Settlement of a high-rise building under construction – measurement and modelling

Tassement d’un immeuble de grand hauteur pendant la construction – mesure and modélisation

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ABSTRACT: The building comprised three 70-storey towers constructed over a mat foundation supported on 399 piles in Bangkok. Design incorporated 11 boreholes, 5 CPTu tests, an instrumented static load test on a trial pile, 5 dynamic load tests, and numerous iterations between the structural engineer and the geotechnical engineer. 15 vibrating wire strain gauges were incorporated into the tops of piles when they were connected to the mat. These strain gauges were monitored and settlements of the mat foundation were surveyed until about 12 months after the towers were topped out. The paper presents the design and the monitoring results, together with a numerical analysis for comparison.

RÉSUMÉ : Le bâtiment situé a Bangkok était constitué de trois tours de 70 étages posées sur une fondation sur radier supportée par 399 pieux. La conception comprenait 11 forages, 5 tests de CPTU, un essai de charge statique sur un pieu d'essai, 5 essais de charge dynamique, et de nombreuses itérations entre l'ingénieur des structures et l'ingénieur géotechnique. Une quinzaine de jauges de contrainte ont été incorporées dans le haut des pieux pendant la construction de la fondation. Ces jauges de contrainte ont été surveillées et la consolidation de la fondation a été contrôlée pendant environ 12 mois après l’achèvement des tours. Ce document présente la conception et les résultats de la surveillance, ainsi qu'une analyse numérique pour la comparaison.

KEYWORDS: piling, foundations, instrumentation, monitoring, numerical modelling.

1 INTRODUCTION

This paper describes the design, construction and monitoring of the foundations for a resdiential development comprising three towers in Bangkok, Thailand. It is reasonably well known and widely reported that Bangkok is situated on the flood plain of the Chao Prayah River which stretches about 400 km north to south and 180 km west to east. The delta is filled with Quaternary Alluvium, the exact thickness of which is not well defined, as very few boreholes have penetrated to rock, but it is believed to be about 1000 m. The profile consists of alternating layers of clay and sand, within which are many aquifers, as illustrated in Figure 1.

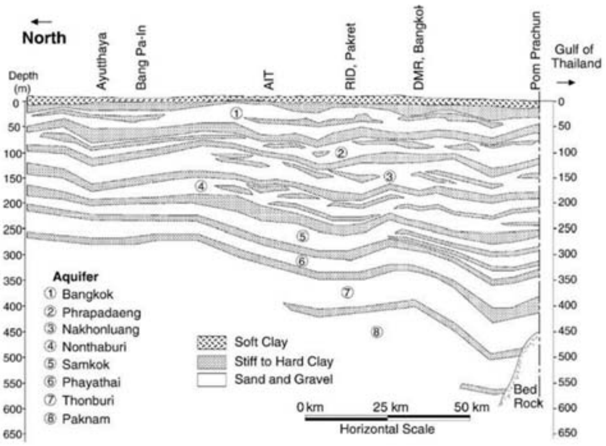


Figure 1. Hydrogeological North-South profile of the Lower Chao Prayah Basin showing principal aquifers (after Groundwater Division, DMR, 1980)

Foundations for high-rise buildings in Bangkok have typically been taken to the “Second Sand Layer”, at a depth below ground level of about 60 m, since the publication of the results of settlement monitoring of a number of high-rise buildings by Sambhandharaksa et al (1987). They are normally bored piles about 1 m in diameter, formed under drilling fluid of bentonite or polymer. Similar piles have been used for many major infrastructure projects in Bangkok, including instrumented static load tests, so the performance of these pile types is reasonably well known.

An additional consideration in the pile design is the lowering of the ground water level within the aquifers illustrated in Figure 1. This has been going on for many decades, with the water being used for commercial, industrial and agricultural use, as well as potable water. The lowering of the water level in the aquifers has been monitored at around 70 m, and the resulting change in effective stresses has led to significant settlement of the ground surface. Nutalaya and Rau (1981) reported 300 to 800 mm over a period of 30 to 40 years, with one location showing 600 mm in 14 years. Close to the subject site, the effects can be seen in the differential settlement between a piled footbridge and its original bottom step, shown in Figure 2.

