

### Keywords

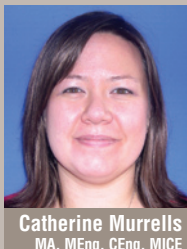
geotechnical engineering; foundations;  
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# Foundation design for the Pentominium tower in Dubai, UAE

The Pentominium tower in Dubai, UAE will be the tallest residential building in the world at over 100 storeys tall when completed in 2012. This paper describes the design of the tower's piled raft foundation in the local carbonate soils and rock. Geotechnical investigations are outlined, along with how the effect of the proposed tower on neighbouring structures, single-pile response and impact of cyclic degradation were assessed. A description of the numerical analyses used to evaluate the overall piled raft response under various static and wind loading combinations is presented as well as some of the techniques used to optimise the foundation design, including preliminary pile testing.

The Pentominium residential development is located approximately 500 m to the east of Dubai Marina and south of the beach near

the Jumeira Palm in the United Arab Emirates (UAE) (Figure 1). The development comprises the construction of a tower over 100 storeys tall



Figure 1. Location of the Pentominium tower on the Dubai waterfront

and associated podium structure (Figure 2). It is set to be the tallest residential building in the world when completed in 2012.

All site levels are related to Dubai Municipality datum (DMD) and original ground level is at about 5 m DMD. There are six basement levels and the structure is supported by a piled raft system comprising large-diameter bored piles cast in situ. The pile cut-off levels are founded about 24 m below existing ground level at an elevation of -19.4 m DMD.

The area of the site is very limited with the structure extending to all boundaries. In addition, construction on the adjacent Marina 23 tower is underway, with a number of levels of superstructure completed before piling started on the Pentominium tower in 2008.

The client for the project is Trident International Holdings and the architect is Aedas. Hyder Consulting carried out the detailed design of the foundation, substructure and superstructure. Arab Centre for Engineer Studies (ACES) carried out the ground investigation and Swiss-boring Limited was the piling contractor.

## Geology

The geology of the UAE and the Persian Gulf area has been substantially influenced by the deposition of marine sediments associated with numerous sea level changes during relatively recent geological times. 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, sabka/evaporite deposits and marine sands.

Dubai is situated towards the eastern extremity of the geologically stable Arabian tectonic plate and is separated from the unstable Iranian fold belt to the north by the Persian Gulf. It is therefore considered that the site is located within a moderately seismically active area. However, it was indicated from the structural analysis that the wind effect was more critical than the seismic effect as is typical for structures of this size in Dubai.

## Geotechnical investigation and testing

The ground investigation was undertaken by ACES and consisted of sinking eight cable percussion boreholes with rotary follow-on methods, in-situ testing and laboratory testing (including specialist testing) on selected samples. The boreholes were drilled to 80–125 m deep with standpipe piezometers installed to monitor the groundwater table. The scope of in-situ testing is summarised as follows

- standard penetration testing
- packer permeability testing
- pressuremeter testing at 3 m intervals in three of the boreholes
- geophysics (cross-hole, cross-hole tomography and down-hole testing).

Disturbed, undisturbed and split-spoon samples were obtained from the boreholes for laboratory testing purposes. The undisturbed samples were obtained using double-tube-core barrels from which 92 mm nominal core



Figure 2. Artist's impression of the Pentominium tower, which, at over 100 storeys tall, will be the highest residential building in the world when completed in 2012

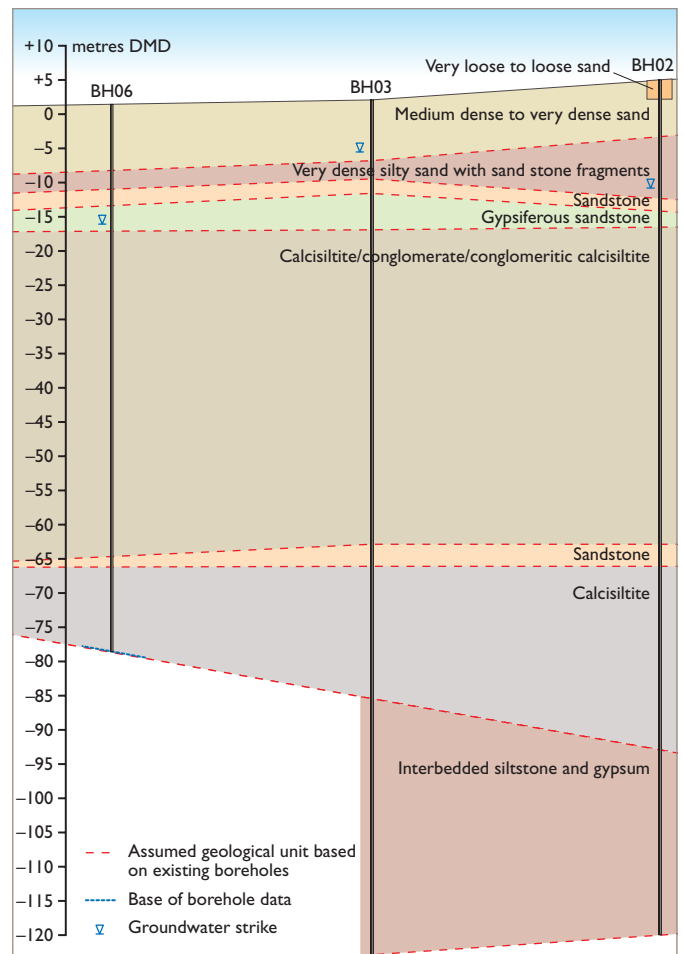


Figure 3. Geological long-section through three of the boreholes

diameters were recovered. The laboratory testing included the following standard and specialist tests

- standard classification testing
- chemical testing
- unconfined compression tests
- cyclic undrained triaxial
- cyclic simple shear
- stress path triaxial testing
- resonant column
- constant normal stiffness testing.

