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ABSTRACT

This paper describes the foundation design process adopted for the Burj Dubai, the world's tallest building. The foundation system is a piled raft, founded on deep deposits of carbonate soils and rocks. The paper will outline the geotechnical investigations undertaken, the field and laboratory testing programs, and the design process, and will discuss how various design issues, including cyclic degradation of skin friction due to wind loading, were addressed. The numerical computer analysis that was adopted for the original design together with the check/calibration analyses will be outlined, and then the alternative analysis employed for the peer review process will be described. The paper sets out how the various design issues were addressed, including ultimate capacity, overall stability under wind and seismic loadings, and the settlement and differential settlements.

The comprehensive program of pile load testing that was undertaken, which included grouted and non-grouted piles to a maximum load of 64MN, will be presented and "Class A" predictions of the axial load-settlement behaviour will be compared with the measured behavior. The settlements of the towers observed during construction will be compared with those predicted.

INTRODUCTION

The Burj Dubai project in Dubai comprises the construction of an approximately 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 1 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 has been undertaken by Coffey Geosciences (Coffey). This paper describes the foundation design and verification processes, and the results of the pile load testing programs. It also compares the predicted settlements with those measured during construction.

GEOLOGY

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 country is generally relatively low-lying (with the exception of the mountainous regions in the north-east of the country), with near-surface geology dominated by deposits of Quaternary to late Pleistocene age, including mobile Aeolian dune sands, evaporite deposits and marine sands.

Dubai is situated towards the eastern edge of the geologically stable Arabian Plate and separated from the unstable Iranian Fold Belt to the north by the Arabian Gulf. The site is therefore considered to be located within a seismically active area.

GEOTECHNICAL INVESTIGATION & TESTING PROGRAM

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

Phase 1 (main investigation): 23 boreholes, in situ SPT's, 40 pressuremeter tests in 3 boreholes, installation of 4 standpipe piezometers, laboratory testing, specialist laboratory testing

and contamination testing – 1st June to 23rd July 2003;

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.

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 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.

Fig 1: Impression of Burj Dubai when Complete

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.

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 1.

Table 1. Summary of Geotechnical Profile and Parameters

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 that the E_u and E' values relate to the relatively large strain levels in the strata.

Stiffness values from the pressuremeter reload cycle, the specialist tests and the geophysics are presented in Figure 2. There is a fair correlation between the estimated stiffness profiles from the pressuremeter and the specialist testing results at small strain levels.

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 3.

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 3.
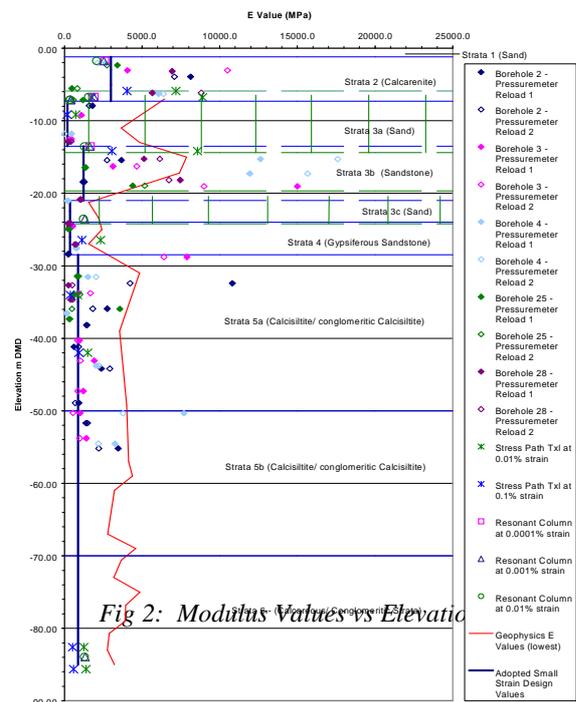

