

## Stiffness parameters for the foundation design of The Landmark tower

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### Introduction

This paper reviews the selection of stiffness parameters used for the foundation design of The Landmark building, by comparing the results from numerical modelling with monitored settlement data. In doing so the role of single pile load tests in determining the stiffness parameters for design is assessed.

The Landmark is a 330m tower on the Corniche in Abu Dhabi (Figure 1). It consists of five storeys of basement and 72 storeys above ground. It is supported on a pile assisted raft (Figure 2) consisting of 34 piles, each 14m long with a diameter of 1.5m.

This paper initially details the geology, engineering properties, and the design parameters used for The Landmark. Stiffness profiles are then back-figured from two single pile load tests using a numerical model. The stiffness profiles back-figured from the pile load tests were used in a numerical model of the piled raft to ascertain if they provided a good prediction of the monitored piled raft settlement.

Finally, a stiffness profile that best matched the monitored settlement of the piled raft was determined. Settlement data that the numerical model could be matched to was available from extensometers in the soil, and survey points on top of the raft.

### Geology and stratigraphy

The geological succession of the region consists of sand deposits overlying weak calcareous sandstone and calcarenite. This overlies interbedded weak mudstone, siltstone and very stiff gypsum bands. The original ground level at The Landmark site was at +2.5mADD. The bearing level of the piled raft is at approximately -21mADD, consequently, the piles and raft are mainly situated in mudstone.

The mudstone is generally weak with bands of weak to moderately weak gypsum. The layers correlated across the borehole sections indicated that the stratigraphy is



Figure 1: Architectural illustration of The Landmark tower in Abu Dhabi

generally consistent and horizontal. No cavities were encountered in the mudstone and fracturing within the rock was generally sub-horizontal, tight, and with no evidence of infilling or weathering. The mudstone was proven to 100m depth. Below approximately 50m depth (-47.5mADD) the mudstone was interbedded with calcareous sandstone bands. Unconfined compressive strength (UCS) tests on mudstone samples recovered from high-quality coring ranged from 0.53MPa to 4.8MPa, bulk density varied between 1,900kg/m<sup>3</sup> and 2,100kg/m<sup>3</sup>. The gypsum bands are stronger than the mudstone with unconfined compressive strength varying between 5.7MPa and 14.5MPa, and bulk density between 2,000kg/m<sup>3</sup> and 2,200kg/m<sup>3</sup>. The ground water was at 0mADD, indicating that dewatering within the basement footprint would be necessary during construction.

