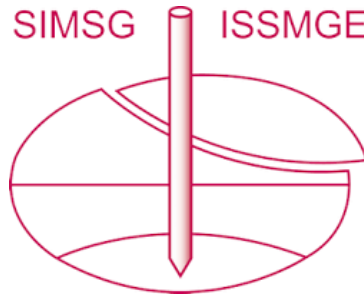


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Bi-directional static load tests in the context of sustainable foundation design – A case study, Dubai, U.A.E.

Essais de chargement statique bidirectionnel dans le contexte de conception de fondations durables – Étude de cas, Dubai, U.A.E.

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ABSTRACT: The design of rock-socketed piles in Dubai is based on several empirical correlations between shaft capacity and the unconfined compressive strength of the rock. Pile load tests have consistently shown that current design methods underestimate shaft capacities, however such observations are generally not reflected in local design practice. Considering the demand for sustainable foundation design it is essential that actual foundation performance is considered in the design process. In this context, bi-directional load tests have proven to be an invaluable tool to overcome design limitations and value engineer foundation design, reducing construction costs and use of natural resources. A case study is presented in which an improved design method was adopted and validated by a programme of bi-directional load tests on instrumented test piles. It was demonstrated that the correlation proposed by Rowe & Armitage (1987) for rough sockets may be adopted in Dubai to design rock-socketed piles when validated by preliminary load tests. The proposed design approach allowed the foundation system to be value engineered, resulting in an overall reduction in pile lengths of about 50% from the original tender design quantities, developed under traditional design methodologies.

RÉSUMÉ : La conception des pieux d'ancrage dans les roches à Dubaï est basée sur plusieurs corrélations empiriques entre capacité de résistance aux frottements latéraux des pieux et la résistance à la compression uniaxial des roches. Les résultats d'essais de chargement sur pieux instrumentés ont montré que les méthodes de calcul actuelles sous-estiment la capacité de résistance des pieux, mais ne se reflètent généralement pas dans les pratiques de conception locales. Compte tenu de la demande de conception de fondations durables, il est essentiel que les performances des fondations soient prises en compte dans le processus de conception. Dans ce contexte, les essais de chargement bidirectionnel se sont avérés être un outil inestimable pour surmonter les lacunes de conception, valoriser la conception des fondations, réduire les coûts de construction et minimiser l'utilisation des ressources naturelles. Une étude de cas est présentée dans laquelle une méthode de calcul améliorée a été adoptée et validée par un programme d'essais de chargement bidirectionnel sur pieux instrumentés. Il a été démontré que la corrélation proposée par Rowe & Armitage (1987) pour les pieux à emboîture rugueuse peut être adoptée pour la conception des pieux lorsqu'elle est validée par des essais de chargement préliminaires. L'approche de calcul proposée a permis d'optimiser le système de fondation, ce qui a entraîné une réduction des longueurs de pieux d'environ 50% par rapport aux quantités de conception de l'appel d'offres d'origine, développées selon les méthodologies de calcul traditionnelles.

KEYWORDS: Rock-socketed piles, bi-directional load tests, instrumentation, value engineering, sustainability.

1 INTRODUCTION

In recent decades, Dubai has seen a significant increase in the construction of high-rise buildings, which require the development of efficient foundation systems. To incorporate sustainability in foundation design it is critical that local design practice is continuously improved by considering observed foundation performance in the proposed foundation solutions.

When assessing the performance of heavily loaded rock-socketed piles, bi-directional static load testing has increasingly become the preferred method providing a safe and cost-effective load testing solution.

This paper presents a case study in which preliminary pile test data from numerous project sites across Dubai was reviewed and incorporated into the foundation design process allowing for an improved design method to be adopted and subsequently validated by a series of bi-directional static load tests on preliminary and working test piles.

Based on the outcomes of this case study, a unified, robust and sustainable approach is recommended as basis of design for rock-socketed piles in the Emirate of Dubai.