The standard ground investigation testing was carried out following British standards BS 5930<sup>1</sup> and BS 1377.<sup>2</sup>

Four preliminary trial pile tests were also carried out to determine single-pile load-settlement behaviour and to assess the pile capacity in skin friction.

### Geotechnical conditions and parameters

The ground conditions comprise a horizontally stratified sub-surface profile. Three layers of sand, varying from very loose to medium dense to dense as elevation decreases, overlies layers of very weak to weak sandstone, gypsiferous sandstone, calcisiltite, conglomerates and calcareous siltstones.

An idealised ground profile used for the whole site is presented in Table 1 and a section through three of the boreholes is shown in Figure 3.

The geotechnical stiffness parameters for the design of the foundation were determined from the tests carried out on the strata at different strain levels. It is presented in Mayne and Schneider<sup>3</sup> that rock behaviour for deformation analyses, which would include piled rafts, ranges between strain values of approximately 0.01–0.1%, which correlate with testing results from the pressuremeter, stress path triaxial, resonant column, cyclic triaxial testing and geophysics which are presented on Figure 4.

It should be noted that the design line shown on Figure 4 is for small strain design at 0.1% strain. The stiffness values provided in Table 1 are for the larger strain levels of approximately 1% determined from standard correlations with unconfined compressive strength results as presented in Tomlinson and Woodward.<sup>4</sup> drained Young's modulus at large strain  $E' = M_r j q_u$ , where  $M_r$  is the ratio of elastic modulus of intact rock to its uniaxial compressive strength,  $j$  is the mass factor and  $q_u$  is the uniaxial compressive strength.

Non-linear stress-strain curves were developed for the rock strata based on all the ground investigation data. The curves were fitted to the data using hyperbolic functions as presented by Mayne and Schneider<sup>3</sup> and are

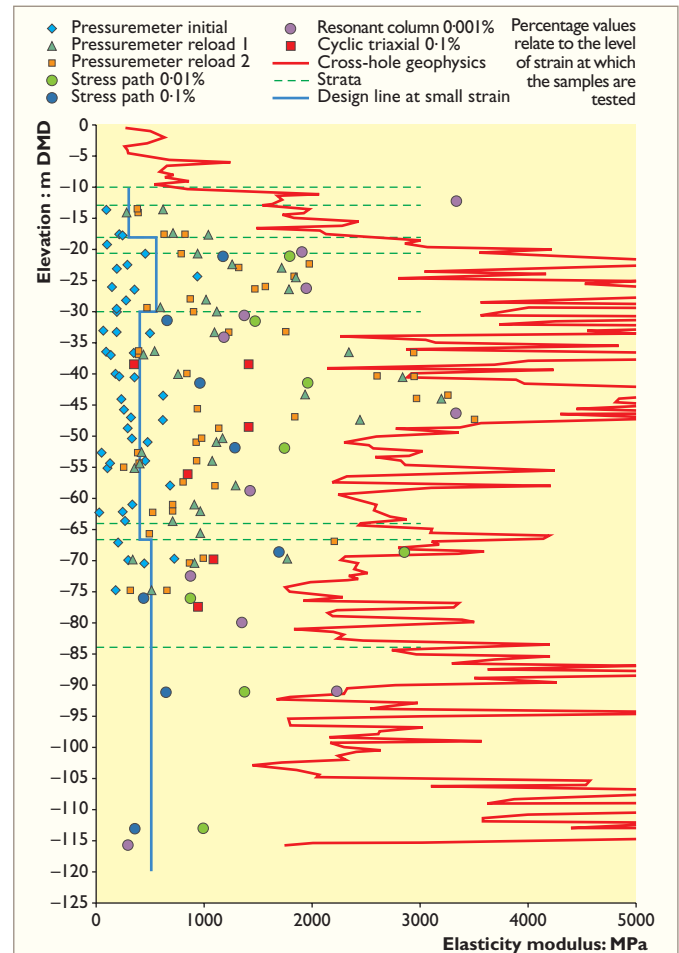


Figure 4. Young's modulus  $E'$  values derived from geotechnical testing

Table 1. Idealised ground profile and geotechnical parameters

Strata number	Sub-strata number	Material	Level at top of stratum: m DMD	Thickness: m	Unconfined compressive strength: MPa	Undrained modulus* $E_u$ : MPa	Drained modulus* $E'$ : MPa
1	1a	Very loose to loose slightly silty sand with occasional sandy silt	+1.40 to +5.59	2.70	–	–	2
	1b	Medium dense to very dense slightly silty to silty sand	–0.13 to +5.59	9.50	–	–	36
	1c	Very dense silty sand with sandstone fragments	–7.50	3.00	–	–	75
2	2	Very weak to weak calcarenite/calcareous sandstone interbedded with cemented sand	–10.50	2.20	0.8	125	100
3	3	Very weak to weak gypsiferous sandstone	–12.70	5.30	0.8	125	100
4	4a	Very weak to moderately strong calcisiltite/ conglomerate/ conglomeritic calcisiltite	–18.00	2.50	0.8	125	100
	4b		–20.50	9.50	3.0	350	280
	4c		–30.00	34.00	2.4	250	200
5	5	Weak sandstone	–64.00	2.60	2.4	250	200
6	6	Very weak to moderately strong calcisiltite	–66.60	17.40	4.1	250	200
7	7	Very weak to moderately weak claystone/siltstone interbedded with gypsum layers	–84.00	>38.00 (proven to base of boreholes)	3.0	250	200

\* Note that  $E_u$  and  $E'$  values relate to large strain level (about 1%) of the strata

presented in Figure 5.

The single-pile load–settlement behaviour was estimated using the Cemset method as developed by Fleming<sup>5</sup>; experience was also utilised from previous pile tests carried out and assessed in the region, which includes tests around the Dubai Marina area as well as the rest of Dubai. The estimated load–settlement behaviour is shown in Figure 6 and is compared with the measured results from the preliminary pile testing.