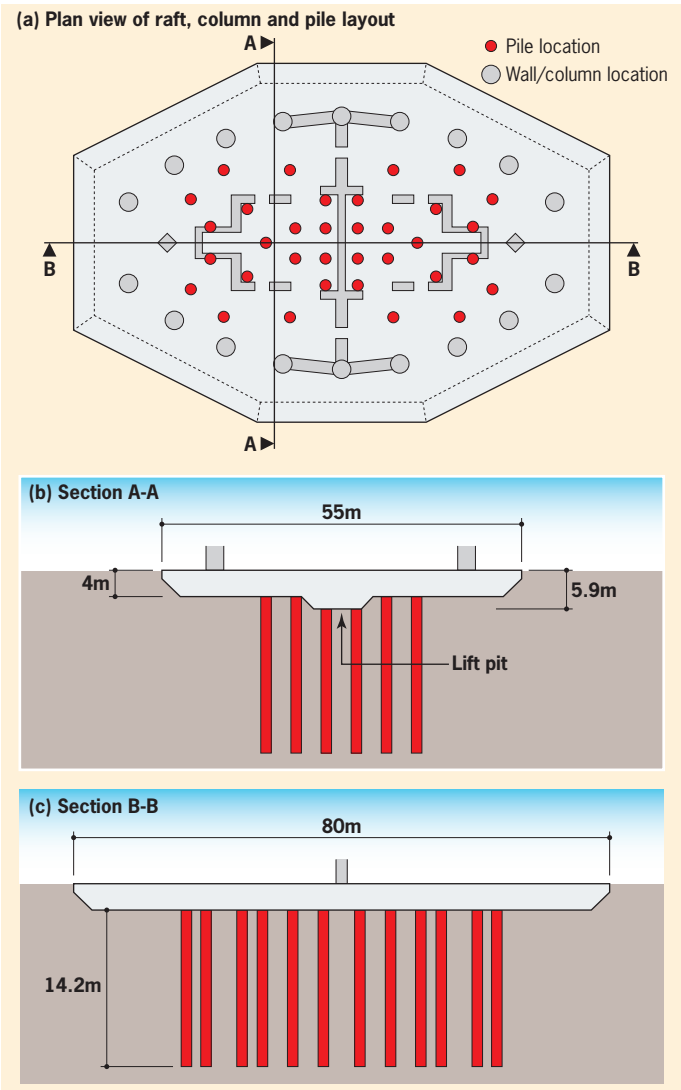


Figure 2: Foundation layout of The Landmark

### Engineering properties

The Young's modulus estimated from downhole geophysics, pressuremeters (high pressure dilatometer), and laboratory UCS tests (instrumented with strain gauges) are compared in Figure 3. The stiffness determined from the UCS tests is the secant Young's modulus at 50% of the maximum

shear stress. The Young's modulus estimated from the downhole geophysics used the average velocity through the strata, ignored the effect of refraction at material boundary layers, and assumed the material was isotropic (Pinches and Thompson, 1990). The modulus estimated from the pressuremeter data is the unload/reload modulus.

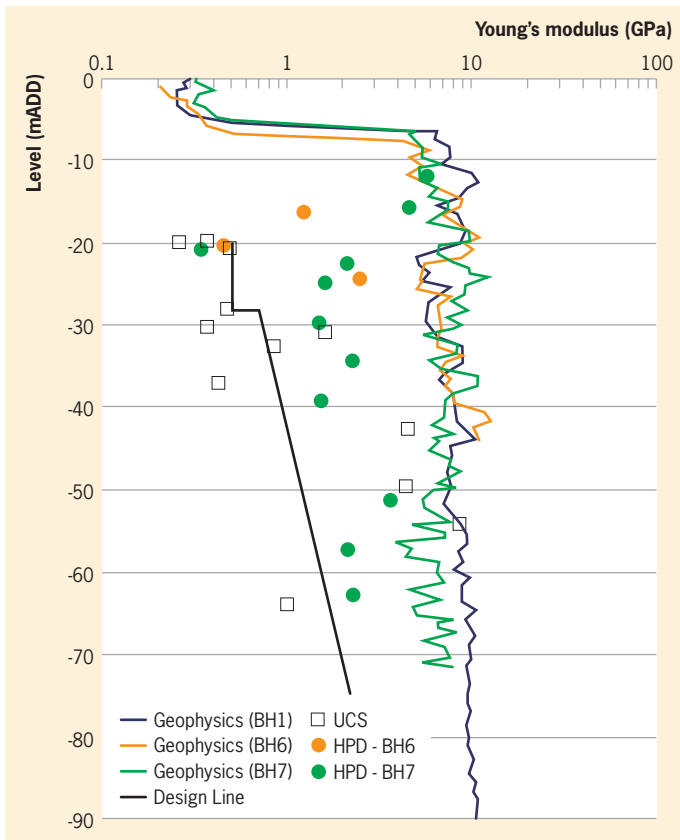


Figure 3: Variation of Young's modulus with elevation

The initial modulus from the pressuremeter was assumed to be accompanied by disturbance, and to give a low estimate of the true insitu stiffness.

Haberfield et al (2011) showed that if the small strain values obtained from the geophysics are reduced by a factor of five to account for the strain level, good agreement with the values of Young's modulus obtained from the pressuremeters could be achieved. This data shows a similar trend.

In general, the Young's modulus derived from the UCS tests (which is likely to have positive bias as only the strongest samples would have survived coring) are marginally smaller than those determined from the pressuremeter data. This may be because of sample disturbance caused by retrieval and handling, although the agreement between UCS and pressuremeter data does improve with depth.

Both the UCS tests and the pressuremeter indicate a zone of high compressibility around -20mADD (Figure 3). It was hypothesised that the zone of high compressibility was caused by drilling as stress relaxation and potential softening of the mudstone can occur due to availability of moisture. However, this could not be confirmed and therefore an

allowance for a weaker zone was made in design.