2 CASE STUDY: ADDRESS BEACH RESORT, DUBAI, U.A.E.

The project consists of a mixed-use 77 storey twin-tower development. The towers are just over 300m high with three common podium levels and three below ground basement levels. The completed structure is shown in Figure 1.



Figure 1. Completed twin-tower development.

Since its completion in 2020 the project has received numerous awards, currently holding the Guinness World Record for the highest outdoor infinity pool in a building located on the 77th floor at 293.90m and the highest occupiable skybridge floor at 294.36m.

2.1 Site-specific ground investigation

This case study highlights several shortcomings in local design practice. One of the most significant limitations is the fact that geotechnical interpretative reports are produced by ground investigation contractors often lacking the required relevant experience to undertake detailed geotechnical design and provide efficient, project specific foundation design recommendations.

At concept design stage WSP was instructed to review the interpretative report available for the project site. Upon review of the pile design recommendations, it was concluded that the required pile capacities were not achievable for the heavily loaded areas within the footprint of the towers. However, based on the authors' local project experience, the recommendations provided in the report were considered overly conservative. This is further discussed in section 2.3.1 and illustrated in Figure 5.

Consequently, an additional site-specific ground investigation was specified, tendered, and awarded to an experienced ground investigation contractor. The scope of works comprised the following:

- Five rotary boreholes reaching maximum depths of 80m below ground level.
- OYO Elastometer pressuremeter testing undertaken at 2.5m intervals in the boreholes located within the footprint of the towers.
- Down-hole seismic profiling to determine small strain stiffness and seismic site class.
- Cone penetration tests.
- In-situ permeability testing.
- Groundwater monitoring and testing, and
- Laboratory testing on representative soil and rock core samples.

The ground investigation layout plan is shown in Figure 2.

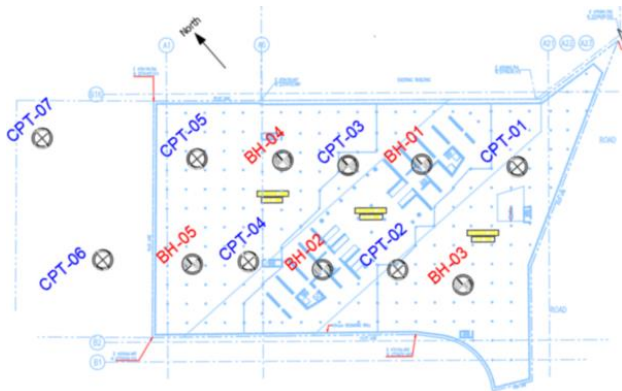


Figure 2. Site-specific ground investigation layout plan.

2.2 Ground conditions

The subsurface conditions encountered at the project site were consistent with several other sites across Dubai comprising an upper layer of carbonate sand underlain by the extremely weak to weak Sandstone/ Calcarenites belonging to the Ghayathi Formation. These strongly carbonated sandstones overlay the Barzaman Formation consisting of alternating layers of weak Conglomerate and Calcsiltite. The ground conditions are summarised in Table 1 along with relevant design parameters recommended by the ground investigation contractor upon completion of the additional site-specific ground investigation.

Table 1. Subsurface conditions and geotechnical design parameters.

Unit	Top Level, mDMD	Thick., m	UCS, MPa	f_{su} , MPa	E_d , MPa
Overburden soils	+6.0	15	-	-	25-60
Calcarenite/ Sandstone	-9.0	22	0.70–1.15	0.28–0.35	110-200
Conglomerate / Calcsiltite	-31	> 40	1.15–4.0	0.35–0.66	300

f_{su} - Ultimate Unit Side Resistance; E_d - Drained Young's Modulus

2.3 Foundation design strategy

In this section the foundation design strategy adopted for this development is described emphasising the improvements made with regards to the design of axially loaded rock-socketed piles. Other aspects of foundation design, such as pile group settlement analysis are beyond the scope of this paper, however pile group performance throughout construction will be briefly addressed in the final section.