Figure 2. Pavement settled away from piled footbridge as a result of surface settlement

This groundwater extraction started in the 1950s and continued right up until the mid-1990s, when the Prapadaeng and Nonthaburi aquifers were depleted to about 60 m below ground level. Then, starting in late 1997, there was quite a remarkable recovery, related to the provision of piped water in the metropolitan area and a ban on extraction. This has been plotted and reported by Ishitsuka et al (2014), as seen in Figure 3.

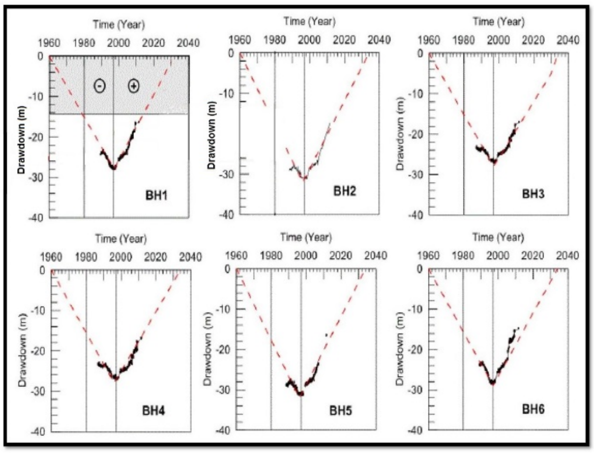


Figure 3. Changes in groundwater level with time in the Prapadaeng aquifer at 6 borehole locations (after Ishitsuka et al (2014)

2 THE BUILDING

The development involved the construction of three towers, each of 70 storeys and therefore in excess of 200 m in height, sitting side by side on a common mat foundation and structurally braced at three levels, as seen in Figure 4.

Figure 4. View of towers nearing completion

3 THE FOUNDATIONS

It was not possible to use a piled raft foundation, since the superficial soils were incapable of supporting the concentrated building loads, and the surface subsidence meant that, over time, the soils were likely to settle away from the underside of a raft rather than provide support. The columns and lift cores were therefore laid out in plan, and the dead and live loads estimated. A conventional structural model was then created in a program called ETABS, in which the framed superstructure was accurately modelled, together with all action effects, while the subprogram SAFE was used to represent the piles as linear elastic springs on a rigid base. This produced the result that the highest pile loads would be under the lift cores because, relative to a rigid base, the superstructure appeared to be flexible.

With a soil profile as illustrated in Figure 1, and rock about 1000 m beneath the surface, to treat piles as linear springs on a rigid base was clearly misleading. By contrast, the geotechnical model was created using PIGLET (Randolph, 2007), which uses a simple soil model with properties either constant or linearly increasing with depth, which allows for load sharing between piles and soil, but can only model the superstructure as fully flexible or infinitely rigid. With the rigid option the structure is much stiffer than the soil with the effect that maximum pile loads occur at the corners. The range of loads predicted was between less than 2,000 and more than 11,000 kN per pile.

In reality neither model is a fair representation of the likely behavior of the foundation. The stiffness of the superstructure will be somewhere between the extremes of rigid and flexible, while the soil will be much more flexible than the assumed rigid base. What is critical is the flexibility of the structure relative to the soil.

Some preliminary analyses had shown that the preferred foundation layout was 399 piles, each 1 m in diameter, laid out on an equilateral triangular grid. The SAFE model was therefore used to generate a distribution of loads on these piles, and the loads were then input into PIGLET with a flexible foundation. The result was a dished profile with a maximum settlement of over 140 mm at the centre, a minimum of less than 60 mm at the ends, and an average of 95 mm. On the other hand, under a rigid pile cap, the average settlement was about 50 mm and the average load was about 6,000 kN. These were used to determine the stiffness of the pile spring at each location, using the settlement at that location with a flexible foundation defines as *disp*. The proposed settlement at each location was then defined as:

$$\delta = \frac{n}{3}(50+[disp-95]) \tag{1}$$

and the spring stiffness as:

$$S = 6{,}000/\delta \tag{2}$$

Three runs of SAFE were then carried out, using these manually determined stiffnesses, with n = 1, 2 and 3. As n was increased the flexibility was increased so the stiffness was decreased, which made the stiffness of the mat and superstructure more dominant and pushed load towards the edges.