Figure 4 compares the strength from the UCS tests and those derived from the pressuremeter tests assuming a Tresca failure criterion. The estimate of the strength from the pressuremeter is generally closer to the higher values of strength predicted by the UCS tests. As shown by Clarke (1995) it is usual for strength estimated from the pressuremeter to be higher than that predicted from laboratory testing. The pressuremeter proved useful in

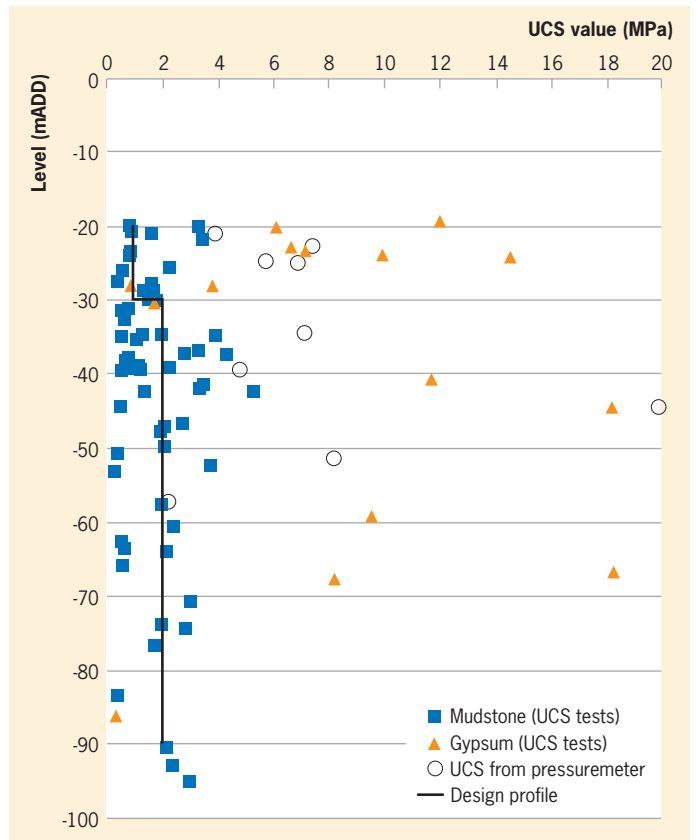


Figure 4: Variation of unconfined compressive strength with elevation

identifying the stiff gypsum layers. As shown in Figure 4, there is good correlation between the material identified as gypsum in the UCS tests, and the strength at similar depths predicted from the pressuremeter data.

### Single pile load tests

Working pile load tests were conducted on two piles (CP8 and CP19). Both load tests were conducted before the basement was excavated, using a bi-directional jack operated from the ground

surface (-7mADD). For CP8 the jack was positioned 3m from the pile toe, whereas for CP19 the jack was positioned 1m from the pile toe (Figure 5). The location of the jacks was chosen to separate the base and shaft response of the pile. Load was applied incrementally up to a value of 18.3MN and to 16.2MN in each direction for CP8 and CP19 respectively. Movement was measured against a reference frame, using extensometers installed at the top and bottom of the hydraulic jack assembly.

The working load tests were back analysed using a three-dimensional numerical model in the finite element software Abaqus CAE 6.7-1. A linear elastic constitutive model with a Tresca failure criterion was adopted for the weak rock. " $K_0$ " was assumed to be unity and the initial insitu vertical stress profile was created based on each of the material's densities multiplied by the gravitational force.

Three different stiffness profiles were used along the shaft of the pile in the numerical model. All three profiles assumed a constant value of stiffness along the pile shaft that was based on the magnitude of stiffness predicted by the UCS data, the pressuremeter, and the geophysics data.

