# Deep foundation systems of ultra high-rise buildings: the Entisar tower in Dubai

## Systèmes de fondation profonde pour des bâtiments de très grande hauteur : tour Entisar à Dubaï

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ABSTRACT: This paper presents the foundation system of Entisar Tower in Dubai. At over 500 m in height, it will be one of the tallest buildings in the world, and the second tallest in Dubai after Burj Khalifa. With a footprint of 60 m by 60 m, it is a very slender structure, inducing very high compression stresses on its base, and therefore requiring a specific deep foundation system, consisting of rectangular barrettes with high-performance concrete, up to 80 m in depth, embedded in soft rock. The paper details the development of both the design process and the challenges of foundation testing and construction. The general geotechnical context, the site investigation and the return of experience from previous Dubai projects are described. The 2D and 3D numerical modeling, used during the design and the review of the design, are specifically addressed. A particular emphasis is made on the various stages of soil structure interaction procedures and the iterative design process. In order to validate the design assumptions, several barrette tests with multi Osterberg cells have been performed. Finally, the monitoring program and the progress of the ongoing construction works are presented.

RÉSUMÉ : Cet article présente le système de fondations de la tour Entisar à Dubaï. Avec plus de 500 m de hauteur, ce sera l'un des bâtiments plus hauts du monde et le second plus haut à Dubaï, après le Burj Khalifa. Avec une emprise au sol de 60 m par 60 m, il s'agit d'une structure très élancée, induisant des contraintes de compression très élevées sur sa base et donc nécessitant un système spécifique de fondations profondes, composé de barrettes rectangulaires en béton haute performance, jusqu'à 80 m de profondeur, ancrées dans du rocher tendre. L'article précise le développement tant du processus de conception comme des défis liés aux essais de chargement et à la construction. Le contexte géotechnique général, la reconnaissance géotechnique et le retour d'expérience de projets environnant à Dubaï sont abordés. Les modélisations numériques 2D et 3D, utilisées pendant la conception et les réévaluations de la conception, sont spécifiquement abordées. Une attention particulière est portée à la présentation des différentes étapes des procédures d'interaction sol-structure et du processus de conception itérative. De façon à valider les hypothèses de conception, plusieurs essais de barrettes avec des multiples cellules Osterberg ont été implémentés. Finalement, les dispositifs d'auscultation et l'état d'avancement de la construction seront présentés.

KEYWORDS: Barrettes, Deep Foundations, Ultra High Rise Buildings, Dubai, Load testing.

## 1 INTRODUCTION

Entisar tower, currently under construction, will exceed 500m in height and is expected to constitute a landmark feature in Dubai's skyline.

Soletanche-Bachy was in charge of the design, testing and construction of the foundations. This paper describes the main aspects of the project, focusing on the design challenges encountered during the process.

## 2 GEOTECHNICAL ASPECTS

The geological context is typical of Dubai, having been influenced by the deposition of marine sediments associated with sea level changes during relatively recent geological time, namely Quaternary and Pleistocene (Poulos, 2009). At the site's location, an upper layer of carbonate sands (13m in depth) is followed by several layers of weak rocks - calcarenite, sandstone, conglomerate and calcisiltite.

The geotechnical investigation and testing program included a total of 12 rotary boreholes, drilled down to depths up to 160m. In situ testing included pressuremeter testing, cross-hole geophysical testing, and also SPT tests for the sand layers. Laboratory testing included conventional testing (densities, UCS, among others) but also specialized tests: Constant Normal Stiffness tests of concrete-rock interfaces (both static and cyclic) and triaxial testing on Bender element-equipped samples.

The rock layers were modelled as Tresca materials, with shear strength representing a bond yield strength, below which the ground is deemed to behave in an essentially elastic manner (Haberfield, 2008). Once past this threshold, plastic deformation is expected to increase severely. As such, the design limited shear stress levels to below Tresca shear strength.

Design shear strength was defined as half of the unconfined compressive strength test results (UCS/2) and also on triaxial test results, both showing good agreement. However, estimates of shear strength based on pressuremeter plots often led to higher results, possibly due to lower disturbance induced by *in situ* testing. Shear strength assumptions, based as they were on lab testing, may therefore be conservative. Table 1 shows the model and parameters adopted for design.

