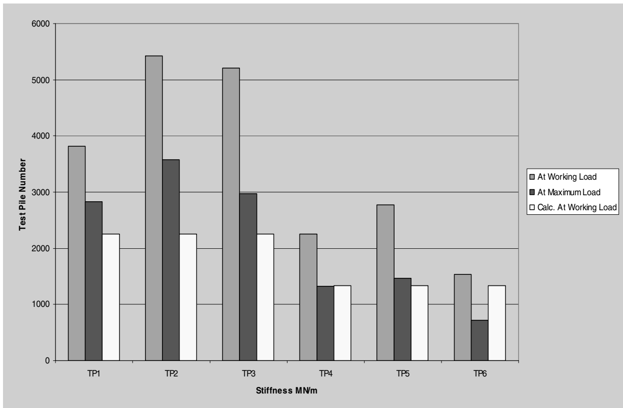


Figure 34 Measured and Predicted Pile Head Stiffness Values

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 Dubai case, and the calculated ratio of tension to compression skin friction was about 0.6, which was reasonably consistent with the assumption of 0.5 made in the design.

Cyclic Loading Effects

In all of the axial load tests, a relatively small number of cycles of loading was applied to the pile after the working load was reached. Table 14 summarizes the test results inferred from the load-settlement data. The settlement after cycling was related to the settlement for the first cycle, both settlements being at the maximum load of the cycling process. It can be seen that there is an accumulation of settlements under the action of the cyclic loading, but that this accumulation is relatively modest, given the relatively high levels of mean and cyclic stress that have been applied to the pile (in all cases, the maximum load reached is 1.5 times the working load).

These results are consistent with the assessments made during design that cyclic loading effects would be unlikely to be significant for this building.

Table 14 Summary of Displacement Accumulation for Cyclic Loading

Pile Number	*Mean Load/P_w*	*Cyclic Load/P_w*	*No. of Cycles (N)*	*S_N/S_1*
TP1	1.0	±0.5	6	1.12
TP2	1.0	±0.5	6	1.25
TP3	1.0	±0.5	6	1.25
TP4	1.25	±0.25	9	1.25
TP5	1.25	±0.25	6	1.3
TP6	1.0	±0.5	6	1.1

Note: Pw = working load; SN = settlement after N cycles; S1=settlement after 1 cycle

Lateral Loading

One lateral load test was carried out, on pile TP7A, with the pile being loaded to twice the working load (50t). At the working lateral load of 25t, the lateral deflection was about 0.47mm, giving a lateral stiffness of about 530 MN/m, a value which was consistent with the designer's predictions using the program ALP (Oasys, 2001). 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 0.7mm at 25t load, which is 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 will develop a surface shear which will tend to oppose the lateral deflection of the test pile.

7.9.2 *Works Pile Testing Program*

A total of eight works pile tests were carried out, including two 1.5m diameter piles and six 0.9m 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 (see Table 14). 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.

7.9.3 *Summary*

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 recognised 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.

7.10 *Settlement performance during construction*

The settlement of the Tower raft has been monitored since completion of concreting. The stress conditions within the raft have been determined with the placement of strain rosettes at the top and base of the raft. In addition three pressure cells have been placed at the base of the raft and five piles have been strain gauged to determine the load distribution between and down the piles. This paper presents only the measured settlements.

A summary of the settlements to February 2008 in Wing C is shown on Figure 35 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 43mm. It will be seen that the measured settlements are less than those predicted during the design process. However, there remains some dead and live load to be applied to the foundation system, and it should also be noted that the monitored figures do not include the impact of the raft, cladding and live loading which will total in excess of 20% of the overall mass. Extrapolating for the full dead plus live load, it might be anticipated that the final settlement will be of the order of 55-60mm, which is comfortably less than the predicted final settlement of about 70-75mm.

Figure 36 shows contours of measured settlement. The general distribution is similar to that predicted by the various analyses.