The foundation system adopted for the twin towers consists of a conventional pile group comprising a 3000mm thick raft and 1500mm bored concrete piles installed at 3.75m centres carrying maximum axial compressive loads of 35.3MN. The foundation layout is shown in Figure 3. This is a typical arrangement adopted in Dubai for high-rise schemes since local authorities typically do not allow for load transfer from the raft to the underlying strata to be considered.

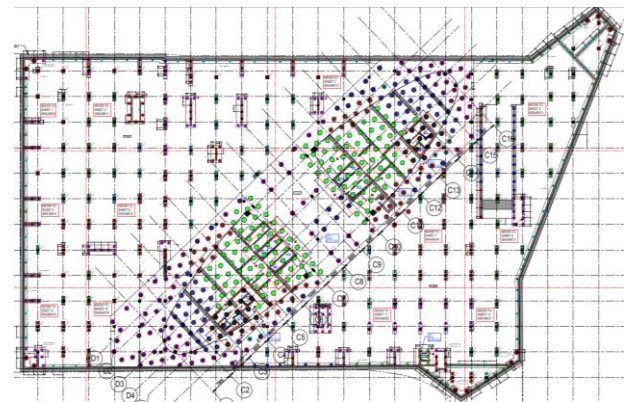


Figure 3. Pile layout – twin towers and podium.

The design strategy adopted consisted of the following stages:

- **Stage 1 - Prior to preliminary load tests (PLTs)**
A review of a large database of bi-directional load test data was undertaken enabling the design of test piles to be optimised.
- **Stage 2 - Following the completion of PLTs**
A total of six single- and multi-level bi-directional static load tests were carried out on instrumented test piles. Load test data was used to validate the design carried out in Stage 1 and further value engineer the foundation design.
- **Stage 3 - During construction with testing on production piles**
This stage was essential to validate the final design established in Stage 2 and subsequently adopted for the production piles.

2.3.1 Stage 1: Test piles design optimisation

Except for infrastructure projects under Dubai's Roads and Transport Authority (RTA) required to be Eurocode compliant, geotechnical design of rock-socketed piles across Dubai is carried out in accordance with BS8004:1986 Code of Practice for Foundations. Consequently, a working stress design approach is adopted with a global factor of safety of 2.5 applied to the ultimate shaft capacity of the pile. The contributions of end bearing and overburden soils to the pile capacity are typically ignored in local design practice, particularly in the context of heavily loaded, large diameter rock-socketed piles.

Although still currently adopted by Dubai Municipality, BS8004:1986 was withdrawn and superseded by BS8004:2015 being now fully compatible with the Eurocodes. At the time of writing, it is unclear when a full transition to a limit state design approach will occur in Dubai. However, foundation design would greatly benefit from such an approach, considering the potential reduction in the partial factors applied to pile design associated with the local requirement to undertake static load testing on at least 1% of production piles and with the fact that for most of the schemes preliminary static load tests are indeed specified and carried out.

Since local design regulations do not prescribe a specific calculation method, numerous empirical correlations are currently adopted to estimate ultimate pile capacities from the unconfined compressive strength (UCS) of the rock. The lack of a consistent design approach is a major shortcoming in local practice since, as highlighted in section 2.1, the quality of design recommendations is heavily dependent on the technical capabilities of the ground investigation contractor.

In addition to the variety of calculation methods locally adopted, data from over 40 bi-directional load tests on instrumented preliminary test piles completed across Dubai have consistently shown that current design methodologies significantly underestimate the ultimate side resistance of rock-socketed piles. Test site locations are indicated in Figure 4.



Figure 4. Relevant project locations across Dubai.