The groundwater levels encountered in the boreholes varied widely between  $-5.91$  m DMD to  $-14.57$  m DMD. However, a large number of projects are under construction in the adjacent area and it is considered that the associated dewatering has artificially lowered the groundwater. The site is sufficiently remote from Dubai

Marina and the main coast for this to be possible. Information from other ground investigation works in the neighbouring sites indicated a groundwater level ranging between  $-2.1$  and  $-2.9$  m DMD. Taking into account the information from adjacent sites and the fact that no tension loading is applicable for the development, a conservative groundwater level of  $-1.5$  m DMD was taken for design purposes.

### Geotechnical models and analyses

A number of analysis techniques have been used to assess the piled raft foundation response for the Pentominium tower. The foundation design methodology has been developed using previous experience in Dubai and a number of

results from projects completed or under construction, such as the Burj Dubai as presented by Poulos and Bunce<sup>6</sup> and the Emirates towers as presented by Poulos and Davids.<sup>7</sup>

The main model was set up using the geotechnical finite-element analysis program Midas-GTS<sup>8</sup> with the model being developed by TNO Diana in the Netherlands. Other models were run to correlate and validate the results from the finite-element analysis including standard pile-group analysis using Repute<sup>9</sup> and equivalent raft analysis using Vdisp.<sup>10</sup> In addition, the results were compared with those obtained from the model set up in Strand 7 which was used for the structural design.

A number of foundation options were considered for the Pentominium tower including  $1.5$  m and  $2.2$  m diameter pile systems and a barrette solution. The basement levels extended to  $25$  m below existing ground level. However, due to the presence of the relatively high groundwater level, it was proposed that the piles/barrettes were installed from approximately  $-4$  m DMD, to reduce flow of water from the pile bores during construction. This was approximately  $10$  m below existing ground level and  $15$  m above pile/barrette formation level. From a preliminary assessment of the generated pile/barrette load distributions, it was determined that the  $1.5$  m diameter pile solution would be the most effective in terms of constructability as well as load distribution, particularly considering that

