LOAD SETTLEMENT PERFORMANCE OF TWO TOWERS

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ABSTRACT

Redevelopment of the south bank of the Yarra River in Melbourne comprises eight multi level residential towers and a number of other related structures. Two of the towers experienced problems during installation of their piled foundations that might have led to increased settlements if left unremediated. This paper describes the foundation problems that arose and the methods used to assess the potential settlement of the towers. On the basis of these estimates additional piling works were required for one of the towers. The foundations for the other tower were considered to be satisfactory. Settlement monitoring carried out during construction of the towers showed excellent agreement with settlement estimates and justified the decisions made.

1 BACKGROUND

A redevelopment of the south bank of the Yarra River in Melbourne comprises eight multi level residential towers and a number of other related structures. Two of the towers, Tower 2 (20 levels with a six level podium) and Tower 3 (31 levels with a six level podium) experienced problems with their piled foundations.

The geology at the site comprises alluvial deposits over siltstone bedrock at about 37 m depth. The subsurface stratigraphy can be described as about 2 m fill overlying 25 m of soft grading to stiff marine clay, over 3 m to 7 m of dense to very dense sand (Moray Street Gravel) and/or very stiff clay and very dense gravel (Werribee Formation) overlying 3.5 m to 6 m of stiff to very stiff clay (Newport Formation) overlying siltstone. Ground water level is at about 2 m depth. At the very eastern end of the site (near Tower 2), a thin layer of basalt overlies the Werribee Formation.

The buildings have a total footprint of about 120 m by 50 m and comprise two heavily loaded tower structures surrounded by a more lightly loaded podium. There is no basement, but excavations up to 4 m depth in fill and soft clay were required for the lift overrun pits.

Preliminary design calculations indicated that for serviceability considerations, the towers should be supported on piles founding in the siltstone. The preferred piling solution for the towers was for the main columns and central core to be supported on large diameter bored piles socketed up to about 10 m into the siltstone. The more lightly loaded podium structures could be founded on smaller diameter CFA piles or driven piles founding in the Moray Street Gravel (MSG), basalt or Werribee Formation (WF).

Several piling contractors proposed alternative driven or jacked segmental precast pile foundation solutions which comprised groups of 350 mm and 400 mm square precast concrete piles driven to refusal in the top of the siltstone. The driven or jacked pile solution potentially provided a more cost effective approach both in terms of cost and construction time, but presented greater risk if driven piles were not able to penetrate through the MSG to the siltstone. The piling contractor was confident that they would be able to penetrate through the MSG and WF and found in the siltstone and carried out preliminary driving tests to confirm this. The tests proved successful and a driven pile solution was subsequently adopted. Figure 1 shows the layout of columns and column loads (in kN) and proposed precast piling solution for Tower 3. The foundations for Tower 2 were similar but loads were significantly lower.

Serviceability loads for the core and main columns of Tower 3 are about 60,000 kN and up to 25,000 kN respectively.

The author’s firm was engaged by the developer to assess the subsurface ground conditions, recommend foundation solutions and assess alternatives from piling contractors.

Following installation of the precast piles for the Towers, excavation of the lift overrun pits was carried out. The unsupported excavation in the fill and soft clay resulted in significant lateral movement of the piles (in excess of 400 mm) at some locations. As a result, there was a significant risk that the moment capacity of the piles had been exceeded and the durability of a significant number of installed piles was considered unsatisfactory. New piles were installed to replace piles that had been assessed to be unsatisfactory.