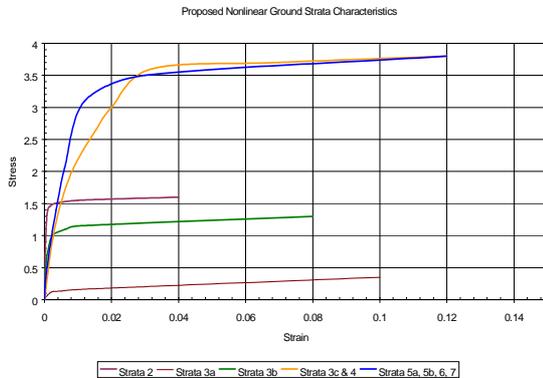
Fig 2: Modulus Values vs Elevation

Fig 3: Non-linear Stress-strain Curves

GEOTECHNICAL MODELS AND ANALYSES

A number of analyses were used to assess the response of the foundation for the Burj Dubai Tower and Podium. The main design model was developed using a Finite Element (FE) program ABAQUS run by a specialist company KW Ltd, based in the UK. Other models were developed to validate and correlate the results from the ABAQUS model using software programs comprising REPUTE (Geocentrix, 2002), PIGLET (Randolph, 1996) and VDISP (OASYS Geo, 2001).

The ABAQUS model comprised a detailed foundation mesh of 500m by 500m by 90m deep. The complete model incorporated a 'far field' coarse mesh of 1500m by 1500m by 300m deep. A summary of the model set up is as follows:

Soil Strata: Modeled as Von Mises material (pressure independent), based on non-linear stress-strain curves*Tower Piles*: Modeled as beam elements connected to the soil strata by pile-soil interaction elements. Class A load-settlement predictions were used to calibrate the elements;
Podium Piles: Beam elements fully bonded to the soil strata;
Tower and Podium Loadings: Applied as concentrated loadings at the column locations;
Tower raft submerged weight: Applied as a uniformly distributed load;
Tower Shearing Action: Applied as a body load to the tower raft elements, in a direction to coincide with the appropriate wind action assumed;
Building Stiffness Effect: Superstructure shear walls (not interrupted at door openings) were modeled as a series of beam elements overlaid on the tower raft elements. The moment of inertia was modified to simulate the stiffening effect of the tower, as specified by SOM.

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):

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

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 1. The ultimate unit pile skin friction of a pile loaded in tension was taken as half the ultimate unit shaft resistance of a pile loaded in compression.

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.

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 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 and are shown in Table 2:

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

Table 2. 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

Gratifyingly, the settlements from the FEA model correlated acceptably well with the results obtained from REPUTE, PIGLET and VDISP.

A sensitivity analysis was carried out using the FE analysis model and applying the maximum design soil strata non-linear stress-strain relationships. The results from the stiffer soil strata response gave a 28% reduction in Tower settlement for the combined Dead load, live load and wind load case analyzed, from 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 4 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.
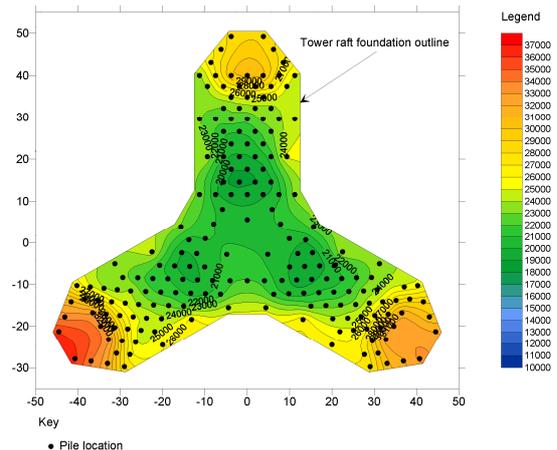

Fig 4: Contours of Maximum Axial Load

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.

In reality the actual pile load distribution is expected to be somewhere between the two models depending on the impact of the different modeling approaches.

OVERALL STABILITY ASSESSMENT

The minimum centre-to-centre spacing of the piles for the tower is 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 just less than 2 was assessed for vertical block movement, excluding base resistance of the block while a

factor of safety of greater than 2 was determined for lateral block movement excluding passive resistance. A factor of safety of approximately 5 was obtained against overturning of the block.

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 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.

INDEPENDENT VERIFICATION ANALYSES

The geotechnical model used in the verification analyses is summarized in Table 3. 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 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 3, 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, 2002).

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 cried 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.

FLAC & PIGS Results For The Tower Alone

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

FLAC analysis, using an equivalent block to represent the piles: 72.9mm

PIGS analysis, modeling all 196 piles: 74.3mm

Thus, despite the quite different approaches adopted, the computed settlements were in remarkably good agreement. It should be noted that 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.