Table 1. Stratigraphy and selected parameters.

| Code | Stratum | Top Level | Thck. | UCS | $E_{LT}$ | $q_s$ |
|---|---|---|---|---|---|---|
| | | [mDMD] | [m] | [MPa] | [MPa] | [kPa] |
| L1-3 | Sand | +3.00 | 13.00 | - | 97-169 | - |
| L4 | Calcarenite | -10.00 | 2.00 | 2.00 | 303 | 354 |
| L5 | Sandstone | -12.00 | 23.00 | 0.81 | 550 | 225 |
| L6 | Calcisiltite | -35.00 | 40.00 | 2.00 | 1 199 | 400 |
| L7 | Calcisiltite / Siltstone | -75.00 | 42.00 | 3.00 | 1 712 | 450 |
| L8 | Mudstone | -117.00 | - | 3.50 | 1 712 | - |

Due to their length, most of the barrette's capacity is provided by skin friction. Ultimate skin friction ($q_s$) estimates were supported by recent Dubai practice (Alrifai, 2007), based on the following equation for rock-socketed piers (Horvath and Kenney, 1980):

$$q_s=0.25UCS^{0.5} \text{ [MPa]} \quad (1)$$

These values will be discussed in the following chapters.

## 3 FOUNDATION SYSTEM

The tower's slenderness (see Figure 1) and weight (more than 7000MN) imply very high loads acting on a relatively small area, resulting in an average pressure of approximately 1800kPa at raft level. The strength and deformability of the weak rock layers lead to the use of a raft supported by an indirect foundation system.

Figure 1. Entisar Tower illustrations.

From the beginning, the client excluded the use of a piled-raft system. However, the base of the raft sits at 11m depth, on a sand layer. Due to the low stiffness of the sand compared to the rock layers supporting the deep foundations, the benefit of a piled raft foundation is deemed to be relatively small.

The solution proposed by Soletanche-Bachy (SB) consisted of barrettes, drilled with Hydrofraise rigs from surface level. In addition to high unitary capacities (large cross section and high strength concrete allowing up to 80MN service loads), the accuracy of the verticality controls allows a high concentration of foundation elements.

The structure of the tower includes a central core, which resists approximately 2/3 of the total vertical loads, and peripheral outriggers, which play an important role in resisting lateral forces. The foundation system (see Figure 2) was established with the objective of keeping the load paths as direct as possible. Consequently, it comprises a large number of barrettes clustered below the core structure, resisting most of the gravity loads, and a peripheral ring of barrettes, with a large lever arm resisting loads induced by lateral forces. Due to the tower's intrinsic Northward imbalance (the tower footprint reduces progressively in height, to make place for terraces, etc.), a larger concentration of foundation elements is located in the Northern corner.

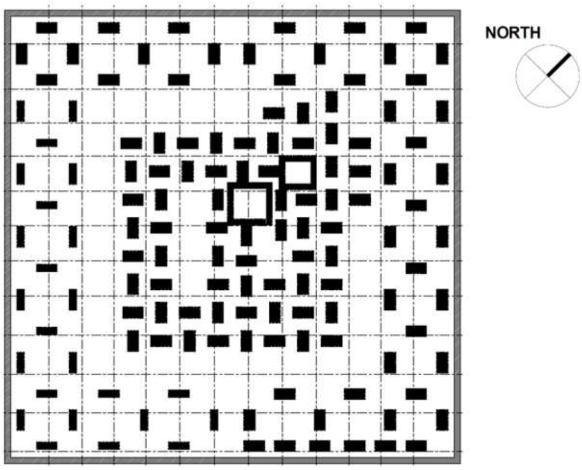


Figure 2. Barrette foundation layout.

## 4 SOIL-STRUCTURE INTERACTION

Soil-structure interaction is an important aspect in the behaviour of Ultra High Rise Buildings, requiring an iterative procedure to perform the design. As such, an initial foundation stiffness is provided by the foundation designers (SB). This stiffness is used for structural calculations, from which foundation loads are obtained. These loads are then used by SB in order to obtain revised stiffness values of the foundations. The process is continued until convergence is obtained.