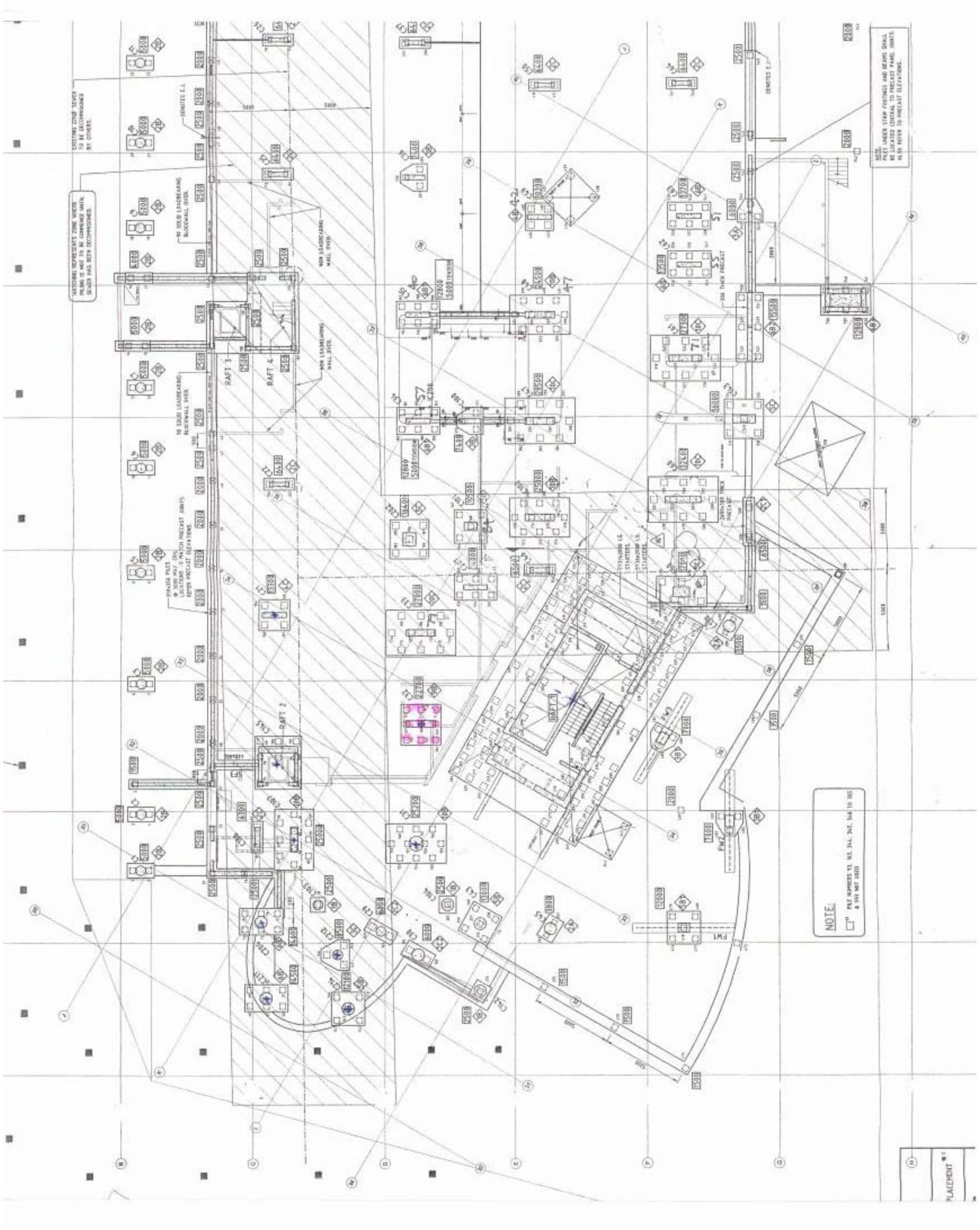

Figure 1: Plan showing driven pile foundation for Tower 3

Towards the completion of installation of the additional piles a review of as-built pile founding levels showed that a significant number of the piles for both Towers 2 and 3 had not penetrated through the MSG and WF as was expected by the piling contractor and which formed the basis of their pile design. The load from the Towers would therefore be directly applied to the stiff to very stiff clay of the Newport Formation (NF) underlying the toe of the piles, thereby creating a significant risk that the settlement of the Tower structures would be unacceptable and remedial action would be required. The developer requested that detailed analyses be undertaken to provide a best estimate of the likely settlement of the as-constructed Tower foundations under the full building loads, and to assess the effectiveness of any remedial actions proposed by the piling contractors.

2 SITE INVESTIGATION

Based on previous experience in the area of the site, it was anticipated that the podium structures could potentially be supported on piles founding in the MSG, basalt or WF but that the piles for the Tower structures would need to penetrate to the siltstone. The site investigation carried out for the Towers therefore concentrated on confirming the subsurface stratigraphy and the depth to and the properties of the potential founding layers.

The site investigation comprised twelve boreholes and nine cone penetrometer tests (CPTs). The boreholes were drilled using casing and bentonite. Undisturbed 63 mm diameter tube samples were taken in cohesive materials for laboratory testing. Standard Penetration Tests (SPT) were performed in granular and cohesive materials for strength assessment. NMLC size core samples were obtained in rock for laboratory testing. Eleven pressuremeter tests were conducted in the siltstone to assess rock stiffness.

As the Towers were to be supported on piles founding below the NF, no testing of samples recovered in this material was carried out.

3 ASSESSMENT OF SETTLEMENT

Preliminary estimates of global settlement of the installed piled footings using standard elastic solutions for a flexible footing (spanning the footprint of the Towers) indicated that the settlement of Tower 3 would probably be excessive and the settlement of Tower 2 was at best marginal. More accurate estimates of settlement were therefore required to assess what form of remedial action, if any, was required to reduce settlement of the Towers to an acceptable level.

To obtain a best estimate of settlement, it was considered necessary that the following aspects of the problem be modelled as accurately as possible:

- the subsurface stratigraphy
- the different founding levels of the piles within each pile cap
- the loads applied to each pile cap
- the interaction between piles in each cap
- the three dimensional geometry of the foundation
- the interaction between pile caps
- the consolidation and creep characteristics of the NF
- the increase in effective stress and variation of this increase with depth and location within the NF
- negative skin friction on the piles due to settlement of the soft clay

In particular, it was recognised that the major component of settlement resulted from consolidation and creep of the NF below the piles. We considered it essential that, to obtain as accurate estimate of settlement as possible, the analysis method must be able to include the effects of creep and consolidation of this layer. Good data on the consolidation and creep properties of the NF were therefore required.

3.1 CONSOLIDATION AND CREEP PROPERTIES

The need for accurate estimates of consolidation and creep properties of the NF meant that additional boreholes were required to obtain samples for consolidation (oedometer) testing. The results of the testing are shown in Figure 2.

The oedometer test results were analysed using two approaches: a simplified approach that does not take into account sample disturbance and the Schmertmann (1955) approach which allows sample disturbance to be corrected for. The simplified approach results in the range of consolidation properties set out in Table 1. After correcting the oedometer results for disturbance the estimates of average consolidation properties obtained are set out in Table 2.

The corrected oedometer results indicate significantly higher preconsolidation pressures than the uncorrected results. Given the importance of this parameter to settlement estimation, a number of consolidated quick undrained triaxial tests were carried out on samples of the NF. These tests indicated values of undrained shear strength of 165kPa to 245 kPa. Using the empirical correlations of Jamiolkowski *et al.* (1985) estimates of preconsolidation pressure in the range of 800 kPa to 1,000 kPa were obtained. This provided further comfort for adopting the higher preconsolidation pressures obtained by correcting the test results for disturbance. Estimates of the coefficient of consolidation obtained from the consolidation phase of the triaxial tests were in general agreement with the values obtained from the oedometer tests.