The conservatism of current calculation methods is highlighted in Figure 5 in which the results from all bi-directional static load tests are presented in terms of mobilised unit side resistance against the UCS of the rock. The most common design methods adopted in local practice are also presented, which define a design envelope limited by the following empirical correlations:

$$\text{Lower bound: } f_{su} = 0.15 * UCS \text{ to } 0.2 * \sqrt{UCS} \text{ [MPa]} \quad (1)$$

$$\text{Upper bound: } f_{su} = 0.33 * \sqrt{UCS} \text{ [MPa]} \quad (2)$$

As discussed in section 2.1, the ultimate side resistance values provided in the report reviewed at concept design stage are also presented in Figure 5. These conservatively plot below the typical design envelope and are significantly lower than the

values recommended by the ground investigation contractor commissioned to undertake the additional ground investigation at the project site (Table 1); further highlighting the requirement for a thorough review of the technical capabilities of ground investigation contractors at tender stage.

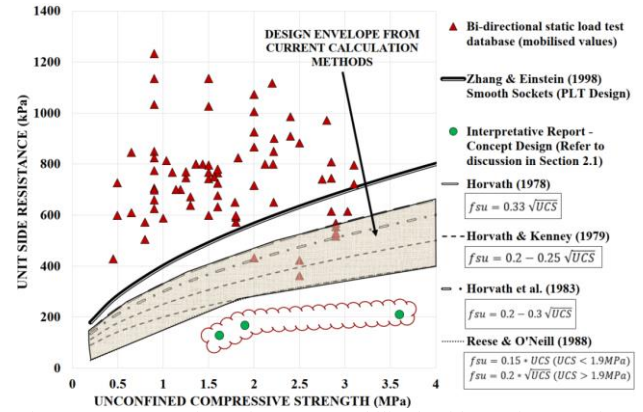


Figure 5. Comparison between mobilised side resistance from preliminary load tests and envelope of predicted ultimate values.

Even though all preliminary test piles were loaded to 250 – 300% of the specified working load, a consistent observation across all project sites was that none reached geotechnical failure and the mobilised side resistance at maximum test loads significantly exceeded the values predicted by the various design methodologies currently adopted in local design practice. Additionally, as shown in Figure 6, the linearity of the equivalent top-down load-displacement curves also provides strong evidence that the ultimate side resistance was not mobilised and test loads were primarily carried by the pile shaft, which is in agreement with typical design assumptions.

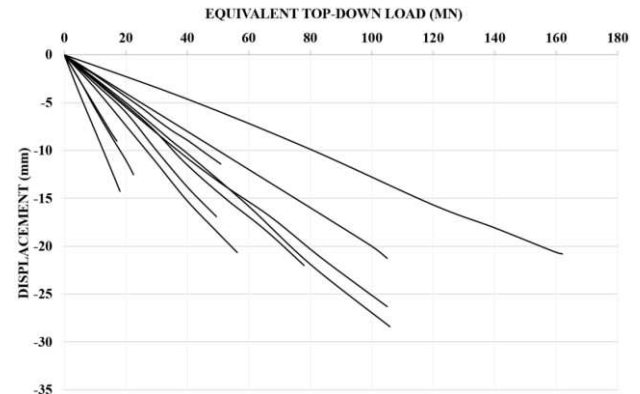


Figure 6. Typical equivalent top-down displacement curves from reference test sites.

Considering these observations, a review of published empirical methods for determining ultimate unit side resistance of rock-socketed piles in weak rocks was undertaken. It was found that the solution proposed by Zhang & Einstein (1998) for smooth rock sockets provided a reasonable lower bound estimate of mobilised values. This empirical correlation is also shown in Figure 5. According to this approach the ultimate unit side resistance is given by:

$$f_{su} = 0.4 * \sqrt{UCS} \text{ [MPa]} \quad (3)$$

This improved calculation method was incorporated in the geotechnical interpretative report produced by the ground investigation contractor and subsequently approved by Dubai Municipality.

This was the first milestone in the foundation design optimisation process allowing for the preliminary test piles to be value engineered considering previous project experience and available bi-directional load test data.

2.3.2 Stage 2: Design optimisation following completion of preliminary load tests.

To validate the design optimisation undertaken in Stage 1, a comprehensive pile load testing programme was specified and carried out at the project site. As summarised in Table 2, the pile testing programme comprised six bi-directional load tests with single- and multi-level load cell arrangements loaded to 250 – 300% of the specified working load. The procedure for multi-cyclic load tests on preliminary compression piles outlined in Clause B17.13.3 of the ICE Specification for piling and embedded retaining walls (SPERW, 2017) was adopted.