For n = 1 the structural stiffness was the highest, and the result was a mat which dished towards the centre, with a maximum settlement of about 62 mm and a minimum of 25 mm. As the flexibility increased, for n = 2, the dishing increased with a maximum of 72 mm and a minimum of 15 mm. For the maximum flexibility with n = 3, The maximum was about 83 mm and the minimum about 20 mm. These gave rise to a distribution of pile load as shown in Figure 5, where the flexibility in the long direction allows the middle of the mat to go down increasing the pile load, whereas the stiffness in the short direction causes the highest pile loads to be on the outside edges near the centre.

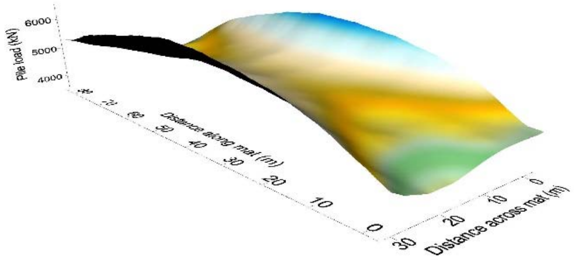


Figure 5. Predicted distribution of pile loads based on structural and geotechnical analysis

4 MONITORING

The opportunity was taken to monitor the performance of the foundation under load. Of the 399 piles 15 were selected, 5 in each of 3 rows, to be monitored with vibrating wire stain gauges (VWSG). A 75 mm diameter hole, 300 mm deep, was cored in the top of each pile after the piles had been trimmed for connection into the mat. The cores were recovered and tested to determine the modulus of the concrete. The VWSGs, fixed to a small reinforcement bar, were then grouted into each hole with fine aggregate concrete, and the electrical leads taken through conduit buried in the mat foundation to a centrally located instrumentation box. The gauges were then read regularly during the construction of the towers.

At the same time the contractor carried out level monitoring of the top surface of the mat foundation. 45 settlement points were nominated, of which 40 were actually monitored, but the requirements of construction necessarily meant that not all points were visible at the time of every survey. Nevertheless 14 were available most of the time.

4 .1 *Settlement monitoring data*

The settlement monitoring data, plotted in Figure 6, uses a scale on the ordinate axis from 0 to 120, and this has been used to register both the number of floors and the settlement in millimetres. It shows that the settlement of the mat foundation closely followed the increase in load represented by the construction of the building, after an initial lag, and then continued at what appears to be a decreasing rate once the building had been topped out. During this period there would have been further increase in load, which cannot be quantified, as a result of internal fit out. In addition to plotting the overall settlement, the shape of the mat foundation can also be examined, as seen in Figure 7. This shows that, by 24 September 2008, at which time the building had been topped out for 3 months, the maximum settlement, near the centre, had increased to just over 80 mm with about 50 mm at either end.

4 .2 *Pile load monitoring data*

The VWSGs were installed in August 2006, and read monthly from October 2006 until May 2008, and then again in April 2009. Figure 8 shows the strain in individual piles plotted against time, as well as a bold line showing the progress of the building, and diamond or square shaped symbols representing average pile loads converted to strain, all to the same numerical scale. This indicates that the increase in pile load lagged behind the progress in building construction in the early stages.

This is considered to be because the initial loads will have been carried by the surface crust and the soft clay. However, as these became too heavy the loads were progressively transferred to the piles. In Figure 8 the diamond shapes indicate the average pile loads if no load is transferred to the soil by the mat foundation, while the squares indicate the average pile loads if the pressure on the soil starts at about 80 kPa, increases to 110 kPa, and drops back to 80 kPa.