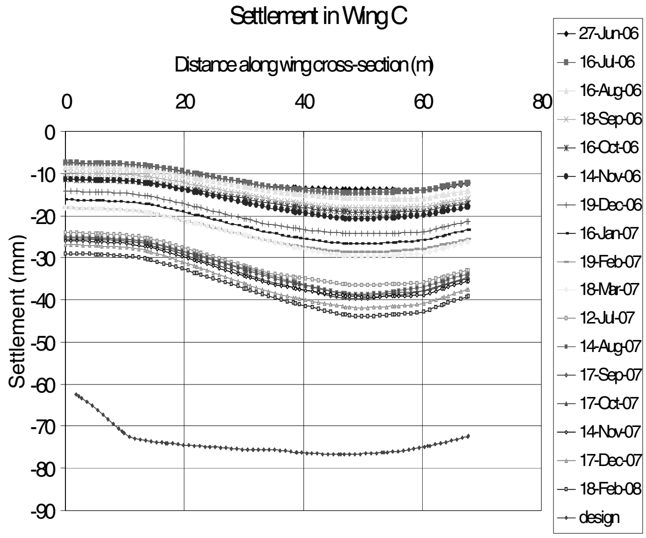


Figure 35 Measured and Predicted Settlements for Wing C, Burj Dubai

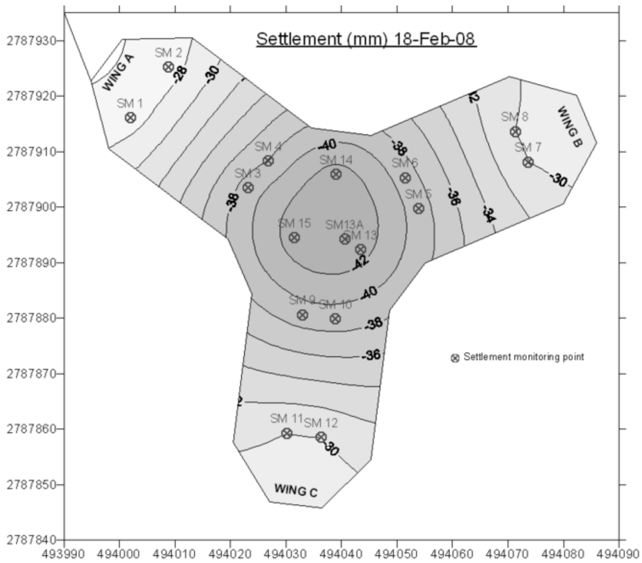


Figure 36 Contours of Measured Settlement as at February 2008

To put the foundation settlements into perspective, reference can be made to Figure 37, which shows the computed shortening of the structure after 30 years (Baker et al, 2007). It can be seen that the shortening is substantially greater than the foundation settlements.

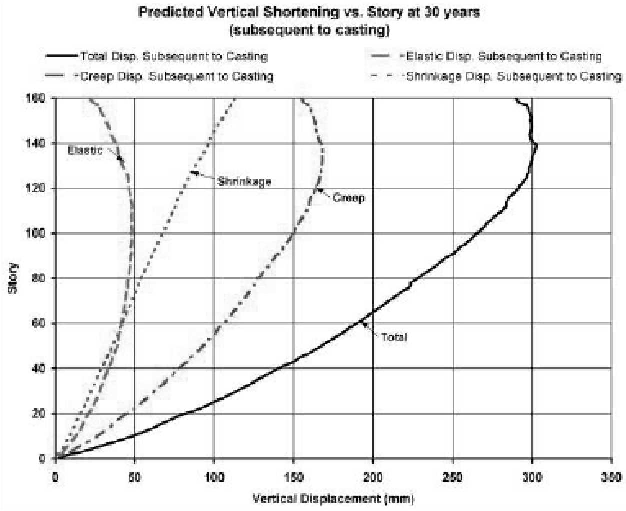


Figure 37 Predicted Vertical Shortening of Tower at 30 Years After Casting (Baker et al, 2007)