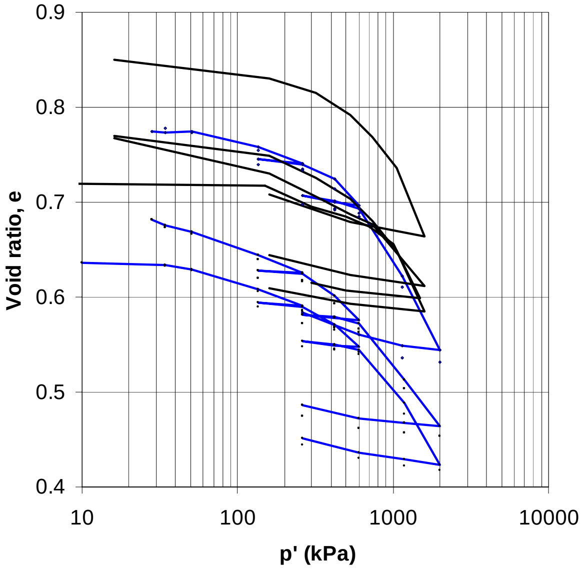

Figure 2: Oedometer test results for NF

Table 1: Consolidation Properties – Results not corrected for sample disturbance.

Compression ratio, $C_c/(1 + e_p)$	0.13–0.21
Recompression ratio, $C_r/(1 + e_o)$	0.015–0.03
Preconsolidation pressure, p'_c (kPa)	500–690

Table 2: Average Consolidation Properties – Results corrected for sample disturbance

Initial void ratio, e_o	0.72
Compression ratio, $C_c/(1 + e_p)$	0.21
Recompression ratio, $C_r/(1 + e_o)$	0.025
Preconsolidation pressure, p'_c (kPa)	750

For the settlement analyses, a tri-linear *e-log p′* model was adopted for the NF. The tri-linear model provides a smoother transition from over consolidated to normally consolidated behaviour which is more consistent with observed behaviour.

Table 3: Tri-linear *e-log p′* Model.

Stress range (kPa)	Slope of *e-log p′* curve
250 – 600	0.043
600 – 800	0.102
>800	0.357

The results of the oedometer tests also showed creep behaviour which should be included in estimates of long term settlement. Creep rates per log cycle as measured during the oedometer tests gradually increased with consolidation pressure from about 0.1% to 0.5% strain per log cycle of time. The oedometer test results for effective stresses less than 250 kPa indicated a creep rate of up to 0.1% per log cycle of time. This would not be expected to occur in the field and was thought to be the result of sample disturbance. The following empirical equation for creep rate was derived taking this into account.

$$\frac{\text{Creep strain rate}}{\text{per} \log \text{cycle of time}} = 0.498 \log\left(\frac{p'}{p'_o}\right) \qquad \textbf{(1)}$$

where p' is the vertical effective stress applied in the oedometer test, and p_o' is the estimated initial vertical effective stress at the mid-height of the NF prior to the addition of foundation loads. This value was estimated to be approximately 250 kPa.

The number of log cycles of creep that should be included in the settlement estimates depends on the time required for primary consolidation to occur in the field and the design life of the structure. The oedometer test results indicate values of coefficient of consolidation varying between about 9 and 3 m^2/year depending on stress level. Field values of coefficient of consolidation are likely to be higher than these laboratory derived values. The laboratory values indicate that for an average thickness of the NF of 4 m, primary consolidation will take between 4 months and 1 year to complete. This would imply the need to include 3 log cycles of creep in the long-term settlement estimates.

3.2 SETTLEMENT ANALYSIS

Various analysis packages were considered for calculating the settlement of the as constructed foundations. However, none of these packages could satisfactorily model all of the aspects listed earlier without major simplifying assumptions and as a result, confidence in the results obtained would have been low. For example, consideration was given to using the pile group analysis program DEFPIG (Poulos, 1990) but it was found to be unsatisfactory as it did not appear to give reasonable results when a soft layer was present below the toe of the piles. General two-dimensional numerical packages such as FLAC (ITASCA, 2000) and finite element codes were considered to be unsuitable due to the three dimensional aspects of the problem.

A hybrid approach was finally adopted. In this approach FLAC was used to model the behaviour of individual pile groups and to obtain the distribution of changes in vertical effective stress throughout the NF. An EXCEL (Microsoft, 1997) spread sheet was then used to model the interaction between pile groups and to calculate the total change in effective vertical stress in the NF. One-dimensional consolidation theory was used to assess consolidation settlements from the change in vertical effective stress. Creep settlements were also estimated assuming a linear log-time settlement response. Details of this approach are set out below.

3.2.1 FLAC Modelling of Isolated Pile Groups

Figure 1 shows the main columns supported by pile groups. Each group was modelled according to approximate (equivalent) pile group (cap) radius. Axisymmetric FLAC analyses were carried out for the five different cap radii listed in Table 4.

Table 4: Pile group size and loads

Pile Group radii (m)	Load (MN)
5.35	68
1.7	19
1.15	11
0.6	4
0.2	1.6

These analyses assumed a subsurface profile comprising 27 m of fill and soft grading to stiff clay, 7 m of MSG and 4 m of the NF over siltstone. Piles were assumed to penetrate 2.5 m into the MSG layer. Pile shaft resistance was ignored through the fill and upper 20 m of the clay and ultimate shaft resistance values of 50 kPa and 200 kPa were assumed for the lower 5m of the clay and MSG respectively.

Using the results of the FLAC analyses, the variation in increase in vertical effective stress with radius from the centre of each pile group was determined for the top, middle and bottom of the NF layer. The variations of vertical stress with radius were normalised with respect to pile group radius and the loads listed above. For each size of pile group, sixth order polynomials were fitted to the vertical stress variation with radius obtained for the middle of the NF layer. Figure 3 shows the variation in increase in vertical effective stress with radius from the centre of the 1.7 m radius pile group. These polynomials were subsequently used to calculate the vertical effective stress distribution in the NF below each pile group supporting the two towers.