Importantly, different stiffness values were derived for different load scenarios. Two different cases are to be considered: on one hand, gravity loads, basically the weight of the building and the surcharges; on the other hand, lateral loads, due to wind and earthquake actions. The strain levels induced by these actions are significantly different and, since strain levels have a marked effect on stiffness, different moduli were assigned to each scenario.

The initial loading moduli obtained from pressuremeter testing presented mean values between 500 and 900MPa, depending on the layers. These, however, correspond to large strain moduli (strain generally well above 1%) and therefore are not representative of the foundation behaviour, both for gravity or lateral loads.

As such, stiffness estimates were based mainly on cross-hole testing. Regarding gravity loading, representative Young's moduli $E_{LT}$ were defined as $0.2E_0$ ($E_0$ being small strain moduli from geophysical testing). This ratio is in accordance with previous Dubai literature (Poulos, 2009 and Haberfield, 2008). Taking into account representative stiffness degradation curves for soft rocks (Thompson and Leach, 1985), it is noticeable that $E=0.2E_0$ corresponds to strain levels in the order of $10^{-3}$ (see Figure 3). The compatibility of these strain levels was found to be adequate in the numerical models developed subsequently.

Concerning lateral load cases, stiffness values were set between $0.4E_0$ and $1.0E_0$, depending on actual strain levels, based on the stiffness degradation curves cited above. In all cases, sensitivity analysis were performed in order to assess impact of ground stiffness variations on structural elements.

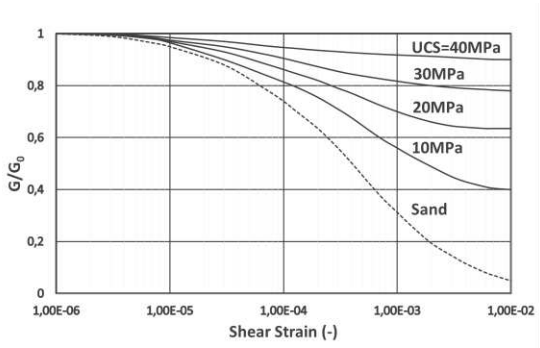


Figure 3. Stiffness degradation curves (Thompson and Leach, 1985)

## 5 NUMERICAL MODELLING

The assessment of the soil structure interaction phenomena required the use of different numerical models.

In order to allow the implementation of multiple geometry changes, sensitivity analysis and load changes, relatively simple 2D-based models were preferred during the initial stages of the project. These were established using PLAXIS 2D. Taking into account the shape of the foundation (broadly consisting on homothetic squares), it was decided to model the foundation under vertical forces by means of an Axisymmetric model. As such, the barrettes in the foundation were transformed into a series of concentric rings. These rings being modelled with volumetric soil elements, the thickness of each ring was adjusted so as to have the correct vertical stiffness (Figure 4). This is a simplified assumption, moreover considering the slight asymmetry of load and barrette distribution, but was deemed to be acceptable as a simplified analysis.

Regarding lateral actions, the most important effect concerned the global bending moments at the base of the raft. In order to model these effects in a 2D model, it was decided to adopt a plane strain 2D model. In this case, the barrettes in the foundation were transformed in a series of parallel rows. As previously, the thickness of each row was adjusted in order to match the vertical stiffness (Figure 4).

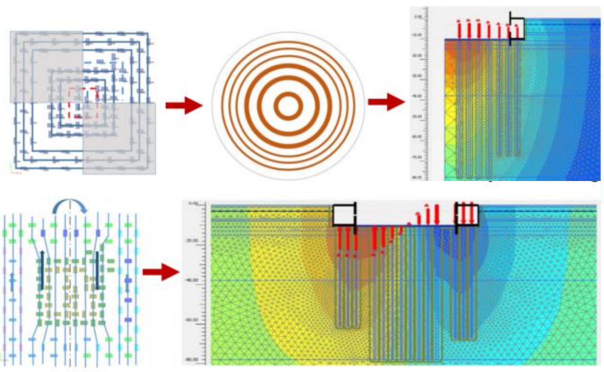

Figure 4. Conceptual 2D-based FEM models for Gravity Axisymmetric (above) and Lateral Plane Strain (below).