8 THE NAKHEEL TALL TOWER

8.1 *Introduction*

The Nakheel tall tower in Dubai was announced in October 2008 and will extend to a height in excess of 1000m. The tower is illustrated in Figure 38 and is to be the centre-piece of Nakheel Harbour, a 270 hectare development that will contain about 19,000 residential units, 950,000 sq m of commercial and retail space, and more than 3500 hotel rooms. The tower is designed by Australia's Woods Bagot in conjunction with the WSP Group, Dr. Leslie Robertson and Norman Disney Young. The geotechnical consultant and foundation designer is Golder Associates, based in Melbourne Australia, and geotechnical peer review has been carried out by the Sydney Australia office of Coffey Geotechnics. Preliminary details of the geotechnical conditions and foundation analyses and design have been given by Haberfield et al (2008) and the brief summary of some of the key features of the project, described below, is drawn largely from this source.

8.2 *Geotechnical Conditions and Ground Investigations*

Nine boreholes were drilled at the site, to depths ranging between 80m and 200m. The geotechnical profile consisted of a capping of sand about 20m thick, underlain mainly by cemented carbonate siltstone (calcisiltite) with gypsum layers up to 2.5m thick occurring at depths greater than about 75m below ground surface. Three broad types of testing were undertaken:

- Routine laboratory classification testing, including carbonate content, unit weight, specific gravity, moisture content, particle size and unconfined compressive strength (UCS).
- Specialist laboratory testing, including cyclic and static constant normal stiffness (CNS) direct shear testing and high pressure oedometer testing. In addition, resonant column, drained triaxial and cyclic triaxial tests were carried out on geologically similar samples from a nearby site.
- In-situ testing, including pressuremeter testing, cross-hole sonic testing and water pressure testing,

Emphasis was placed on the in-situ testing because of the concerns about stress relief of the cemented soils when sampled and brought to the surface. In addition, Golder developed a simple but useful hardness classification test for the core samples.

A key aspect of the behaviour of the foundation materials was the existence of a "bond yield strength", below which the materials were very stiff, but above which their compressibility increased by almost an order of magnitude (similar to the transition from over-consolidated to normally consolidated behaviour in a clay). One of the foundation design objectives was to keep the imposed stresses on the ground from the foundation system below the bond yield strength in order to avoid large settlements.

Figure 39 shows values of Young's modulus derived from the pressuremeter testing, the geophysical cross-hole testing and the laboratory UCS testing. The latter values were considerably lower than the other values, as might be expected. The values from the seismic testing were about five times those from the pressuremeter testing, a similar finding to that from the other two sites considered in this paper.

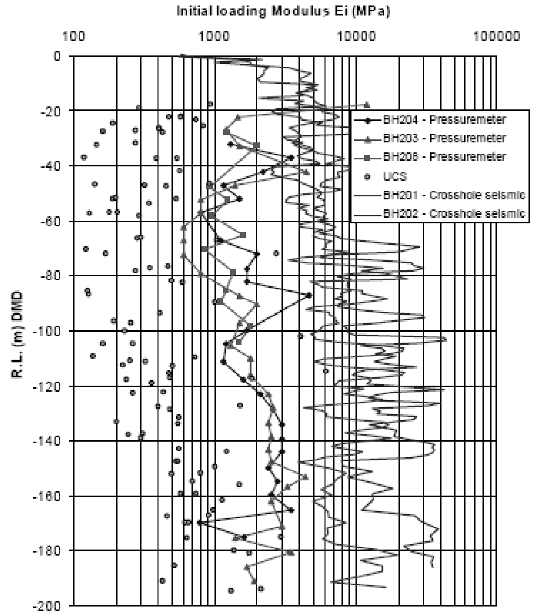


Figure 39 Variation of Young's Modulus with Elevation (Haberfield et al, 2008)

Figure 38 The Nakheel Tower and Surrounding Development – Artist's Impression

8.3 *Foundation System*

Because of the very high loads imposed by this unprecedented structure, the foundation system employed was a raft supported by barrettes. The raft was of variable thickness, up to a maximum of 8m, and the base was to be founded within the carbonate cemented siltstone below the near-surface sand deposits, with a total of 392 barrettes extending to depths of 37, 42 and 72 m below this level, depending on the location. Figure 40 shows the foundation layout. The barrettes were to be installed from the ground surface, and then excavation was to take place to the cut-off level after barrette installation was complete. At the time of writing this paper (February 2009), the barrette installation was approximately 50% complete.