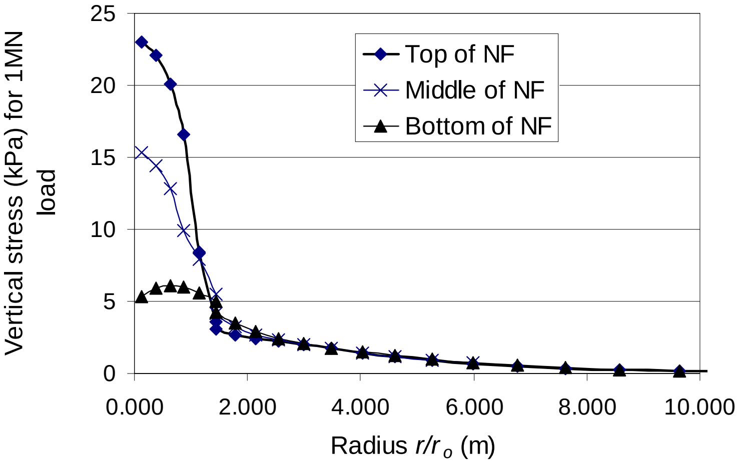

Figure 3: Variation of change in vertical stress with radius

To check the FLAC solutions we also modelled the pile group as a circular elastic footing resting on the top of the MSG and used an elastic finite layer program (FLEA) (Small and Booker, 1995) to assess the vertical effective stress change with radius at the centre of the NF layer. The FLEA vertical stress distributions were found to be similar to those from the FLAC analysis beyond a radius of about 2 m from the centre of the footing. For the larger pile groups considered (r = 5.35 m to 1.15 m) the FLAC model gave a slightly lower vertical stress increase beyond a radius of about 2 m and a higher stress increase inside this radius. For the smaller pile groups/single piles, the distribution obtained from FLAC was similar to that obtained from FLEA.

The consistency between the FLAC and FLEA distributions confirmed that a large proportion of the load applied to the top of the piles is carried by base resistance at the toe of the piles. This causes a higher concentration of stress within the NF beneath the pile group than would be expected from elastic analyses using pile group programs such as DEFPIG.

3.2.2 Modelling Interaction between Groups

An EXCEL spreadsheet was developed which contained the (x, y) coordinates (in plan) of the centre of every pile group (including single piles) for both Tower 2 and Tower 3. The centre to centre distances of each group to every other group was calculated. The vertical stress in the NF at each pile group location due to all other pile groups was then calculated by using the appropriate stress polynomial and the load and diameter of each pile group.

The distribution of vertical stress in the NF at each pile group location was then estimated by summing all of the stress contributions for that group. A contour plot of the estimated increase in vertical stress at the middle of the NF is shown in Figures 4 and 5 for Towers 2 and 3 respectively. Column locations and identification numbers are shown in both figures. For Tower 3, the increase in vertical stress in the vicinity of the core was estimated to be in the order of 500 kPa. Based on the results of the oedometer tests, this stress increase is sufficient to load the NF to near its preconsolidation pressure. (The initial effective stress in the NF was estimated to be about 250 kPa. The addition of 500 kPa increases the final effective stress to 750 kPa).

The settlement at each pile group was obtained by applying the tri-linear stress - strain curve for the NF listed in Table 3. Settlement estimates were made for the short term (which included 1 log cycle of creep) and the long-term (which included 3 log cycles of creep). Settlements were calculated assuming a one-dimensional model. Consideration of the size of the loaded area with respect to the thickness of the NF indicates that a one-dimensional approach appears reasonable as the NF is a relatively thin layer in close proximity to the base of the piles. Nevertheless, the one-dimensional approach adopted may slightly over-estimate settlements. Comparison of the results of FLEA analyses with one dimensional analyses indicated the settlements estimated using the one dimensional analyses should be reduced by between 20% and 25%.

Allowance was also made within the spreadsheet for settlement of the MSG and siltstone, as well as for pile shortening.

Contour plots showing estimated short-term settlements using the tri-linear model (with settlement values reduced by 20% to account for 3-D effects) are shown in Figures 6 and 7 for Towers 2 and 3 respectively. Long term settlement estimates were about 20 % higher than the short term values.

On the basis of these settlement estimates, it was decided that Tower 2 would not require any remedial foundation works. However, the estimated settlements for Tower 3 were considered to be too high, and remedial foundation works were deemed to be necessary.

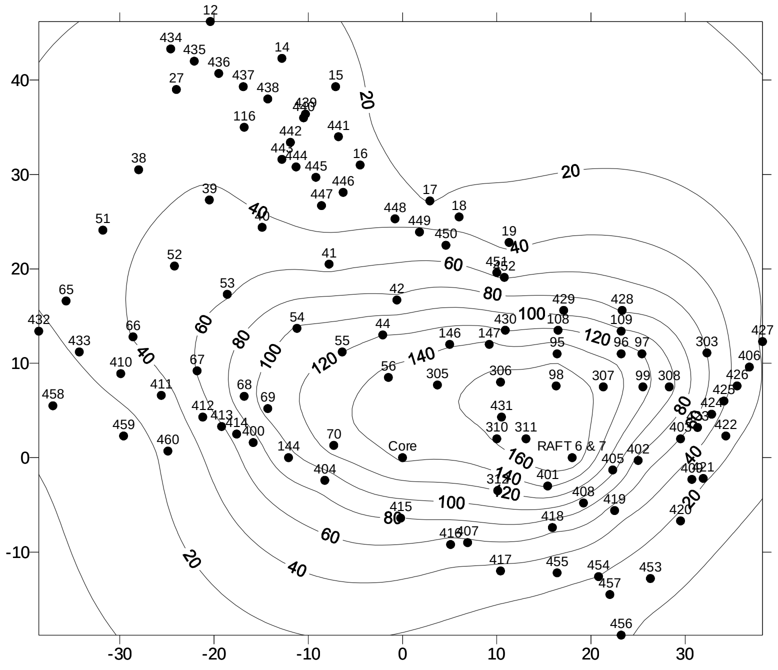

Figure 4: Estimated increase in vertical stress (kPa) at mid-height of NF for Tower 2.