Table 2. Summary of preliminary static load testing programme.

PTP No.	Pile dia. (m)	Pile cut-off level (mDMD)	Pile length (m)	Working Load (MN)	Test load (MN)
1 ¹⁾	1.5	-6.5	45.0	35.3	105.9
2 ¹⁾	1.5	-6.5	35.0	25.0	75.0
3 ²⁾	1.5	-6.5	45.0	35.3	88.25
4 ²⁾	1.5	-6.5	35.0	25.0	75.0
5 ²⁾	0.9	-5.0	20.0	6.45	19.35
6 ²⁾	0.75	-5.0	17.0	4.6	13.8

¹⁾ Multi-level load cell arrangement
²⁾ Single-level load cell arrangement

Test piles were constructed under polymer support fluid and loaded once the concrete reached the required design strength. Caliper testing was carried out in all test piles prior to concreting and cross-hole sonic logging undertaken to assess its structural integrity, with no irregularities being detected.

Each test pile was fully instrumented with up to eleven levels of vibrating wire strain gauges (Geokon Model 4991 Series) positioned at various levels to enable the calculation of load distribution along the pile shaft. The strain gauges were installed in sets of four per level, equally spaced around the perimeter of the pile to allow for some redundancy for damaged or malfunctioning gauges. A schematic section showing the test pile setup in relation to the subsurface conditions is given in Figure 7.

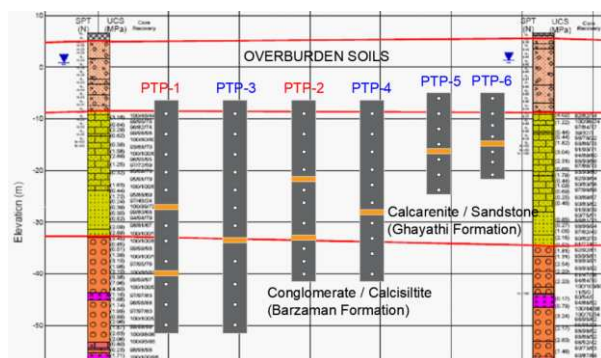


Figure 7. Bi-directional preliminary test piles configuration.

The displacements of the bi-directional load cells were measured using tell-tale rod extensometers with displacement transducers with the pile head movement being directly measured at the top using displacement transducers. The

hydraulic pressure applied to the bi-directional load cells was measured using a high-pressure Bourbon gauge and a pressure transducer. Throughout the test, the displacements, load and strain data were automatically recorded at 1-minute intervals.

The introduction of multiple load cells at different levels was intentional with the purpose to overcome the limitations of the single-level test setup observed in previous projects. As shown in Figure 8, there is a noticeable reduction in load transfer with distance to the load cells. Consequently, with a single level of load cells very low side resistance is mobilised in sections of the rock socket closer to the pile head and pile toe levels, which is a limitation for design optimisation. Latapie (2019) also reports a significant reduction in mobilised side resistance at a distance beyond 2.5 - 5m from the load cell assembly.

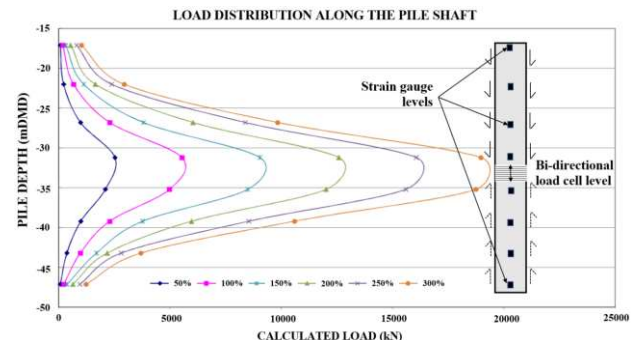


Figure 8. Typical load distribution in a bi-directional load test with single-level arrangement.