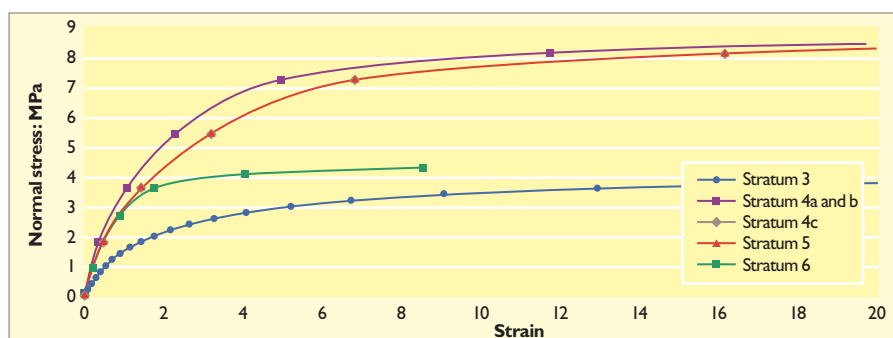


Figure 5. Non-linear stress strain curves for rock strata

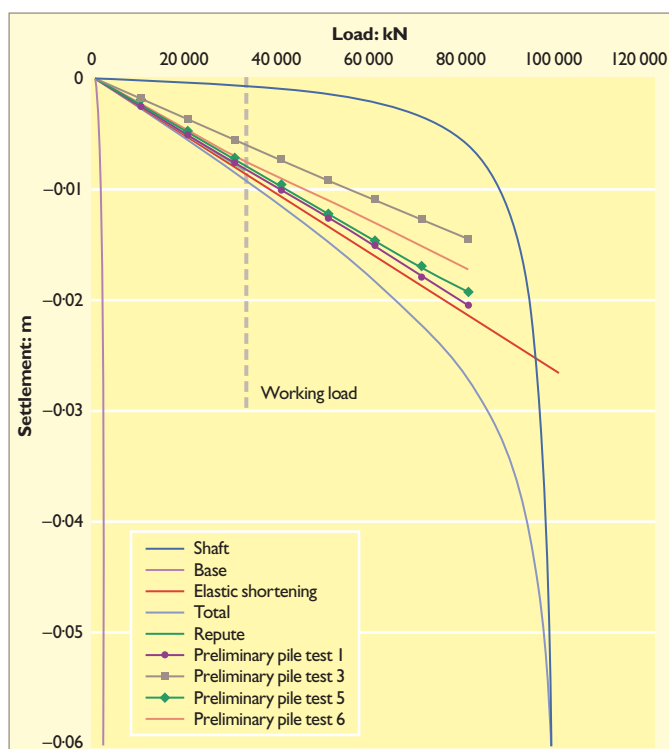


Figure 6. Single-pile load–settlement behaviour

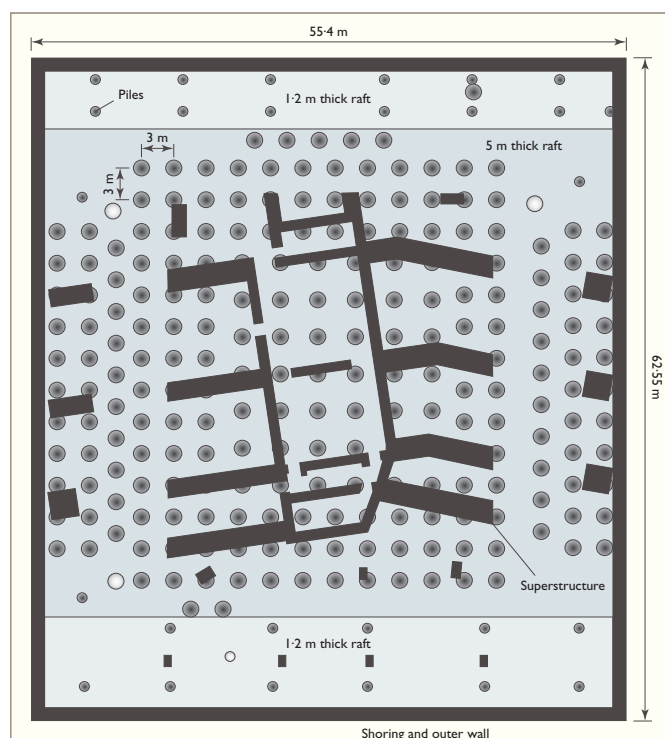


Figure 7. Layout of the  $1.2$ – $1.5$  m diameter  $32$ – $42$  m deep piles under the foundation raft (see Table 6 for pile details)

the plant for the construction of the required lengths of 2.2 m diameter piles or barrettes was not easily available in Dubai.

The adopted pile layout is shown on Figure 7. The raft within the tower area is 5 m thick and 1.2 m thick within the podium area. Preliminary analyses determined that the longest piles proposed were 56 m long, however after value engineering; these pile lengths were reduced significantly resulting in an optimised design.

The entire raft plan area for the Pentominium tower is approximately 60 m by 53 m and the resulting finite-element analysis model was 250 m × 250 m in plan and 220 m vertically to avoid boundary effects. Within the raft, the average mesh size was 1.5 m with the mesh size increasing to 30 m at the model boundary. A summary of the set up of the model is as follows.

- **Soil strata** – modelled as solid elements with parameters defined for  $E'$  and Poisson's ratio for the elastic linear runs and using a user-defined material within MidasGTS for the non-linear runs.
- **Piles** – modelled as beam elements with each node of the beams connected to the nearest soil solid elements using pile interface elements.
- **Superstructure** – modelled as beam and plate elements to represent the columns, walls and slabs. The six basement levels were included as well as nine levels of structure above ground level. Within the structure, there are large inclined columns to spread the load from the centre of the tower to the outer edges. The superstructure included in the model is presented on Figure 8.
- **Loadings**. These were applied at level 9 due to the presence of the large inclined columns such that the distribution of load through those columns could be simulated. Self-weight of the superstructure elements were included to level 9. Hydrostatic loading was included as an uplift pressure on the raft foundation.

The foundation was also assessed by structural engineers through modelling the soil as brick elements in Strand 7. The settlements obtained from the geotechnical MidasGTS model were incorporated into the Strand model to calibrate the pile stiffness values such that the behaviour of the raft could be determined and its effect on the superstructure above.

## Foundation design results

The estimated settlements from the finite-element 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 model outputs using the following general equation for a rectangle

$$\delta_{\text{rigid}} = 1/3 (2\delta_{\text{centre}} + \delta_{\text{corner}})_{\text{flexible}}$$

The computed settlements from all the analyses are presented in Table 2. It is indicated that the results from the finite-element analysis, Repute and Vdisp correlate relatively well for the same soil profile used. It is noted that large differential settlements have been calculated from the Vdisp analysis, however this does not

include the 5 m thick raft or the effect of the superstructure above and therefore the values are not realistic.

From the geotechnical finite-element analysis, when non-linear soil is assumed, it is indicated that while the overall settlement increases by 15 mm, the differential settlement only increases by 3 mm, which is within tolerances for design of the raft. A contour plot of the settlements under dead plus live load from the MidasGTS run using the smaller strain soil stiffness profile

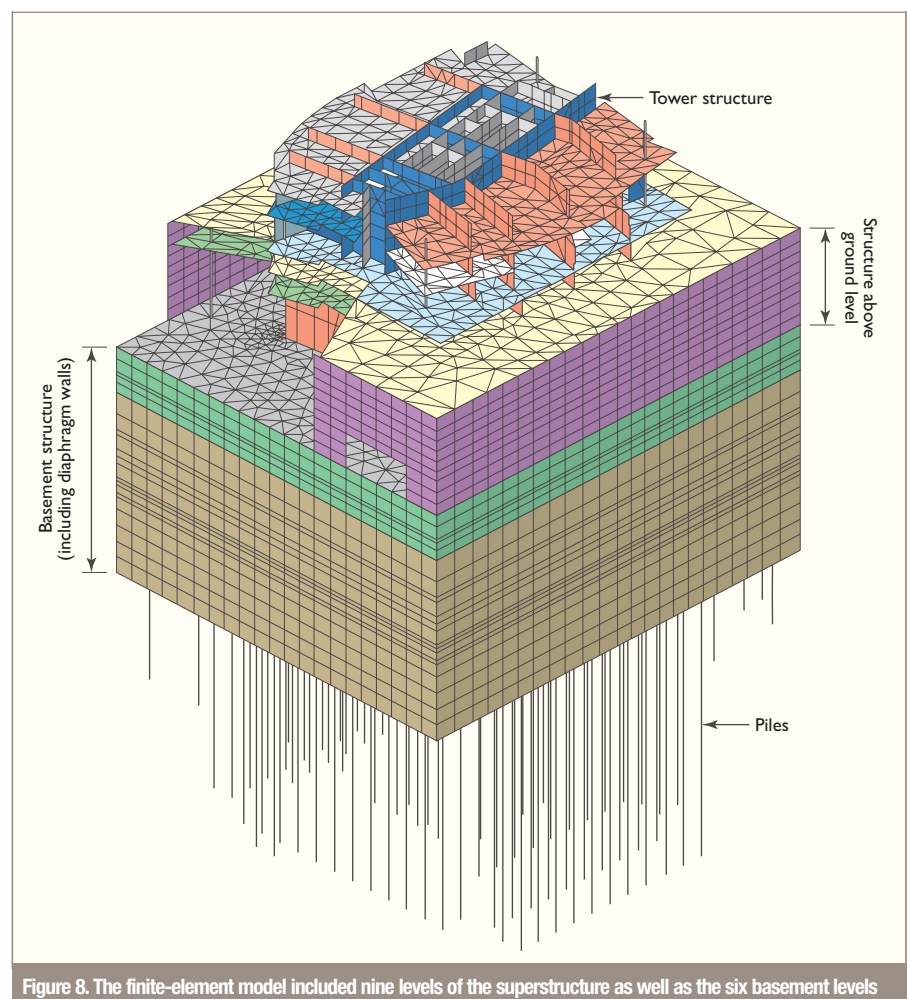


Figure 8. The finite-element model included nine levels of the superstructure as well as the six basement levels