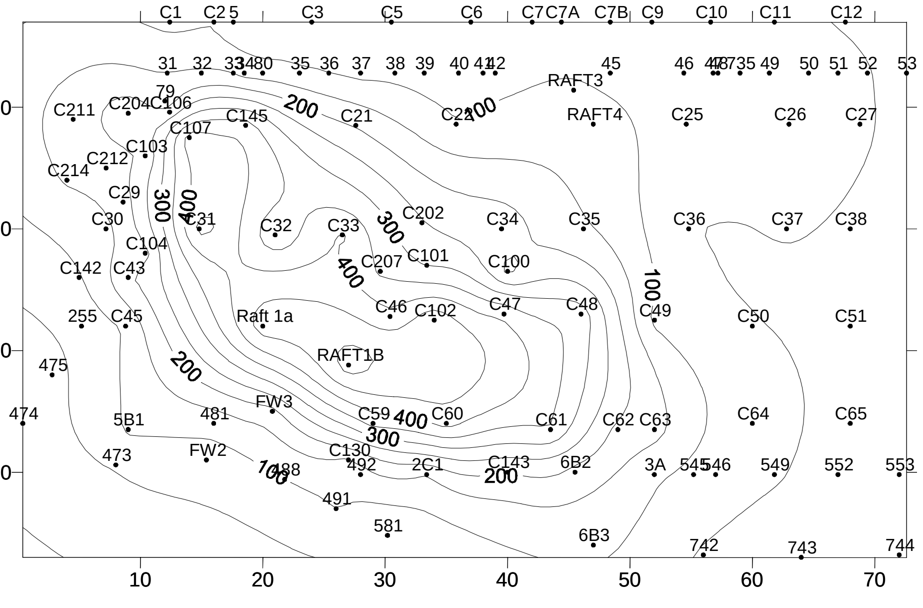

Figure 5: Estimated increase in vertical stress (kPa) at mid-height of NF for Tower 3.

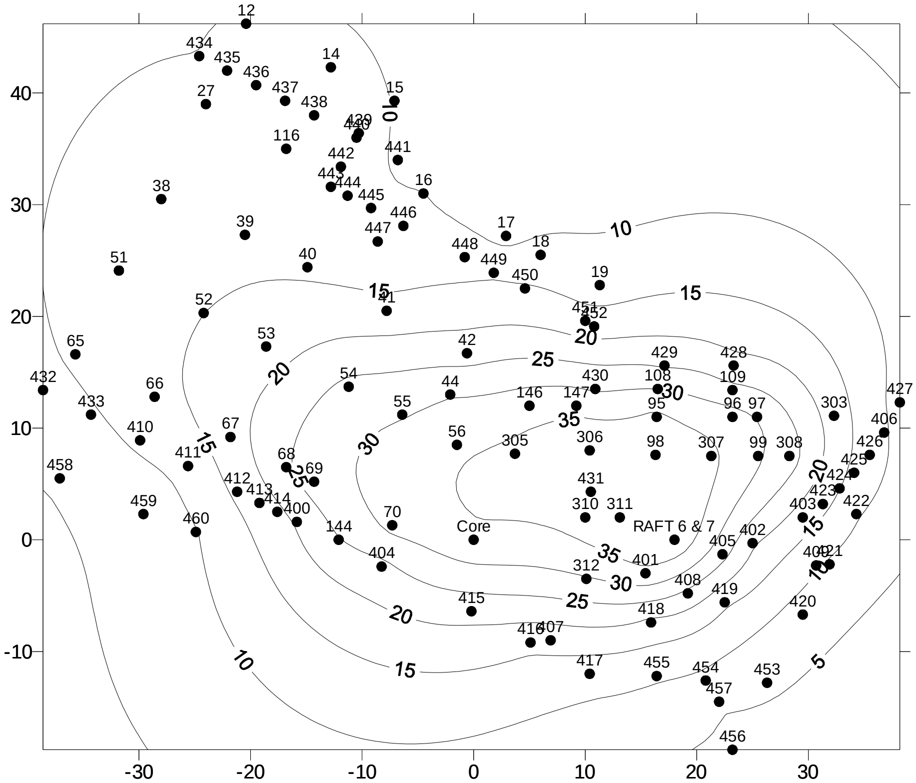

Figure 6: Estimated short-term settlement (mm) of Tower 2.