On the other hand, as presented in Figure 9, adopting a multi-level test setup at this project site allowed for higher loads to be transferred to different segments of the rock socket enabling the mobilisation of comparatively higher side resistance.

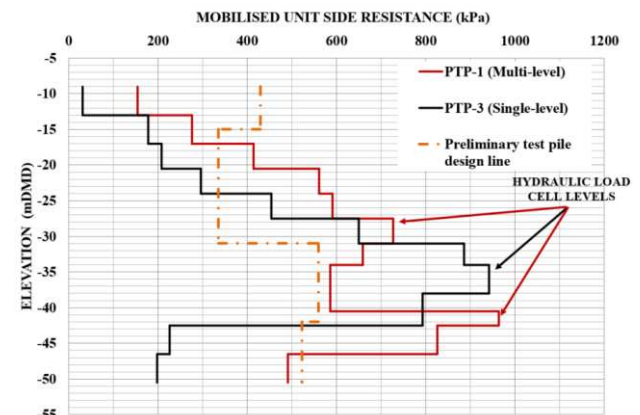


Figure 9. Mobilised unit side resistance at maximum test load for single- and multi-level load cell setup.

Even though the test piles were designed with an improved methodology, the mobilised unit side resistance curves shown in Figure 10 indicate that the ultimate side resistance was not achieved in any of the preliminary test piles at maximum test load. For the reasons discussed earlier only values mobilised in the pile section between the jack assembly and the nearest level of strain gauges are presented in Figure 10.

The stiff pile response to axial loading is also noticeable in the equivalent top loaded load-displacement curves shown in Figure 11. The load-displacement curves are essentially linear with pile behaviour mostly shaft controlled with negligible load transferred to the pile base. Additionally, it was observed that a significant proportion of the pile head displacements was the result of the elastic compression of the test piles.

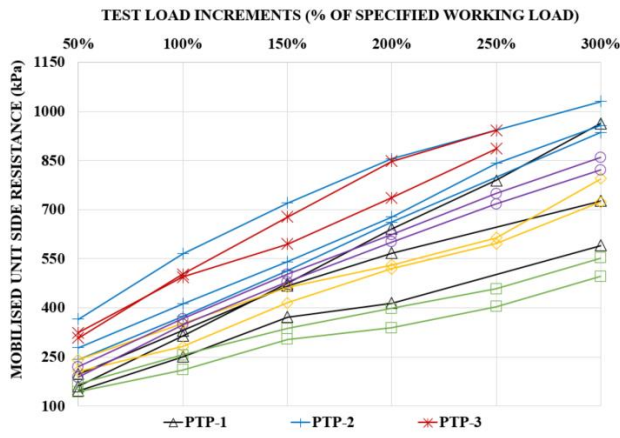


Figure 10. Mobilised unit side resistance against test load increment.

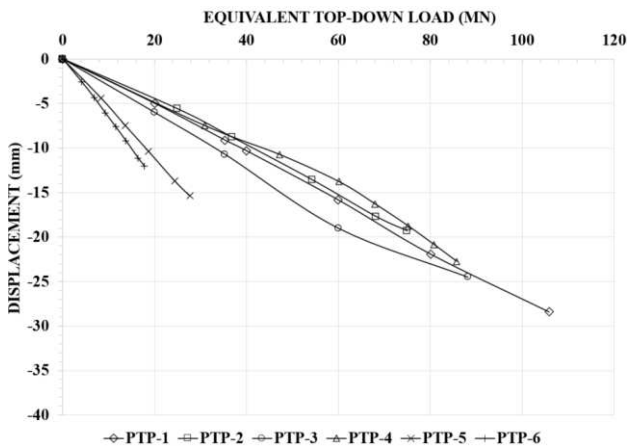


Figure 11. Equivalent top-down displacement curves from test piles.