PIGS Results For Tower & Podium

Figure 5 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.

Figures 6 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.

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

Table 3. Summary of Geotechnical Model for Independent Verification Analyses

| 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-v.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/conglom.	-28.5 to −50	250	200	250	2.5
5B	V. weak-weak-mod. Weak calcisiltite/conglom	-50 to −70	275	225	275	2.75
6	Calcareous siltstone	-70 & below	500	400	375	3.75
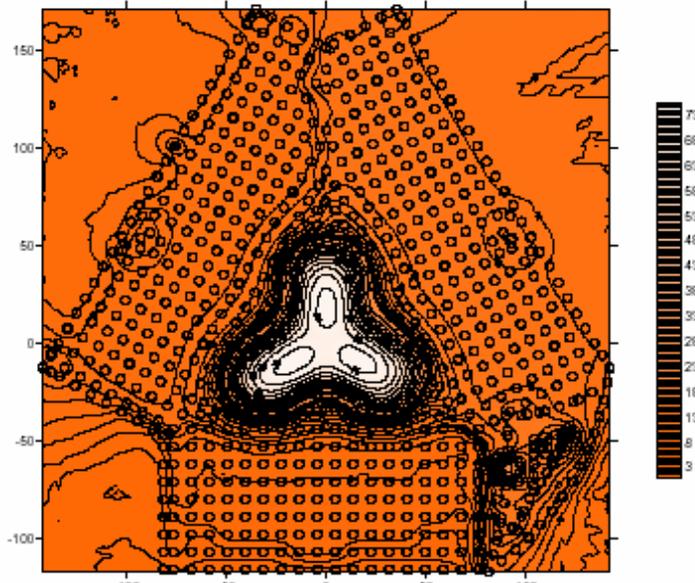
Fig 5: Computed Settlement Contours for Tower and Podium

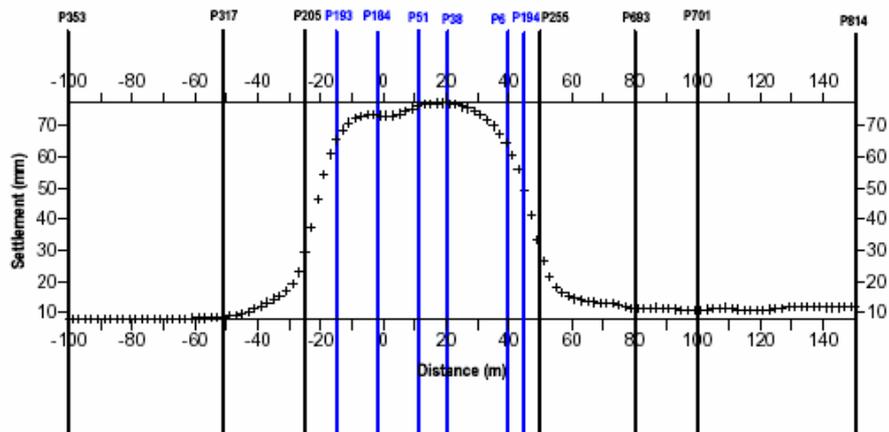*Fig 6: Computed Settlement Across Section Through Centre of Tower*

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 is 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 are 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.

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.

Preliminary Pile Testing Program

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

Ultimate Axial Load Capacity

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.

Ultimate Shaft Friction

From the strain gauge readings along the test piles, the mobilized skin friction distribution along each pile was evaluated. Figure 7 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:

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.
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.

The original assumptions appear to be comfortably conservative within the upper part of the ground profile.
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.
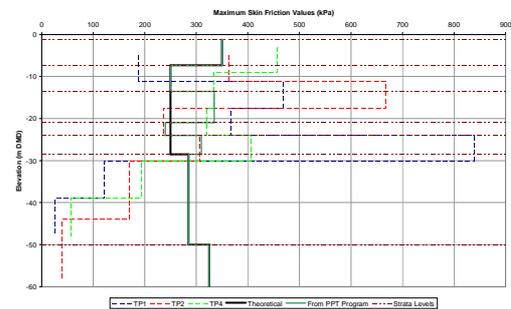

Fig 7: Measured and Design Values of Shaft Friction

Ultimate End Bearing Capacity

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.