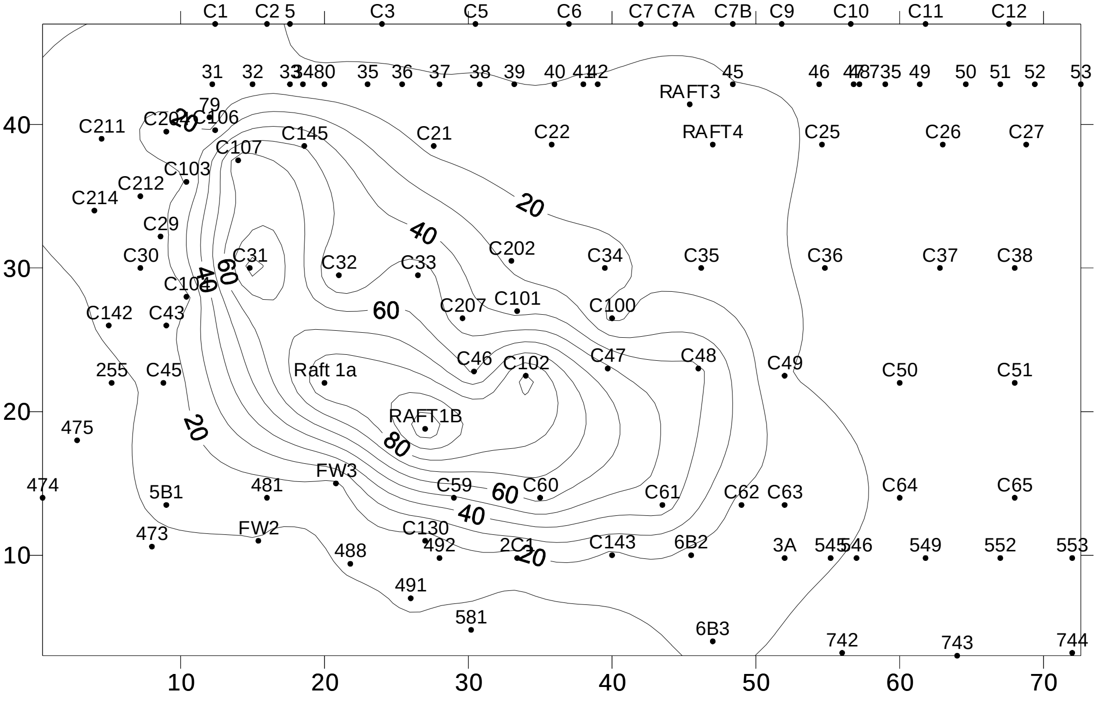

Figure 7: Estimated short-term settlement (mm) of Tower 3.

4 REMEDIAL WORKS FOR TOWER 3

Several remedial actions were considered including installation of additional piles to rock, preloading of foundations, removal of soil support (Poulos *et al.*, 2001), spanning between pile groups with a raft and jet grouting. The solution finally adopted was to supplement a number of the more heavily loaded footings with steel H-piles driven to refusal in the siltstone. It was assessed that by installing H-piles the increase in vertical stress experienced by the NF due to construction of Tower 3 could be reduced, thereby reducing settlements.

HoweverThe 8th ANZ Young Geotechnical Professional Conference

The 8th Australia - New Zealand Young Geotechnical Professionals Conference is a joint initiative between the Australian Geomechanics Society and the New Zealand Geotechnical Society and will be held this November 5-8 in Wellington, New Zealand.

This Conference has attracted over 50 submissions of interest from across Australia and New Zealand from members of prominent consulting and contracting firms, industry bodies and research institutions. Generous sponsorship of the conference is being provided by a number of bodies and firms, namely: The New Zealand Earthquake Commission, ISSMGE, BECA, Coffey Geotechnics, Golder Associates, Griffiths Drilling, OPUS International, Riley Consultants, Tonkin & Taylor, Broons Ltd, Connell Wagner, Geotechnics, Geotek Services, Maunsell, Pro Drill and Websters Drilling.

The aims of the 8th Young Geotechnical Professionals Conference are to:

- Promote the professional development of delegates through sharing of ideas and experiences, by each delegate preparing a written paper and presenting their paper to peers and senior professionals.
- Provide a forum at which delegates can benefit from the insight of acknowledged experts within the profession
- Expand and strengthen the lines of communication between young professionals across Australia and New Zealand within the field of geomechanics
- Promote an enhanced perspective of the varied roles, responsibilities and opportunities encompassed by the geotechnical profession
- Encourage the exchange of knowledge between consultants, research institutions, contractors and industry associated with the geotechnical field.

The conference will draw upon the experiences of the seven previous Australia – New Zealand Young Geotechnical Professionals Conferences (Sydney 1994, Auckland 1996, Melbourne 1998, Perth 2000, Rotorua 2002, Gold Coast 2004 and Adelaide 2006), to provide an enjoyable and informative occasion from which Australia's and New Zealand's future leaders in the geotechnical profession will benefit.

This 3 day live-in conference will include a keynote speech by Professor John Atkinson, who organised the inaugural Young Geotechnical Engineers Conference at City University, London, UK in 1981. (Note that, uniquely in the world, and reflective of the nature of the profession in Australia and New Zealand, the ANZ conference includes Engineering Geologists as well as Geotechnical Engineers, hence the use of the term "Young Geotechnical Professionals" or YGP, rather than "Young Geotechnical Engineers" or YGE).

All delegates have prepared a technical paper and will be asked to give a ten minute presentation followed by a question and answer session at the conference. In addition, the delegates will have a field trip visit of some of the particularly interesting geological and project sites around Wellington city and a conference dinner.

Two awards, the *Don Douglas Youth Fellowship* (AGS Members Only) and *NZGS Young Geotechnical Professionals Fellowship* (NZGS members only) will be made to those judged to present the best papers of the conference. The award recipients will receive financial assistance to attend the next International Young Geotechnical Engineers (IYGE) Conference in Cairo, Egypt in 2009. several geotechnical issues still required resolution, including

- could H-piles be driven hard enough to penetrate to the siltstone?
- how many H-piles are required and at which columns?
- how could the interaction between H-piles and the existing floating footings (pile groups) be assessed and what is the proportion of load carried by each. Would the H-piles be over stressed as they are founded on rock?
- how is the interaction between adjacent footings which may or may not contain H-piles to be assessed?

- how can the composite H-pile/floating footings be analysed so there is sufficient confidence in the estimated performance?
- what impact does variability in the thickness and properties of the various soil units have on the assessed performance of the composite footings?