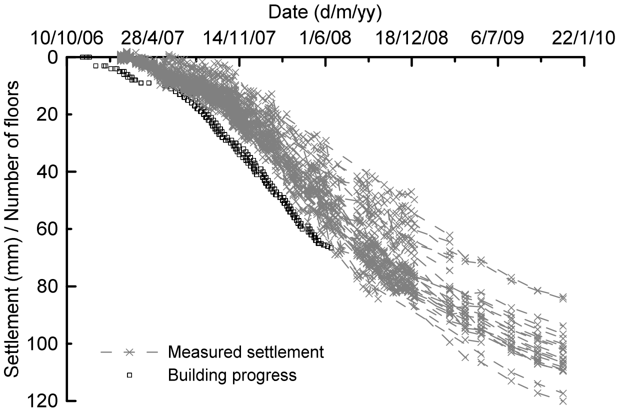


Figure 6. Settlement measurements of mat foundation during construction period

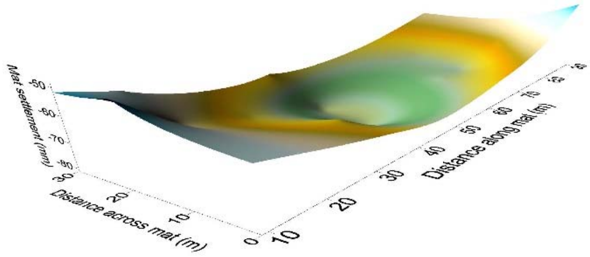


Figure 7. Shape of mat foundation in September 2008

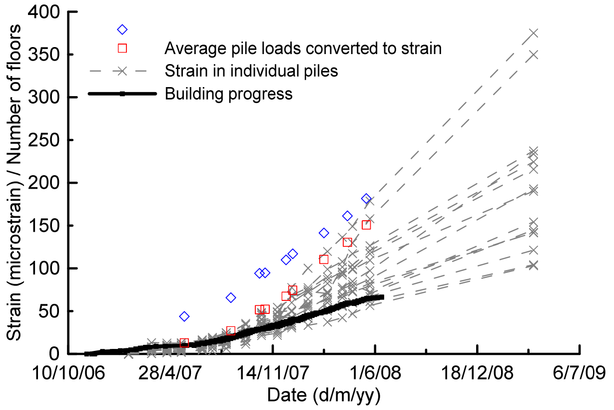


Figure 8. Variation of pile load and building height during construction

5 NUMERICAL MODELLING

As a part of the contract works, an instrumented static load test was carried out, and in advance of the test a prediction of the load settlement performance was made using the program RATZ. Once the test had been carried out, the data was corrected for the effects of tension piles, and compared with the Cemset model, after Fleming (1992). An axisymmetric model was analysed using the Finite Element code PLAXIS 2D and the Hardening Soil model. The results of all of these analyses are compared with the measured data in Figure 9, and it can be seen that very good agreement was obtained.

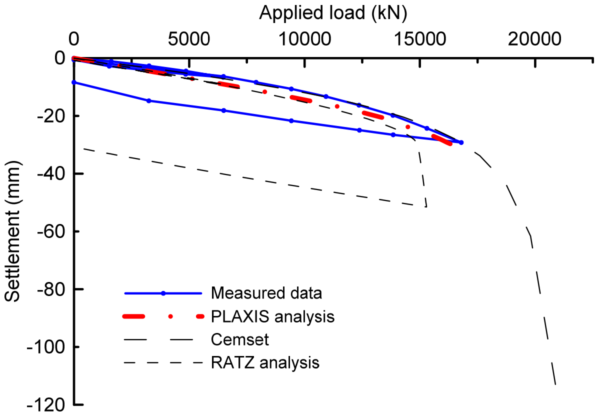
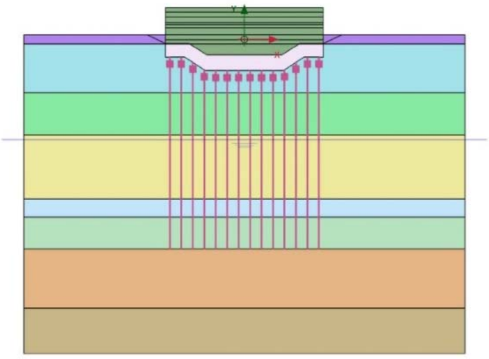