Table 5. 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

Load-Settlement Behaviour

Table 5 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.

Effect of Reaction Piles

On the basis of the experience gained in the nearby Emirates Project (Poulos and Davids, 2005) site), 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. This is shown in Figure 8, which compares the measured stiffness values with the predicted values, at the working load. As mentioned above, the high measured stiffness may be, at least partly, a consequence of the effects of the adjacent reaction piles. An analysis of the effects of these reaction piles on the settlement of pile TP1 revealed that the presence of the reaction piles could reduce the settlement at the working load of 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.
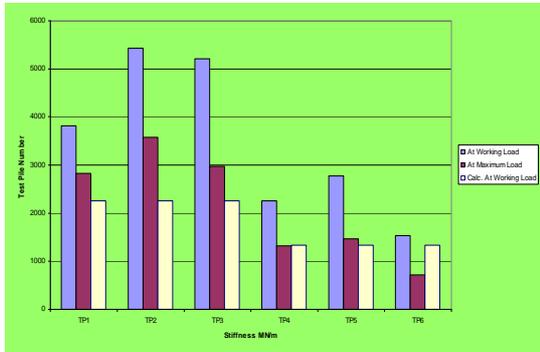

Fig 8: Measured and Predicted Pile Head Stiffness Values

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 6 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 6. 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.

Works Pile Testing Program

A total of eight works pile tests were carried, 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 6). 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.

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 are that the design estimates of capacity and settlement may be conservative, although it must be borne in mind that the overall settlement behaviour (and perhaps the overall load capacity) are dependent not only on the individual pile characteristics, but also on the characteristics of the ground within the zone of influence of the structure.

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 pile. This paper presents only the current situation on the settlement. The results from the strain gauges will be presented at a later date.

A summary of the settlements to 18 March 2007 is shown on Figure 9 which also shows the final predicted settlement profile from the design. At that date, it is estimated that about 75% of the dead load would have been acting on the foundation. It should 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. It will be seen that the measured settlements are considerably less than those predicted during the design process, although there remains some dead and live load to be applied to the foundation system.

CONCLUSIONS

This paper has outlined the processes followed in the design of the foundations for the Burj Dubai and the independent verification of the design. The ground conditions at the site comprise a horizontally stratified subsurface profile which is

complex whose properties are highly variable with depth. A piled raft foundation system, with the piles socketed into weak rock, has been employed and the design of the foundation is found to be governed primarily by the tolerable settlement of the foundation rather than the overall allowable bearing capacity of the foundation. The capacity of the piles will be derived mainly from the skin friction developed between the pile concrete and rock, although limited end bearing capacity will be provided by the very weak to weak rock at depth.

The estimated maximum settlement of the tower foundation, calculated using the various analysis tools are in reasonable agreement, with predicted settlements of the tower ranging from 45mm to 62mm. These results are considered to be within an acceptable range.

The maximum settlement predicted by ABAQUS for the tower and podium foundation compares reasonably well with the maximum settlement estimated by the revised PIGS analysis carried out during the independent verification process.

There is 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.
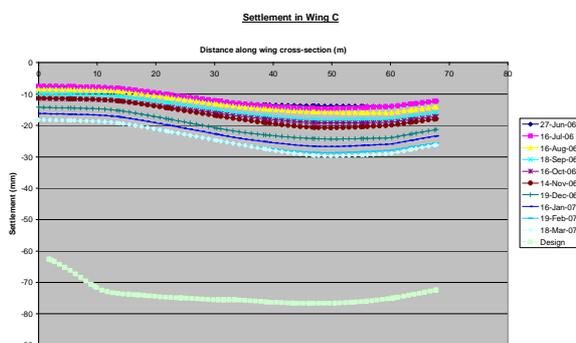

Fig 9: Measured and Computed Settlements for Wing C.

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 have been 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 appears to be in excess of that predicted, and none of the tests appears to have fully mobilized the available geotechnical resistance.

The works piles have performed even better than the preliminary trial piles, and have demonstrated almost linear load-settlement behaviour up to the maximum test load of 1.5 times working load.

The settlements measured during construction are consistent with, but smaller than, those predicted, and overall, the performance of the piled raft foundation system has exceeded expectations to date.

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