The piling contractor carried out pile driving tests in several areas already densely populated with existing precast piles to confirm that the H-piles would penetrate to the siltstone. The installed H-piles were dynamically tested and the results analysed using CAPWAP to assess their load-settlement characteristics. The load versus settlement curves obtained from the CAPWAP analyses were analysed to obtain estimates of secant pile stiffness. Stiffness values varied with load level and from pile to pile with values between about 0.12 MN/mm and 0.16 MN/mm being obtained.

The settlement performance of the composite footings was modelled by modifying the EXCEL spreadsheet described earlier to account for the presence of H-piles within individual footings. This was achieved by the following iterative process :

1. The stresses and settlements within the NF were calculated using the spreadsheet analysis and ignoring the presence of the H-piles.
2. The stiffness of each (floating) footing was assessed by dividing the load applied to the footing by the calculated footing settlement.
3. For footings that contained H-piles and/or existing precast piles to rock, a new total footing stiffness was calculated by modelling the floating footing and the H-piles and precast piles as springs acting in parallel.
4. The proportion of column load carried by the floating footing, H-piles and precast piles was calculated according to the ratio of their individual stiffness values to the total stiffness of the composite footing.
5. The loads applied to the floating footings were corrected to their new (reduced) values and the vertical stress in the NF calculated. The settlements of the floating footings were then calculated.
6. The steps 2 to 5 were then repeated until convergence was obtained.

The above calculation process was set up within EXCEL so that it ran automatically without the need for user intervention. Typical input and run times per analysis were less than 5 minutes. The process converged rapidly and provided estimates of vertical stresses in the NF, settlement at every footing location and loads in the H-piles. Figure 8 shows contours of the increase in vertical stress at the mid-height of the NF due to the footing loads for an analysis that includes 76 H-piles. Figure 9 shows contours of estimated short-term settlement for the same analysis. Maximum H-pile load was estimated to be 2.7 MN in the short-term and 3.3 MN in the long-term which were satisfactory with respect to long-tem durability and structural requirements.

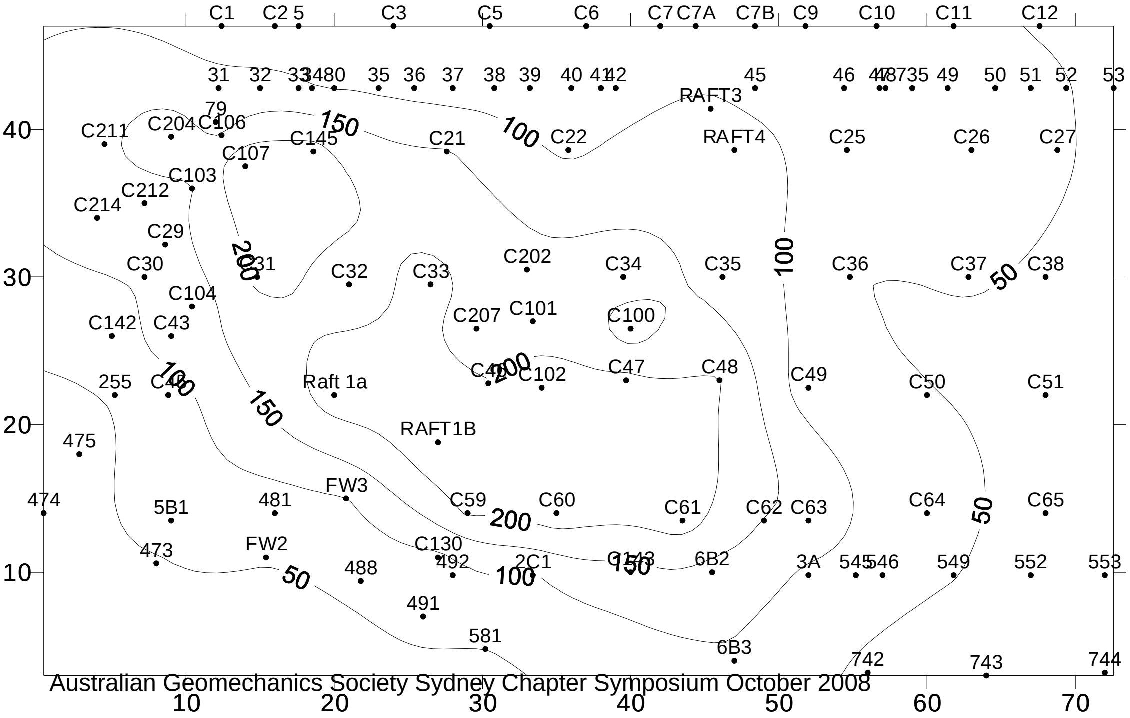

Figure 8: Estimated increase in vertical stress (kPa) at mid-height of NF for Tower 3 after installation of H-piles.