Table 2. Computed settlements under dead and live loading from analyses

Computer package	Soil parameters	Maximum settlement: mm		
		Flexible	Rigid	Differential
MidasGTS	Linear, small strain	77	69	20
MidasGTS	Non-linear	92	85	23
Vdisp	Linear equivalent raft, large strain	217	198	32
Vdisp	Linear equivalent raft, small strain	102	78	37
Vdisp	Non-linear equivalent raft	147	115	97
Repute	Linear, large strain	–	166	–
Repute	Linear, small strain	–	87	–

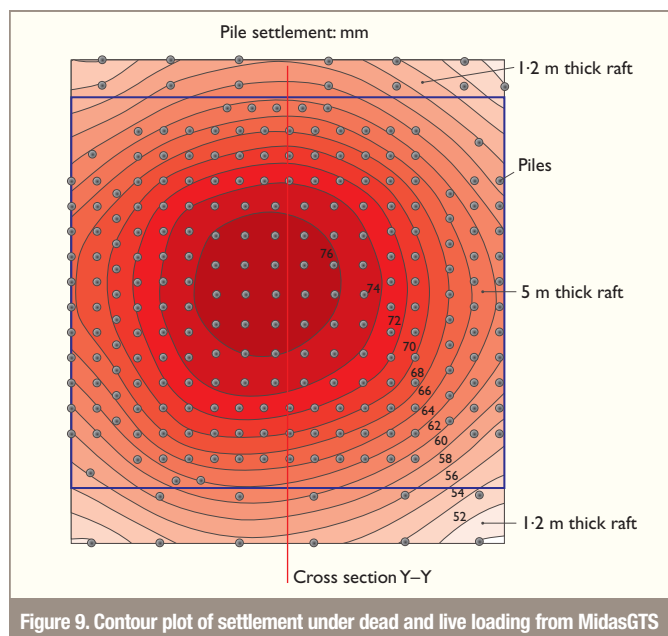


Figure 9. Contour plot of settlement under dead and live loading from MidasGTS

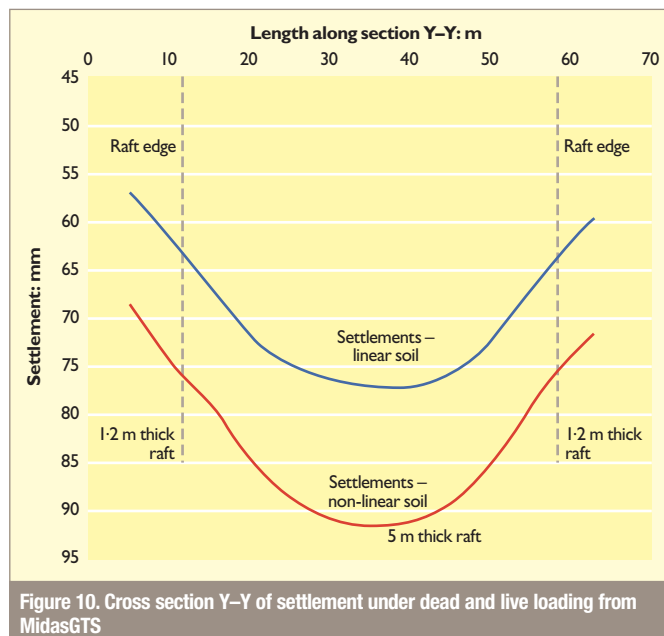


Figure 10. Cross section Y-Y of settlement under dead and live loading from MidasGTS

is presented in Figure 9. A cross-section of settlement through the foundation is presented in Figure 10, which compares the settlements from the MidasGTS model for linear and non-linear soil stiffness profiles.

Pile axial forces were determined by comparing the results from the MidasGTS model, structure's Strand model and Repute. The resultant contour plots of the pile axial loads are presented in Figure 11. The standard pile-group analysis program, Repute, assumes that the pile cap is infinitely rigid and therefore, the pile loads are concentrated towards the outside of the group. The pile axial load distribution obtained from MidasGTS and Strand, on the other hand, are similar and both programs take

account of the stiffness of raft and superstructure as well as the wall and column locations at which the foundation is loaded. From the contour plots, it is observed that the pile axial loads are concentrated towards the outer left and right sides of the foundation as this is where the inclined columns in the superstructure spread the applied load.

### Pile load testing

Four preliminary trial pile tests were carried out as summarised in Table 3. In addition, static and dynamic working pile tests are specified to be carried out on the piles prior to construction of the raft in accordance with Dubai Municipality

regulations. The purposes of the tests were to validate the design assumptions made during the design including the load-settlement response of the piles and the ultimate skin friction mobilised along the pile shaft.