A full PLAXIS 3D model was later developed by Setec Terrasol, so as to validate the results obtained with the 2D-based analysis. The results from the Gravity Axisymmetric models results were very close to the results from the 3D model (for example, maximum settlements were estimated as 70 and 68mm, respectively for PLAXIS 2D and 3D analyses).

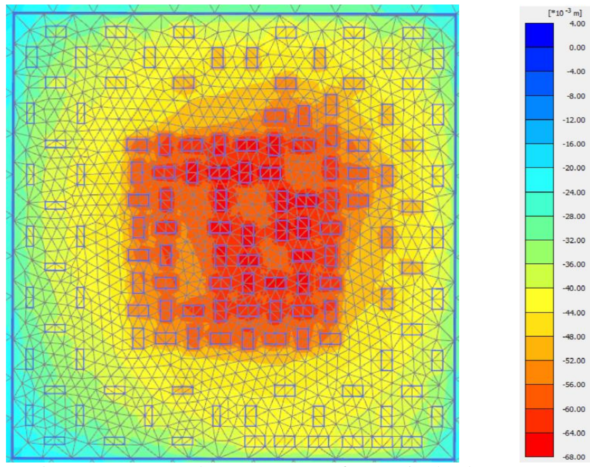

Figure 5. PLAXIS 3D settlement contours for gravity load case.

Regarding the Lateral Plane Strain models, these were shown to underestimate the stiffness of the foundation, which in all the 3D calculations was approximately 40% larger. Since this 40% difference was systematically found, it was assumed that the Lateral Plane Strain stiffness, if multiplied by a factor of 1.40, was adequately representative of the foundation behaviour. Lateral Plane Strain models were thus corrected and continued to be used.

The foundation was also modelled with the software PIGS (Pile Group Settlement). This code (Poulos, 2008) uses a simplified approach to compute the settlements both within and outside pile groups subjected to vertical loading. The results show slightly larger maximum settlement values (74mm in PIGS vs 68mm in PLAXIS 3D, see Figure 6 and Figure 5 respectively) and smaller minimum values, but globally results show good agreement with both PLAXIS analysis.

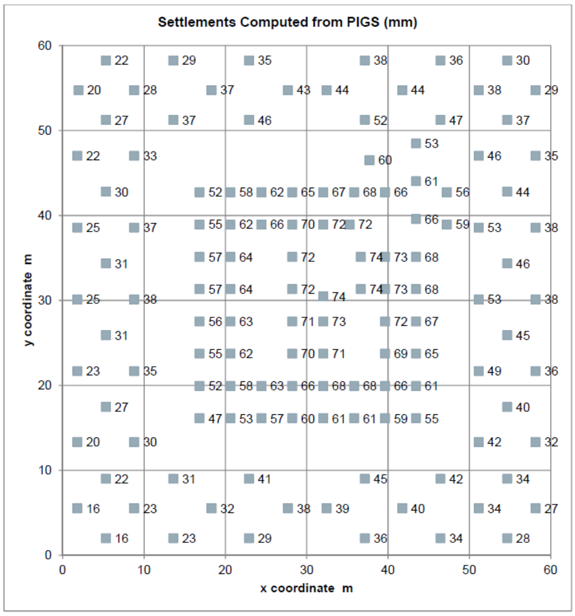


Figure 6. PIGS settlement estimates for gravity load case.

Numerical models were also valuable to estimate stress levels in the vicinity of the barrettes, especially in the barrettes clustered closer together in the core. The analysis led to the conclusion that, except for localized areas in the weaker Sandstone layer, or close to the barrette base, shear stress levels were below 60% of ultimate. This means that the response of the system is expected to be broadly elastic for the expected load levels.

## 6 GROUP EFFECT ASSESSMENT

Group effects in dense foundation networks are often important. In the Entisar project, the effects were analysed regarding both stiffness and capacity.
Group effect on stiffness was assessed via the FEM analyses presented previously. While an isolated barrette displays an estimated stiffness in the order of 4GN/m under gravity loading, the same barrette while part of the foundation shows a stiffness lower than 1GN/m.