The unit side resistance mobilised at maximum test loads plotted against the representative UCS value is presented in Figure 12. It is shown that for any given UCS value, the mobilised side resistance exceeded the values predicted by the empirical correlation proposed by Zhang & Einstein (1998) for smooth rock sockets used in the geotechnical design of the preliminary test piles, validating the design approach adopted in Stage 1.

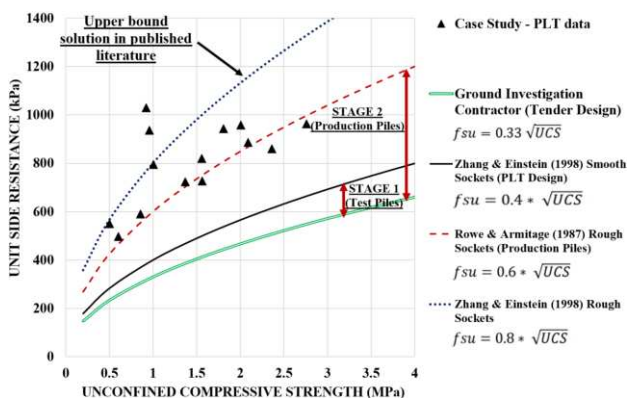


Figure 12. Mobilised unit side resistance against UCS.

Taking into consideration the mobilised values and the literature review undertaken during Stage 1, the final design adopted for the production piles was based on the correlation proposed by Rowe and Armitage (1987) for rough sockets, in which the ultimate unit side resistance is given by:

$$f_{su} = 0.6 * \sqrt{UCS} \text{ [MPa]} \quad (4)$$

The subsequent approval of the piling package by the Dubai Municipality was a critical project milestone setting an important precedent in local practice on the use of a more efficient design methodology for rock-socketed piles.

Preliminary pile test data presented herein also suggests that published design methods may not find direct application in the ground conditions encountered in Dubai, since ultimate side resistance was not mobilised and some of the test data plot above the upper bound solution in published literature, as shown in Figure 12. Therefore, there is the possibility to further improve design practice and develop a local design methodology for rock-socketed piles considering pile construction methods and prevalent ground conditions.

2.3.3 Stage 3: Foundation design verification

Production piles were subject to a rigorous testing regime, comprising caliper logging on 10% of piles; integrity testing on 100% of piles; cross-hole sonic logging on 15% of piles; dynamic load test on 5% of piles and static load compression test on 1 % of piles. The proof load test procedure defined in Clause B17.13 of the ICE SPERW (2017) was followed with the test piles loaded to 150% of specified working load. The test results are summarised in Table 3.

Table 3. Summary of Proof Load Test Results.

Pile Type	Pile dia. (m)	Pile Length (m)	Specified Test load, 150% WL (MN)	Settlement at 150% WL (mm)
P1	1.5	34.0	52.95	11.8
P2	1.5	34.0	52.95	12.3
P4	1.5	27.5	37.5	9.5
P5	0.9	14.0	9.7	4.5
P6	0.9	14.0	9.7	4.5
P10	1.5	14.0	18.75	3.3
P12	0.75	13.0	6.9	5.5
P13	0.75	13.0	6.9	5.4

The piles subjected to proof loading performed well with pile settlements at working loads found to be within the permissible values specified in local design regulations further validating the design solution adopted.

2.4 Pile group performance

Tower settlements were monitored throughout construction by means of settlement monitoring points installed across the basement slab at level 3. The instrumentation layout plan is shown in Figure 13.

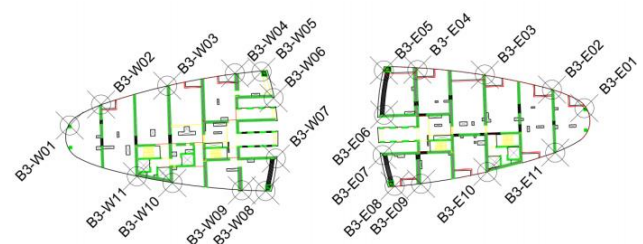


Figure 13. Basement 03 – settlement points.