Loading of the piles was achieved using Osterberg cells installed part-way down the piles. Pile displacements were measured above and below the Osterberg cells and an equivalent top-loaded pile load-settlement curve was assessed. The single-pile load-settlement curves from the preliminary pile testing are compared with those determined from theory on Figure 6. It is noted that the single-pile behaviour observed from the preliminary pile testing is stiffer than that predicted. However, at the

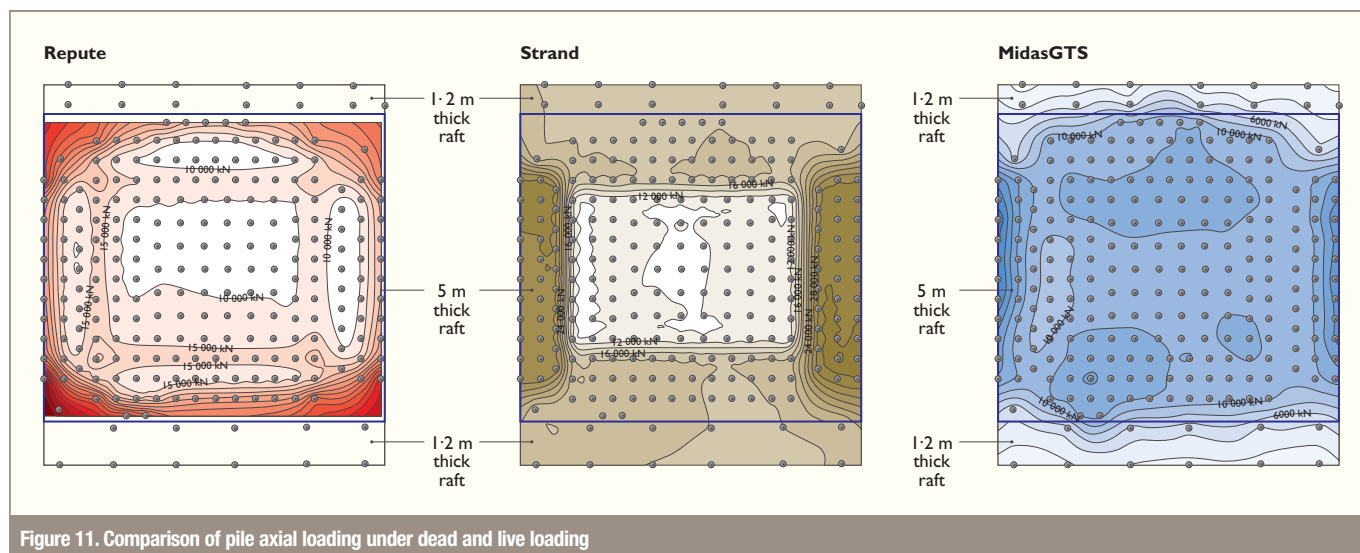


Figure 11. Comparison of pile axial loading under dead and live loading

working load of the pile of 32 500 kN, the settlements calculated from theory are very similar to those measured during testing.

Strain gauges were installed at eight levels along the length of each pile such that the skin friction mobilised along each section of pile could be calculated. The skin friction values were originally calculated using theory based on the design recommendations given by Horvarth and Kenney<sup>11</sup> as

$$f_s = 0.25 \text{ to } 0.33 (q_u)^{0.5}$$

where  $f_s$  is the ultimate unit shaft resistance and  $q_u$  is the uniaxial compressive strength in MN/m<sup>2</sup>.

The skin friction values from theory are compared with those calculated from the preliminary pile testing in Table 4. Based on results from the preliminary pile testing, the idealised rock profile has been simplified which also takes into account potential pile–rock interface degradation. The skin friction values have been improved from those calculated from theory and the factor of safety used for design was 2.5. It is noted that, due to the number of consistent pile test results received, the factor of safety could have been reduced to 2, however the pile–soil block mechanism proved to be critical to the design and therefore the factor of safety was maintained at 2.5.

The results from the preliminary pile tests are shown in Figure 12. Ultimate skin friction has not been mobilised, at the pile locations furthest from the Osterberg cell and as such none of the piles were tested to failure.

Substantial movements at the base of the pile would be required before the ultimate base bearing capacity can be mobilised, particularly if relatively soft materials are left at the base of the piles. From previous experience in the region it has been found that it is very difficult to clean sufficiently the base of such large diameter long piles. Therefore, taking this into account, together with the structural serviceability limits in terms of settlements, the pile base resistance contributions were excluded in the estimate of the pile bearing capacity.

## Cyclic loading analysis

Several cyclic loading tests were carried out on both the rock mass and the pile–rock interface. In each case, in situ stresses were applied.

Constant normal testing was carried out on samples of the rock mass sheared against concrete that had been profiled by a water-jet technique to simulate the in situ pile–soil interface. Samples were sheared both monotonically and cyclically. From the results, it was indicated that the peak shear strength was not affected by cyclic shearing, however the residual shear strength was reduced by 15%.

Table 3. Preliminary test pile (PTP) configuration

	PTP1	PTP3	PTP5	PTP6
Max test load: MN	61.80	67.50	59.20	56.30
Working load: MN	30	30	30	30
Diameter: m	1.50	1.50	1.50	1.50
Ground elevation: m DMD	+4.50	+3.50	+3.50	+3.50
Top of concrete: m DMD	−15.10	−15.15	−15.60	−15.23
Design cut-off level: m DMD	−19.10	−19.10	−19.10	−19.10

Table 4. Pile ultimate compressive skin friction values (see Table 1 for strata details)

Sub-strata number	Ultimate unit shaft friction from theory, $f_s$ : kPa	Ultimate unit shaft friction recommended from pile tests, $f_s$ : kPa
2	215	215
3	215	215
4a	215	215
4b	415	415
4c	372	415
5	372	415
6	486	415
7	415	415