Regarding group effect on capacity, it was checked whether the sum of individual barrette resistances was smaller than the resistance of a block or row comprising several barrettes, in order to comply with the equation below (see Eq. 2):

$$R_{group,c,d}=\min(R_{sum,c,d};R_{block,c,d};R_{row,c,d}) \quad (2)$$

For all the cases tested (several block failure modes were assessed, so as to cover the plausible failure scenarios), individual barrette capacity was found to govern the design. This was due to the barrette-rock interface resistance, which is significantly lower than the rock shear strength mobilized in a block failure mode. In addition, the end bearing of the block surfaces is quite large, further compounding block capacity.

## 7 OSTERBERG LOAD TESTS

So as to validate the design assumptions for skin friction (and also end bearing), four load tests were executed.

Preliminary load tests, performed on two 2800x1000mm$^2$ barrettes, were established in order to achieve skin friction up to 3.0 times larger than the estimated serviceability values. The only practical solution to achieve the high load tests magnitude, was to perform 2levels of O-Cells tests “in series” (see Figure 7). TB02 was concreted from -14mDMD to -51mDMD and TB01 from -41mDMD to -72mDMD (mobilizing respectively 160 and 120MN).

In the upper materials of layer L5, the skin friction values obtained with Eq. 1 showed good agreement with the Osterberg results, and also with the results of CNS testing at equivalent depths. In layer L6, however, larger values in the order of 700kPa were attained in barrette TB01. Design values of layer L6 were optimized, taking into account the load tests and all remaining information.

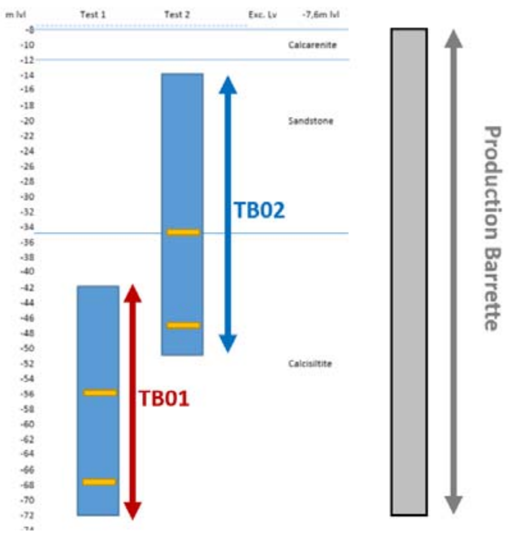


Figure 7. Osterberg-cell preliminary load tests: schematic layout.

Working load tests were implemented on two foundation barrettes. The purpose was to assess the behaviour of the actual barrette geometry (2800x1500mm$^2$ barrettes; from -14mDMD to -72mDMD).

WTBs were tested to 1.5 times the service load (one O-Cell level only, test up to 110MN). The tests showed linear behaviour, signalling a significant reserve capacity at service load levels. Stiffness estimates for equivalent top-down loads were obtained via different methods (namely t-z curves and PLAXIS 2D) and the results showed reasonable agreement with equivalent top-down curves using design parameters.

## 8 LATEST DEVELOPMENTS

In order to monitor foundation behaviour, strain gages were installed in several barrettes. These will be complemented with other systems: pressure cells under the raft, settlement tell tales and topographic controls. Results will thus allow to compare the calculated and actual behaviour of the foundation. In parallel, minor adjustments to superstructure definition and loads are expected during construction.

## 9 CONCLUSION

This paper has described the design processes of the foundations for the Entisar Tower in Dubai.

Soil-structure interaction phenomena were found to have a significant impact in the design. Finite element analyses were carried out to support the structural design, and to obtain settlement and capacity estimates.

Skin friction values in load testing matched Dubai practice for upper layers, but larger skin friction values were mobilized in deeper layers, leading to possible optimizations. Top-load stiffness derived from working test barrettes reasonably agrees with design assumptions.

Construction is ongoing, and predicted versus actual performance will be analyzed in the future.

## 10 ACKNOWLEDGEMENTS

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