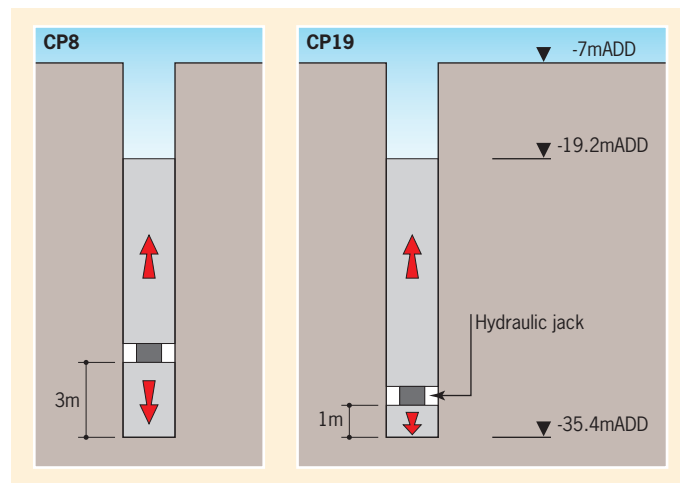


Figure 5: Illustration of pile load test

The borehole records indicated >>

» two stiff gypsum bands along the length of the pile. These were considered to be of significance and were included in the numerical model. They were taken to occur at -19.15mADD and -22.4mADD (Figure 4) with a thickness of 0.25m and 1.2m respectively. In the original design, gypsum bands were not modelled. The adopted stiffness values for the materials along the pile shaft are shown in Table 1.

As shown in Figure 6 and 7, for load test CP8 and CP19 respectively, the shaft response of both piles was very stiff and non-linear. The first two load steps of CP8 (Figure 6) agreed with the numerical model using the geophysics data, then on increased loading the pressuremeter data, and finally for the last three load steps, with the UCS data. This result indicates the inability of a linear elastic constitutive model to match the full pile shaft response. It was decided that using a stiffness profile based on the pressuremeter data in the numerical model would provide the best estimate of pile shaft displacement, for the majority of load steps.

The recorded displacement after the first load step of CP19 (Figure 7), matched the predictions from the numerical model using the stiffness profile based on the pressuremeter the best. On increased loading, the stiffness profile based on the UCS data matched better. Thus, it was decided that a numerical model with a stiffness profile based on the UCS data would provide the best estimate of shaft displacement for the most load steps of CP19.

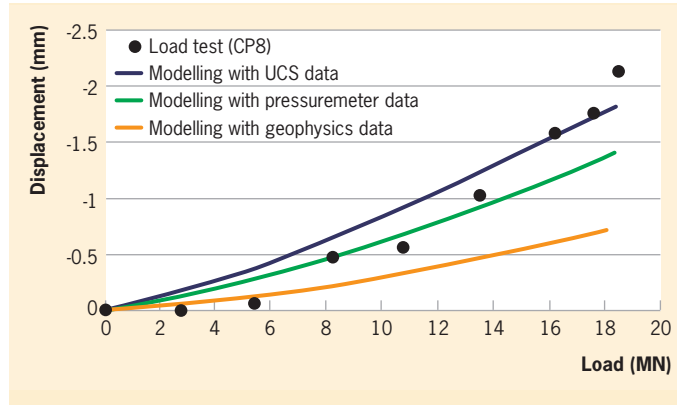
The base response of pile load tests CP8 and CP19 are shown in Figure 8. As would be expected the response of CP8 is stiffer than that of CP19, because it tested a larger portion of pile below the hydraulic load cell. To match the base response of both load tests it was necessary to reduce the Young's modulus of the soil directly surrounding the base (Figure 9).

For CP8 and CP19, the Young's modulus in the numerical model for soil directly surrounding the pile base, was reduced to 0.4GPa and 0.1GPa respectively. This represented a decrease by a factor of 2.5 and 10 from that assumed in the design for load test CP8 and CP19 respectively.

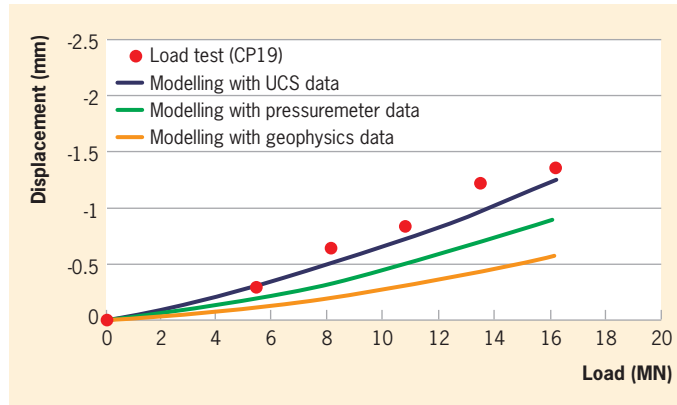
The site investigation data indicated that the mudstone was susceptible to softening if subjected to stress relaxation and moisture. If, during construction, there was a delay in pouring the concrete, or the drilling fluid was not completely removed, the mudstone may have softened. This would explain the soft pile bases indicated by both pile