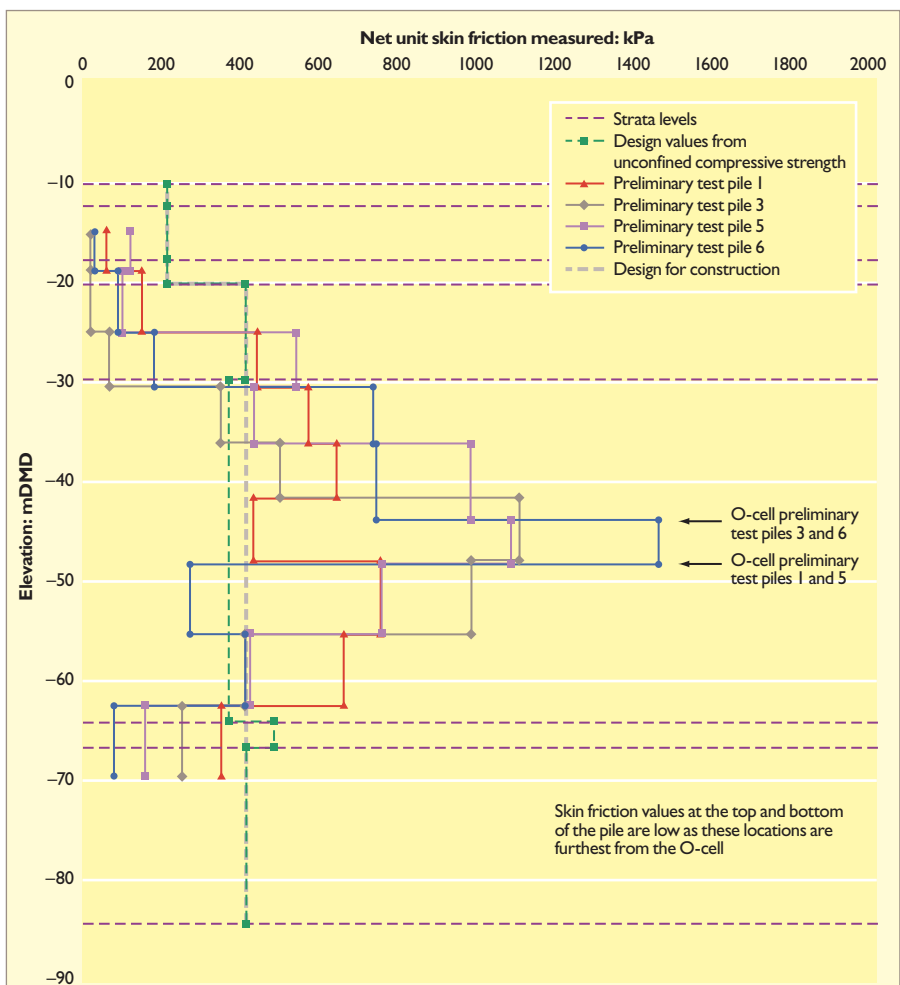


Figure 12. Pile skin friction values from theory and pile testing

Therefore, the maximum mobilised skin friction values as measured from the preliminary pile testing have not been adopted in the final design to allow for any potential degradation effects due to cyclic loading.

In addition, during the preliminary pile testing programme, ten cycles of loading were applied to each pile to determine whether cyclic degradation at the pile–rock interface occurred. Through a Timeset analysis<sup>12</sup> of the results obtained, which can be used to assess long-term degradation, it was indicated that no degradation had been observed. These results are also compared with those from the constant normal testing above.

Cyclic triaxial testing and cyclic simple shear testing were conducted on samples of the soil mass collected during the ground investigation works. From the cyclic triaxial testing, it was determined that after cyclic loading, the stiffness of the samples was similar to those samples tested monotonically during the stress path triaxial testing. This can be observed from Figure 4 where stiffness values from the cyclic triaxial testing and monotonic stress path triaxial tests have been determined at the same strain level of 0.1% and it is shown that the stiffness values from both tests were similar. The cyclic simple shear tests, which were carried out at a higher strain level, indicated that some degradation due to cyclic loading could occur at these larger strains. Young's modulus design values at smaller strain have thus been determined based on a lower bound to take account of any degradation of the soil mass.

### Overall stability assessment

The minimum centre-to-centre spacing of the piles adopted in the design is about 2–2.3 times

the pile diameter. Therefore, one of the failure mechanisms of the piles could be a block movement of the piles and soil.

Taking into account only the side frictional resistance along the perimeter of the soil–pile block to the depth of the pile toe levels, an acceptable factor of safety of 2.5 was achieved against applied vertical loading. When the lateral resistance of the block is considered, the sliding frictional resistance between the raft and the underlying rock was found to be far in excess of the applied lateral loading.

Considering the overturning failure mechanism and assuming the most onerous point of rotation of the block, a factor of safety of 2 is achieved. Therefore, it is considered that the overturning block movement of the piles is critical to the design of the pile length. Hence, despite the possibility for reducing the factor of safety on the ultimate skin friction capacity of an individual pile, as discussed previously, this has not been implemented and the pile lengths were not shortened due to the concern of overturning block movement failure.

### Effect on adjacent structure

The Pentominium tower is being constructed immediately adjacent to the Marina 23 tower, which is currently under construction. A plan of the locations of the two structures is shown in Figure 13. A study was carried out to determine the effect of the two structures being constructed concurrently. The two foundations were modelled using the equivalent raft technique in Vdisp and the construction staging was estimated as presented in Table 5. A cross-section of the estimated settlement of the foundations of both structures is shown in Figure 14.

As construction of the Marina 23 superstructure is significantly ahead of the Pentominium structure, a large amount of settlement will occur before the superstructure construction of the latter commences. Therefore in terms of tilt of the structure, Pentominium may tilt towards Marina 23, however, this can be addressed during the construction of Pentominium. The Marina 23 tower does not tilt towards Pentominium and the differential settlement for both tower foundations is significantly reduced at the boundary.

It should be noted that a number of simplifications have been made during the assessment as the stiffness of rafts and superstructure were not included in the analysis. The bored pile shoring wall which has been installed at the boundary of the Pentominium and Marina 23 sites to enable excavation of the basement levels has also not been included in the analysis. These elements will assist in reducing the differential settlement.