Figure 9. Load settlement curves as measured and modelled

Since conventional analysis within a 2D FE model involves conditions of plane strain, it is not suitable for modelling the behavior of a single pile. The 2D FE analysis was therefore applied to a 1 m wide strip across the 35.8 m width of the foundation, as illustrated in Figure 10. Since the piles were set out on an equitriangular grid, at 5.2 m centres on one line with the intervening piles on a line 1.5 m away, these were reduced to simple parallel lines at 2.6 m spacing and 3 m apart. This led to 14 piles across the width, and about 30 rows giving 420 piles in total, which was considered close to the 399 actually used. Since the structure included three levels of transverse girders which provided significant structural stiffness, this was modelled by forming the structure as a heavy material, so that the depth of the material “beam” increased from about 2 to about 9.5 m as the load increased.

Figure 10. 2D plane strain model used in FE analysis



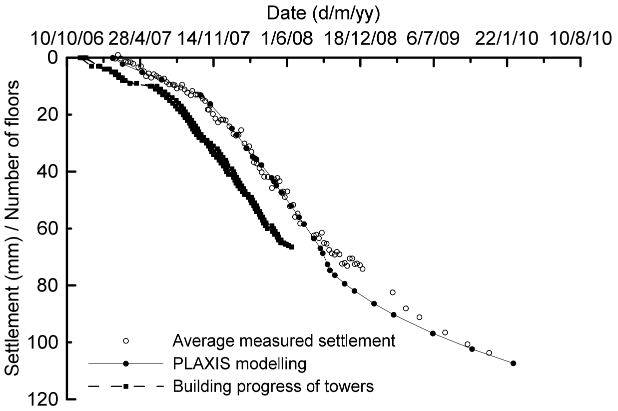


Figure 11. Comparison between measured settlement (grey points), computed settlement (heavy black line) and building height (black squares) during construction

Using exactly the same soil models as in the axisymmetric analysis, the modelled settlements have been compared with the average of the measured settlements of the mat foundation in Figure 11, which is considered to show very good agreement.

The solid squares show the progress of the building construction, and hence load, in floors, the hollow circles show the average of the measured settlements on each available date, and the line and dots show the results of the numerical model.

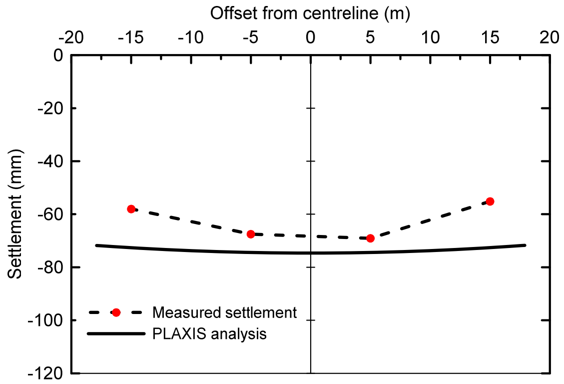


Figure 12. Settlement across the width of the mat

Finally the measured and computed settlements across the width of the 2D slice are shown in Figure 12. This suggests that the structure has been modelled too stiffly in this direction.

6 CONCLUSIONS

- The design process highlighted some of the complexities of combining structural models with geotechnical models.
- The pile design acknowledged that the relative stiffness of the soil and the structure would control the distribution of pile loads.
- The instrumentation showed that, whereas typical structural models consider the superstructure to be flexible on springs on a rigid base, in practice the stiffness of the structure is very high compared to that of the piled foundation.
- The numerical modelling shows very good agreement between the axisymmetric model and a static load test, and between a plane strain model and the mat foundation settlement.

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