**Table 1: Magnitude of stiffness assumed for back analysis of shaft load test**

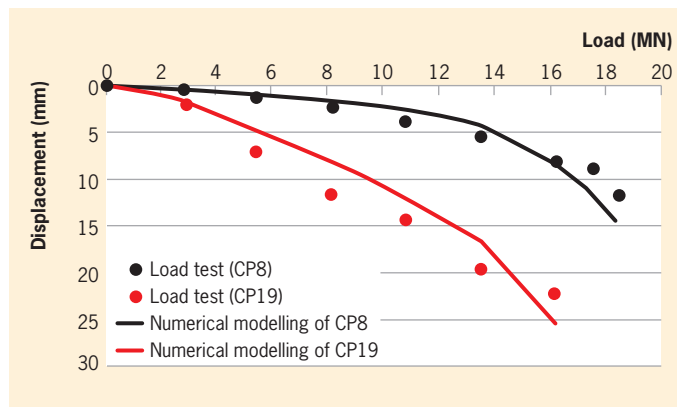
Material Reference	Young's Modulus (GPa)			UCS (MPa)	Poisson's ratio	Density (kg/m <sup>3</sup> )
	From UCS	From pressuremeter	From Geophysics			
Mudstone	0.67	1.01	5.00	1.00	0.23	2,040
Gypsum	5.00	5.00	10.0	8.00	0.24	2,240
Concrete	35.0	35.0	35.0	n/a	0.15	2,450



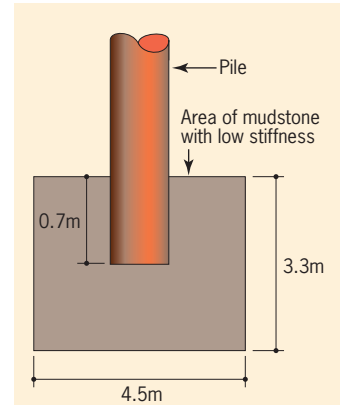
**Figure 6: Comparison of shaft response from load test CP8 and numerical modelling**



**Figure 7: Comparison of shaft response from load test CP19 and numerical modelling**



**Figure 8: Comparison of base response from load tests and numerical modelling**



**Figure 9: Area of mudstone reduced in stiffness to match base response of pile load test to numerical model**

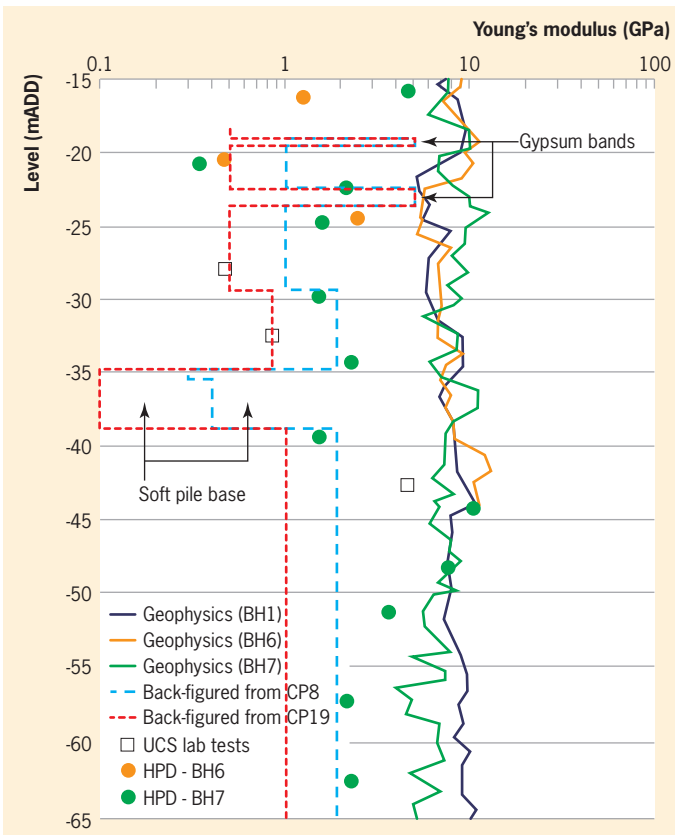
load tests. The final adopted stiffness profiles for each load test are compared to the site investigation data in Figure 10.