### Final pile design

The final pile design was determined based on an assessment of all the ground investigation and preliminary pile data as well as all the results from the equivalent raft analysis, the standard pile group analysis and the geotechnical and structural finite-element analysis models. From the preliminary design carried out, the maximum pile axial working loads were 36 000 kN and the proposed pile lengths were 56 m long. However, as a result of the detailed structural and geotechnical design through assessment of the geotechnical parameters and finite-element analyses, the working load was reduced to 32 500 kN. In addition,

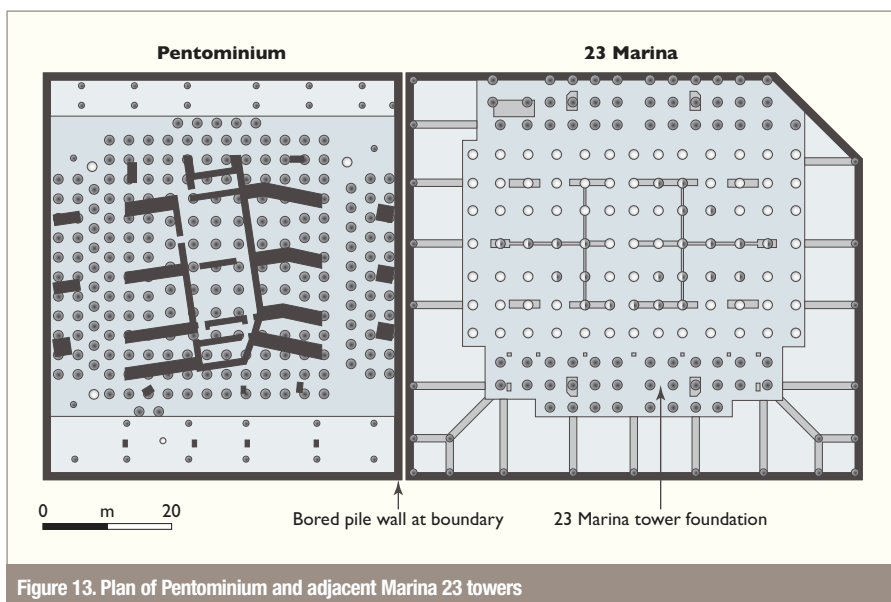


Table 5. Construction staging of Pentominium and Marina 23 towers

Stage	Pentominium progress	23 Marina progress
1	–	3 basement levels and 3 storeys
2	Raft complete	48 storeys above basement
3	6 basement levels	69 storeys above basement
4	29 storeys above basement	90 storeys above basement (construction complete)
5	52 storeys above basement	
6	74 storeys above basement	
7	97 storeys above basement	
8	120 storeys above basement (construction complete)	

ultimate skin friction values were increased based on the results from the preliminary pile testing. From a combination of these two factors, the pile lengths could be decreased to 42 m. The final pile design details are presented in Table 6.

Monitoring of the structure will be carried out during construction (Figure 15) such that the design assumptions used can be verified and the experience can be used by future designers to ensure the optimisation of the design for such prestigious structures in the region.

## Conclusion

A substantial amount of testing has been carried out for the design of the Pentominium tower foundation, which has included soil and rock testing as well as a comprehensive pile-testing programme.

In addition, complex geotechnical finite-element analysis has been carried out, which has been validated using standard geotechnical calculation techniques. The application of such testing and analysis approach has resulted in a cost-effective

and optimised foundation design solution.

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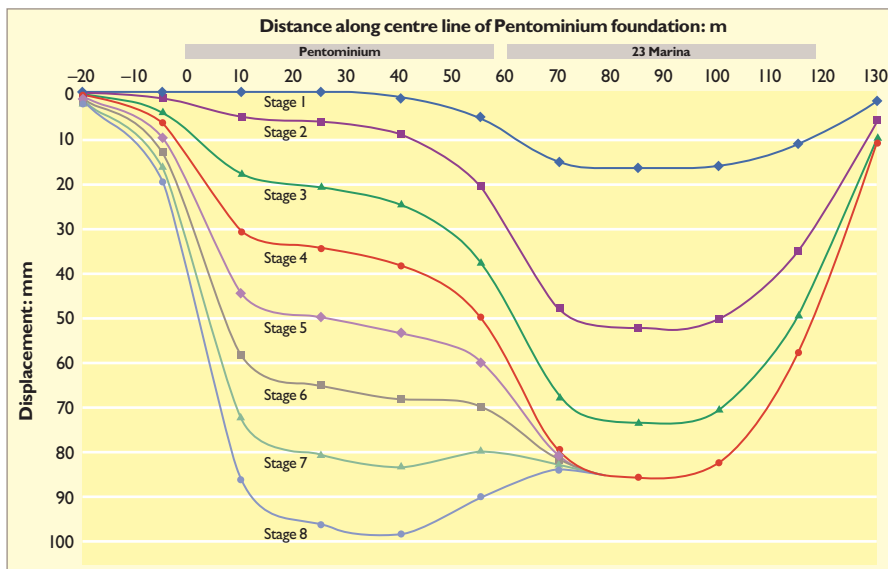


Figure 14. Predicted settlement of Pentominium and Marina 23 towers during construction (see Table 5 for construction stages)



Figure 15. Excavation under way in July 2009, showing proximity to adjacent Marina 23 building (far right) (Imersolt.com)

Table 6. Pile design configuration

Pile type	Pile diameter: m	Pile compressive working load: kN	Pile moments: kNm	Pile shear force: kN	Pile embedded length: m
A	1.5	32 500	3 500	400	42
B	1.5	26 000	3 500	400	36
D	1.2	18 000	1 500	100	32
E	1.2	18 000	1 500	100	32

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