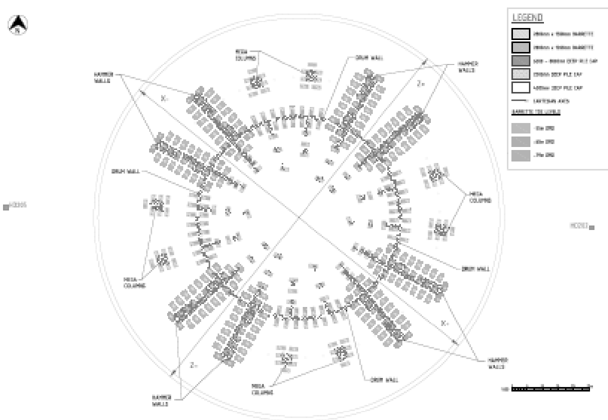


Figure 40 Foundation layout for Nakheel Tower

8.4 *Foundation Analysis and Design*

The foundation designer adopted the following process:

1. Design profiles of strength and Young's modulus with depth (mean, upper bound and lower bound) were developed from the results of the investigations.
2. Preliminary analyses were carried out using an axisymmetric model via PLAXIS v8. The barrettes were modelled as either equivalent circular rings or structural plate elements. The various strata were modelled as elastic-perfectly plastic materials.
3. Detailed analyses were undertaken using a full three-dimensional finite element analysis with PLAXIS 3D.
4. Probabilistic analyses were also undertaken to estimate the potential tilt of the tower due to variability of ground conditions.

The three-dimensional analyses gave settlement profiles and barrette loads under dead and live loading that were consistent with those derived from the axisymmetric analyses.

8.5 *Barrette Load Tests*

Three full-scale barrette tests were undertaken as part of the investigation and design process. Two of the test barrettes (TB02 and TB03) were founded at a level about 45m below the proposed raft base level, while the third (TB01) was founded much deeper, about 75m below raft base level. Load testing was carried out via the Osterberg Cell (O-Cell) technique, using two levels of cell in each test barrette. Attention was focussed on the lower parts of each barrette, since the close spacing of the barrettes meant that a significant proportion of the load on each barrette would be transferred to levels near the base. Bi-directional loads of up to 83MN were applied to each of the test barrettes.

Both the designer and the peer reviewer undertook "Class A" predictions of the test barrette behaviour. The designer estimated shaft friction values from the program ROCKET (Seidel, 1997) and based predictions of stiffness largely on data from in-situ pressuremeter tests. The program PLAXIS was used to simulate the Osterberg cell test.

The peer reviewer considered only one of the barrettes (TB02) and relied to a large extent on data from previous conventional load testing at the Burj Dubai and the Emirates, in conjunction with the data obtained from the geotechnical investigation at the site. The simplified boundary element program PIES was used to simulate the Osterberg Cell test.

Figures 41 and 42 show the designer's predictions, the peer reviewer's predictions and the measured behaviour for TB02 when the upper O-Cell was loaded. The designer's predictions were generally in good agreement with the measurements. The load-settlement behaviour of the upper part of the shaft was stiffer than that predicted by the peer reviewer from the PIES analysis, but the computed stiffness of the lower portion is similar to that measured.

The marked non-linearity of the load-settlement behaviour beyond a load of about 18 MN may be due to the presence of debris at the pile base. On the basis of these comparisons, it became apparent that both the barrette capacity and stiffness characteristics were greater than had been expected. Accordingly, the geotechnical models were refined to reflect this more favourable behaviour.

8.6 *Peer Review Process*

The peer review process, in which the author participated, involved the following steps:

1. Review of the designer's reports and design methods;
2. Review of the available geotechnical information and independent development of a design model for the site;
3. Detailed independent analyses for the proposed foundation system for both ultimate limit state (ULS) and serviceability limit state (SLS) conditions.