### Piled raft

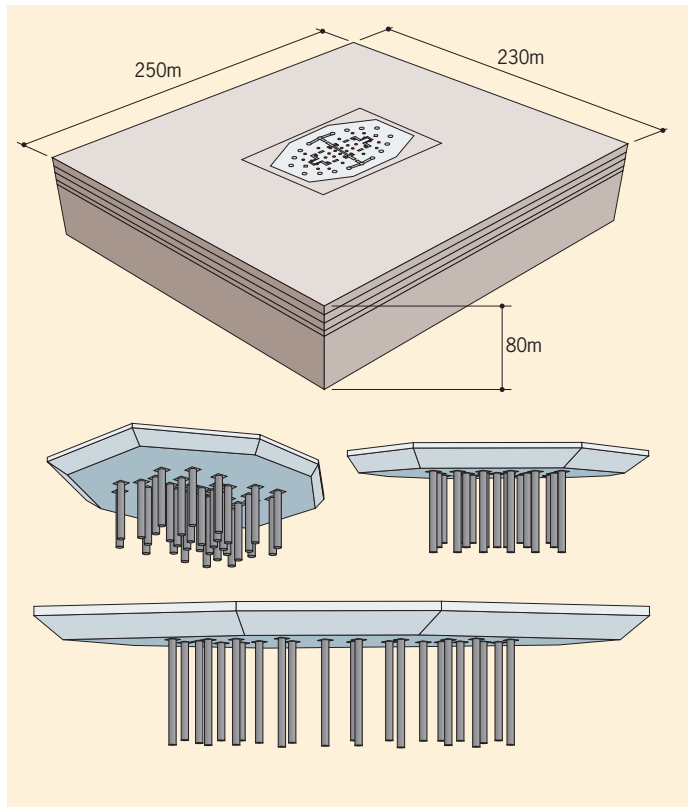
The stiffness profiles back-figured from the pile load tests (Figure 10) were used in the three dimensional numerical model of the piled raft. The numerical model is shown in Figure 11. The results of the numerical analysis were compared to the monitored settlement of the piled raft after the completion of floor 36, 52, and 72 (the top).

Figure 12 compares the settlement predicted from the numerical model when using the parameters derived from load test CP19, and those recorded by the extensometers just beneath the raft. The locations of the extensometers are shown by the inset in Figure 12. The settlement from the numerical model is taken from the centre of the slab at the ground/slab interface. It will be noted that the use of the parameters derived from CP19 over predicted the settlement when compared to the monitored data.

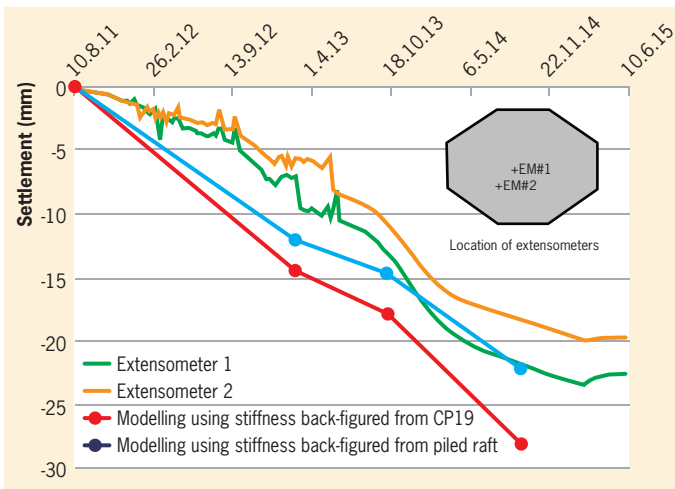
To match the monitored settlement after the construction of the 72nd floor, it was necessary to adopt stiffness values similar to those estimated from the pressuremeter. This meant a profile of stiffness twice that assumed in design. The prediction from the