Tower settlements upon topping out of the structure were less than 10mm suggesting a very stiff pile group settlement behaviour comparable to observations from other project sites.

Moreover, measured settlements are found to be significantly less than predictions based on the stiffness profile recommended by the ground investigation contractor derived from standard laboratory testing and initial loading phase from pressuremeter tests, therefore corresponding to large strain moduli in the range of 1 – 1.5% (refer to Table 1). This is another limitation in current design practice in which the strain-dependency of stiffness is largely ignored by local ground investigation contractors.

As shown in Figure 14, recommended stiffness values should be based on appropriate strain levels for the geotechnical structure under consideration. To that effect, a pile group settlement analysis under gravity loading was carried out considering a representative rock mass stiffness of $0.2E_0$ (E_0 being the small strain stiffness from *in situ* down-hole geophysical testing), which has been adopted by the authors in previous schemes and has also been mentioned in several publications as an appropriate stiffness for deformation analysis (Pereira, G. et al. 2017).

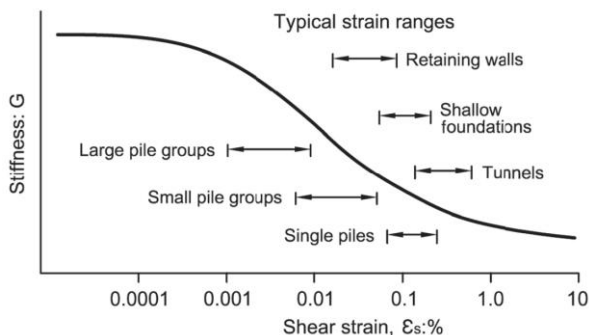


Figure 14. Influence of pile-group size on the selection of appropriate stiffness parameters, with large pile groups defined as groups with > 25 piles (O'Brien, A.S. 2017).

Taking into consideration previous project experience and measured tower settlements, this approach still provides conservative settlement estimates and may be adopted as a lower bound value in routine foundation design. Nevertheless, to further improve local design practice it is recommended that advanced laboratory testing is carried out to establish rock stiffness design profiles appropriate for the strain levels under consideration and to develop site specific degradation curves for local weak carbonate rocks.

Observed tower foundation settlement also appears to validate the work recently published by Alzaylaie (2017) and Latapie (2019) in which significantly higher rock mass stiffness values, up to 200 times higher than typically recommended by ground investigation contractors, were back-analysed from over 100 pile load tests carried out in Business Bay and Downtown Dubai.

3 CONCLUSIONS

Bi-directional static load tests have become an important tool in the development of cost-effective foundation systems. The main advantages when compared to conventional top-down load test methods are the possibility to install single or multiple bi-directional load cells at prescribed depths within the rock socket and apply significantly higher test loads that would otherwise be limited by the structural capacity of the test pile or reaction system. These were key features that, over the past decade, led to the observation that design methods currently adopted in Dubai significantly underestimate the load carrying capacity of rock-socketed piles in weak carbonate rocks.

A case study is presented in which test data from over 40 bi-directional load tests carried out across Dubai was incorporated into the design process to overcome local foundation design limitations and establish more appropriate design methods for rock-socketed piles.

Upon completion of a site specific preliminary load testing programme comprising 6 bi-directional load tests on instrumented test piles it was demonstrated that the correlation proposed by Zhang & Einstein (1998) for smooth sockets may be used as a robust basis of design approach for rock-socketed piles in Dubai. Additionally, the correlation proposed by Rowe & Armitage (1987) for rough sockets may also be adopted, when supported by data from a well-executed ground investigation and validated by a programme of preliminary test piles.

Tower settlement monitoring data and observations from other project sites also support the approach to adopt a stiffness ratio of $0.2E_0$ as a lower bound value in routine foundation design.

The savings in construction materials associated with an overall pile length reduction of about 13,600m corresponding to 700 tonnes of steel reinforcement and 17,000 m³ of foundation concrete established a new precedent in sustainable foundation design in the Emirate of Dubai.

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