The initial independent geotechnical model was developed largely on the basis of previous experience with the other projects in Dubai described in this paper, together with correlations with UCS values. Subsequently, the geotechnical model was refined in the light of the barrette test results, and the values of Young's modulus, skin friction and end bearing were generally increased in the final model.

The computer programs PIGS (Pile Group Settlement) and CLAP (Combined Load Analysis of Piles), developed by Coffey Geotechnics, were used for the ULS analyses. The foundation capacities were reduced by a geotechnical reduction factor of 0.65, and the various combinations of ULS loadings were applied. In all cases, the foundation system was found to be stable, thus satisfying the ultimate limit state stability criterion.

The computer program GARP (Geotechnical Analysis of a Raft with Piles), also developed by Coffey Geotechnics, was used to compute the settlement and differential settlement of the barrette-raft foundation system. Simplified finite element analyses were also carried out to check the GARP calculations. In all of the analyses undertaken by the designer and the peer reviewer, each of the 392 barrettes was modelled, and hence it was possible to obtain individual values of pile head stiffness for input into the structural analysis programs employed by the structural designers.

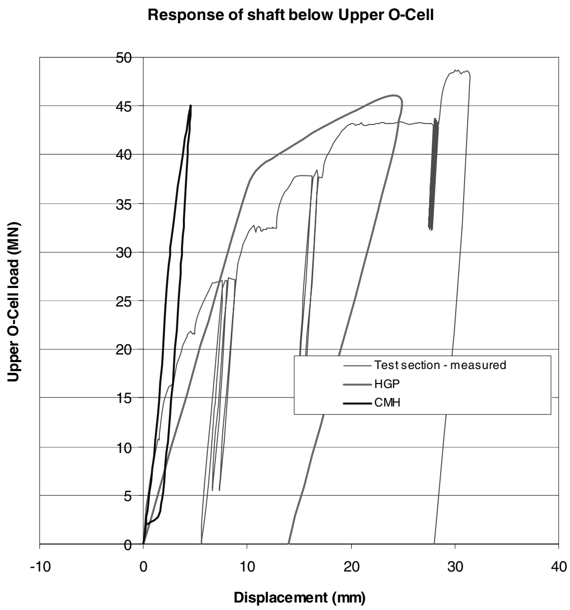


Figure 41 Measured and Predicted Behaviour of Lower Section of TB02 for Loading of Upper O-Cell. CMH=Designer, HGP = Peer Reviewer

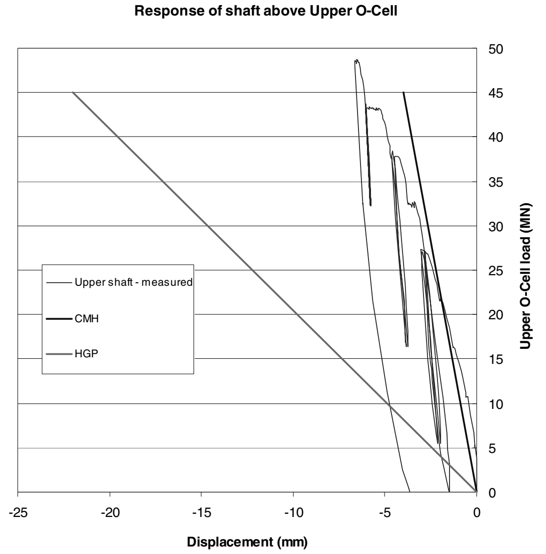


Figure 42 Measured and Predicted Behaviour of Upper Section of TB02 for Loading of Upper O-Cell. CMH=Designer, HGP = Peer Reviewer

Gratifyingly, it was found that the maximum settlements obtained from the GARP analysis (95mm) was very close to that obtained by Golder using a full three-dimensional finite element analysis with the PLAXIS 3D Foundation program. The minimum settlement from GARP of 60mm was however somewhat smaller than that computed by Golder (70mm). In view of the fact that the dead load alone is about 2,000,000 tonnes (making this one of the heaviest structures on earth), the computed settlements appeared to be quite acceptable.