**Figure 10: Comparison of SI data and stiffness profiles back-figured from pile load tests**



**Figure 11: Numerical model of piled raft supporting The Landmark**



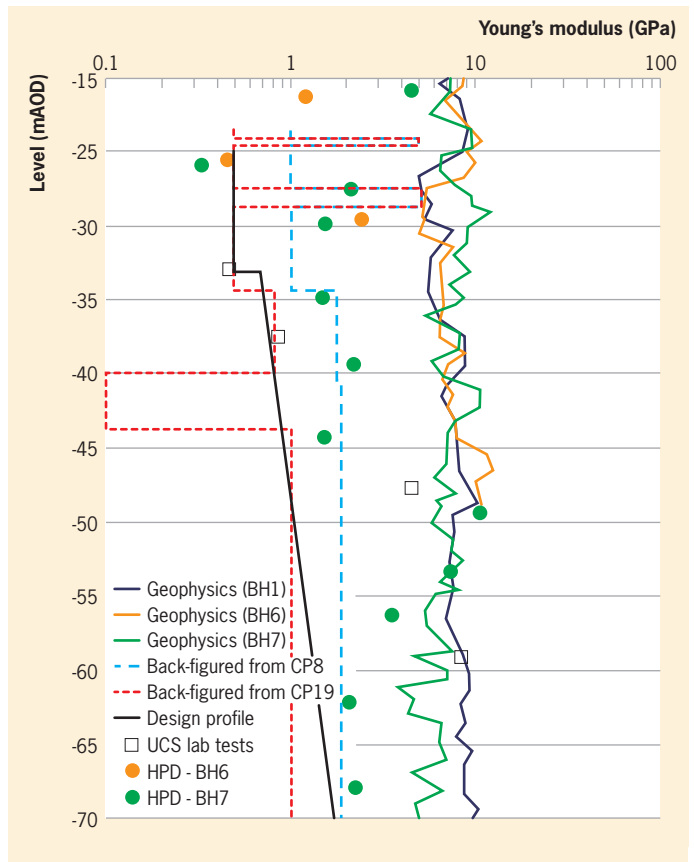
**Figure 12: Comparison of monitored settlement and predictions from numerical modelling of piled raft. (Settlement taken from centre of raft at raft/ground interface, location of extensometers shown).**

numerical model using the back-figured stiffness profile, and the monitored settlement for all three construction stages, are shown in Figure 12. Interestingly, the back-figured stiffness profile was the same magnitude of stiffness back-figured from load test CP8. For this reason, the prediction from the numerical model using the stiffness back-

figured from load test CP8 is not plotted in Figure 12. The back-figured stiffness profile is compared to the site investigation data, and the original design profile in Figure 13.

### Discussion

Back analysis of the piled raft suggests that the moduli derived from the unload/reload cycle of >>



**Figure 13: Comparison of back analysed stiffness profile, design profile and SI data**

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» the pressuremeter test provides a good estimate of the in situ stiffness for piled rafts in weak rock. Similar evidence has been presented by Haberfield et al (2011) for the loading of barrettes in soft calcareous rock in Dubai.

The stiffness derived from the pressuremeter data was about twice that assumed in design. However, it should be remembered that the pressuremeter provides a measurement of horizontal shear stiffness.

Adopting the stiffness derived from the pressuremeter assumes that the material is inherently isotropic. Clayton (2011) has described how most soils and weak rock can be expected to display stiffness anisotropy.

Comparison of the numerical model and the single pile load tests shows that the linear elastic constitutive model adopted in these analyses may not be optimal. Both pile load tests indicated non-linear shaft behaviour after very small

displacements, with CP8 indicating yield at around 0.1% of the pile diameter. Therefore, future analysis may benefit from adopting a non-linear elastic perfectly plastic constitutive model.

The stiffness back-figured from the pile load test CP19 was about half that back-figured from the monitored piled raft settlement, whilst the stiffness back-figured from CP8 was about the same magnitude. Both load tests indicated soft pile bases, although the inclusion of soft pile bases in the numerical model was not critical to achieving a match with the monitored settlement.

## Conclusions

This case study has shown that:

- The use of a stiffness profile derived from the pressuremeter data in the numerical model provides the best match with settlement of a piled raft in weak rock;
- The use of a linear elastic perfectly plastic constitutive model

was not appropriate for modelling a single pile load settlement response in weak rock;

■ The shaft response from pile load tests can be used to obtain estimates of ground stiffness for use in piled raft settlement analyses, but the results need to be compared with data from other sources before it can be used with confidence.

■ The pile base response from load tests cannot be relied upon to estimate ground stiffness due to the potential for softening during pile construction.

## Acknowledgements

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Special thanks must also go to Andrew Websper for his patience when dealing with my "question a day" about the original design of The Landmark.

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Contact: Steve Poulter, email: [s.poulter@fes.co.uk](mailto:s.poulter@fes.co.uk)

#### Cone Penetration Testing

4 May, Fugro House, Wallingford. This is an introductory course and technology update on Cone Penetration Testing theory and application including its advantages and limitations. The course will be repeated at Wallingford on 28 September and 14 December, in Edinburgh on 15 June, on 19 October in Nottingham and on 23 November at Exeter.

Contact: Steve Poulter, email: [s.poulter@fes.co.uk](mailto:s.poulter@fes.co.uk)

#### Burlington House Lecture – Thames Tideway Tunnel

15 May, 6pm. Burlington House, London. Phil Stride, head of London Tideway Tunnels at Thames Water, will talk about the proposed 25km long and 7.2m diameter Thames Tunnel, which will broadly follow the route of the river from west to

east, up to 67 metres below ground, to capture sewage from the 34 most polluting combined sewer overflows. Contact: Admission is free but by ticket only, available from the conference office, Geological Society on 020 7434 9944 or email: [registrations@geolsoc.org.uk](mailto:registrations@geolsoc.org.uk)

#### John Mitchell Lecture – Track Displacement and Energy Loss in a Railway Embankment over Peat

16 May, 6pm. ICE, London. Dr Michael Hendry from University of Alberta and Dr David Hughes from Queen's University, Belfast, will present the research conducted with NIR as it was presented in the ICE-Geotechnical Engineering Journal paper "Track Displacement and Energy Loss in a Railway

Embankment over Peat". The lecture will continue with an introduction to the Canadian Rail Research Laboratory and the research being conducted in Canada into soft peat foundations. Contact: BGA, tel: 020 7665 2007 or visit: [www.bga.city.ac.uk](http://www.bga.city.ac.uk)

#### Cannon Place Redevelopment – Micropiles and Associated Piling Works

16 May, 6.30pm. Renold Building, University of Manchester. Jim Martin and Hugh St John from GCG will talk about the use of micropiles and other geotechnical solutions used for the redevelopment of Cannon Place. Contact: Martin de Krester, tel: 0161 927 8257, or email: [martin.de.krester@acem.com](mailto:martin.de.krester@acem.com)

## Piling and Foundations conference

8 and 9 May, 9am-5pm. London *Ground Engineering's* 2012 Piling and Foundations conference will give attendees insights into the market through innovative case studies, updates on essential technical guidance and the opportunity to participate in inspirational

discussion sessions.

Topics that will be covered include geothermal piles, Eurocode 7, pile reuse, risk management and cost efficiency.

Contact: James Crumly, email: [james.crumly@emap.com](mailto:james.crumly@emap.com) [www.pilingevent.co.uk](http://www.pilingevent.co.uk)

#### GeoDATA 2012

17 May. Digital World Centre, Manchester. GeoDATA 2012 will bring together leading professionals from the geographic industry to share practical advice on how to think smarter and do more with geographic information within your organisation. Contact: tel: 01223 880077, or email: [training@geoinformationgroup.co.uk](mailto:training@geoinformationgroup.co.uk)

#### Anomalies in Tunnelling

23 May, 6pm. Burlington House, London. A joint meeting of the British Tunnelling Society and Engineering Group. Contact: Paul Emmerson, tel: 01225 855002 or email: [Paul.Emmerson@emdrilling.co.uk](mailto:Paul.Emmerson@emdrilling.co.uk)

#### Shale Gas in the UK: What, Where, Why, How?

24 May, 6.30pm. Birchwood Park, Warrington. BGS geologist Ed Hough will deliver this presentation to the Geological Society North West Regional Group. Contact: [www.geolsoc.org.uk/nwrg](http://www.geolsoc.org.uk/nwrg)