9 CONCLUSIONS

The design of foundations for high-rise structures in the Middle East involves a number of challenges from a geotechnical viewpoint. The foundation system is subjected to large vertical, lateral and moment loadings, which incorporate cyclic components which have the potential to degrade the capacity and stiffness of the piles within the system. The nature of soil and rock deposits in the Middle East gives rise to additional potential problems, including generally weak to very weak founding conditions, a greater propensity for cyclic degradation, the possibility of cavities within some of the deposits, and the absence of hard rock layers on which end bearing piles can be founded. Because of these difficulties, piled raft systems, with their high level of redundancy, have proved to be an effective and relatively economical foundation solution.

For the Emirates Project, the comprehensive investigation and testing program enabled the site to be characterized in a more complete manner than is usually possible with many projects. Modern methods of in-situ and laboratory testing were used in conjunction with advanced methods of foundation analysis to design the piled raft foundations. The limit state design approach used for the foundation design involved a great deal of analysis (particularly because of the large number of load combinations to be considered. One of the major challenges was to process and portray the results from the analyses into useful design information.

The substantial design effort was complemented by a comprehensive program of pile load testing, including static compression tests, static and cyclic tension tests, and lateral load tests. Class A predictions of the performance of the test piles were found to be in fair agreement with the measurements, although more conservative. In particular, the values of ultimate skin friction along the pile inferred from the load tests were in good agreement with the values used for design, which were derived from Constant Normal Stiffness (CNS) laboratory tests. There appears to be potential for this type of test to provide a rational means of measuring pile skin friction characteristics in the laboratory.

The expectation that the tower settlements would be as well-predicted as the pile load tests was not realized. The measured values during construction were only about 25% of the predicted values. Possible reasons for the significant discrepancy were investigated, and it was found that at least some of the differences could be attributed to the conservative assumptions made in deriving the pile settlement interaction factors which were used in the piled raft analysis. By making what are considered (in retrospect) to be more realistic assumptions regarding the soil modulus values between and below the piles, it was found possible to obtain a closer match to the measured settlements. The importance of taking proper account of interaction effects in pile group analyses, and of allowing for a more realistic distribution of ground stiffness at depth, was therefore re-emphasized.

For the Burj Dubai, the maximum settlement predicted by ABAQUS for the tower and podium foundation compared reasonably well with the maximum settlement estimated by the revised PIGS analysis carried out during the independent verification process. It was assessed that there was a potential for a reduction in axial load capacity and stiffness of the foundation strata under cyclic loading; but based on the pile load test data, laboratory tests and on theoretical analyses, it would appear that the cyclic degradation effects at the pile-soil interface are relatively small.

Both the preliminary test piling program and the tests on the works piles have provided very positive and encouraging information on the capacity and stiffness of the piles. The measured pile head stiffness values were well in excess of those predicted, and those expected on the basis of the experience with the nearby Emirates Towers. However, the interaction effects between the test piles and the reaction piles may have contributed to the higher apparent pile head stiffnesses. The capacity of the piles also appeared to be in excess of that predicted, and none of the tests appeared to have fully mobilized the available geotechnical resistance. Similar findings were made for the Nakheel Tall Tower.

The works piles performed even better than the preliminary trial piles, and demonstrated almost linear load-settlement behaviour up to the maximum test load of 1.5 times working load. The foundation settlements measured during construction were consistent with, but comfortably smaller than, those predicted. Overall, the performance of the Burj Dubai piled raft foundation system has exceeded expectations.

For the Nakheel Tall Tower, use was made of modern methods of analysis and design, laboratory testing and pile load testing. As a consequence, the foundation barrettes were able to be designed more economically than would have been possible previously, and with an accompanying high degree of confidence in the foundation performance.

All three projects described herein involved close interaction between the structural and geotechnical designers in designing piled raft foundations for the complex and significant high-rise structures. Such interaction has some major benefits in avoiding over-simplification of geotechnical matters by the structural engineer, and over-simplification of structural matters by the geotechnical engineer. Such interaction therefore promotes the development of effective and economical foundation and structural designs.

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