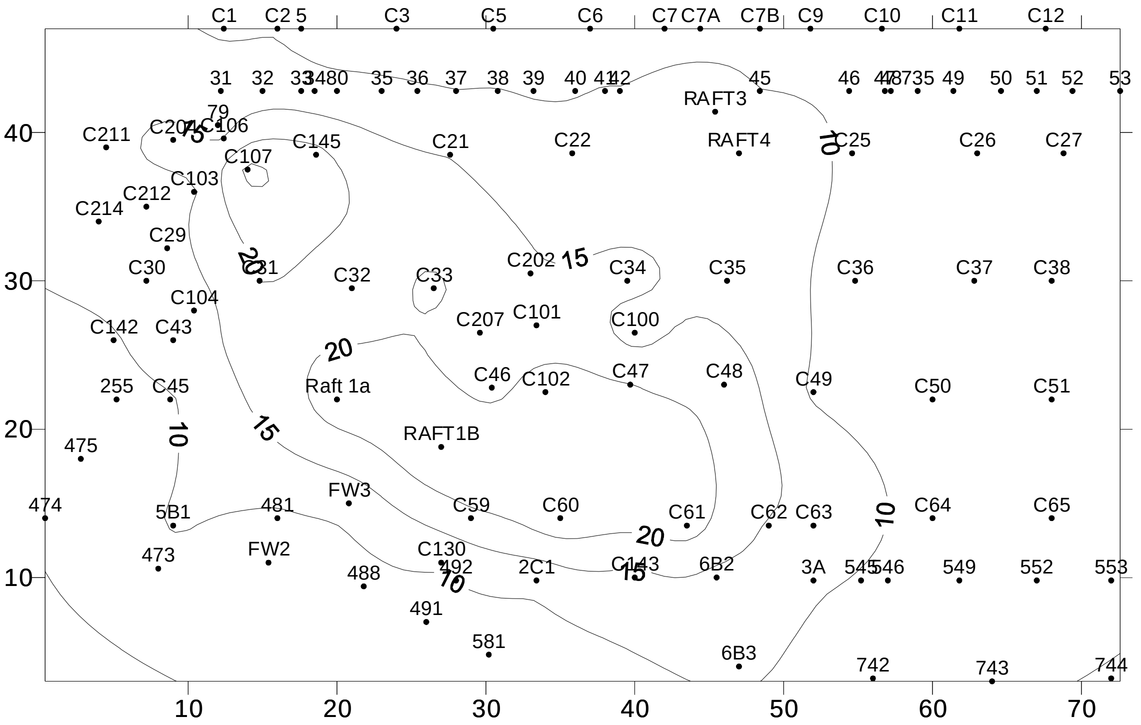

Figure 9: Estimated short-term settlement (mm) of Tower 3 after installation of H-piles.

Using this calculation process it was possible to rapidly assess a wide range of variables. In particular, it provided a rapid method to assess the impact of the following parameters :

- Number and location of H-piles
- H-pile stiffness
- Variability is the thickness and properties of the NF and MSG

The final H-pile configuration that was adopted included 73 H-piles spread over 26 footings. The calculated results shown in Figures 8 and 9 are for this configuration.

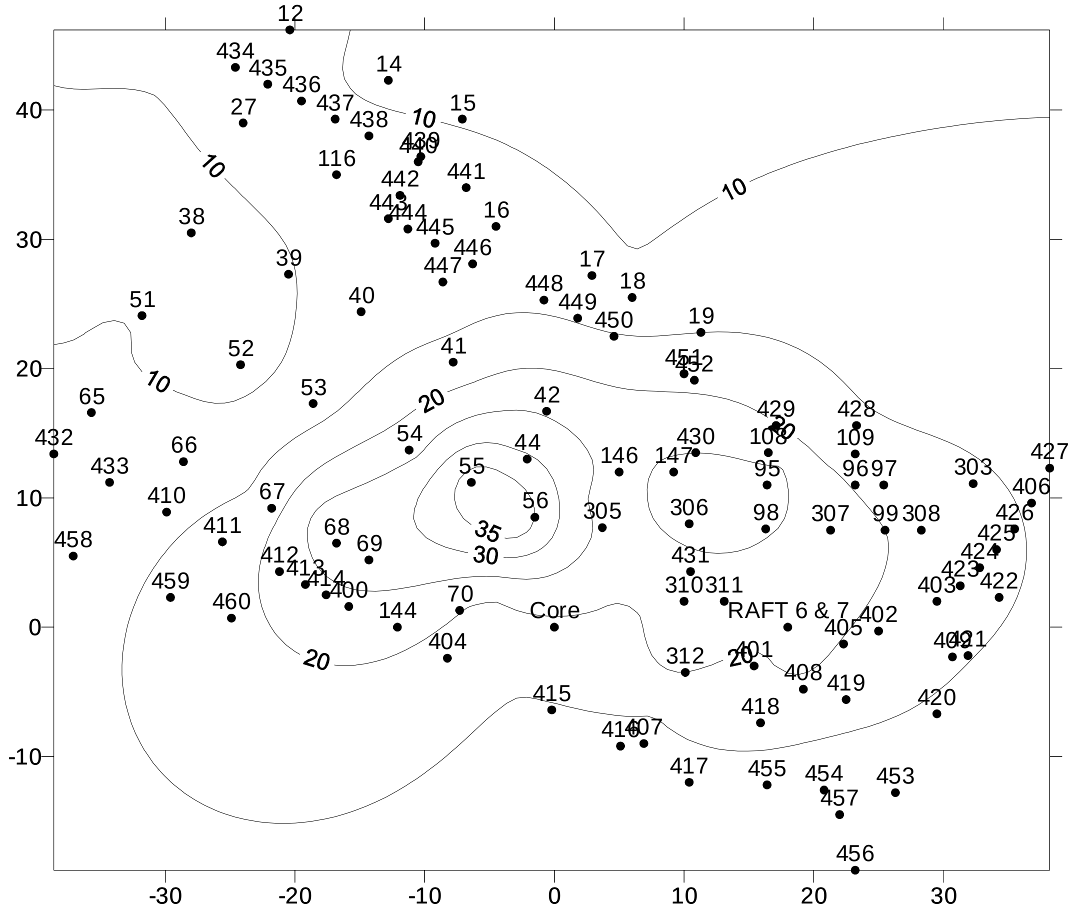

Figure 10: Measured settlement of Tower 2 at end of construction.

5 SETTLEMENT MONITORING

The settlement of selected columns was monitored throughout construction of both towers. Figure 10 shows a contour plot of measured settlements at the end of construction of Tower 2. These can be compared to the estimated settlements shown in Figure 6. The column location and numbers are indicated on the contour plots. The maximum settlements that have been measured are in the vicinity of Columns 55 and 56. Measured settlements were about 35 mm. Estimated settlement at these locations was about 24 mm to 28 mm. The difference between estimated and measured settlements for these two columns can be attributed to heave of the precast piles following driving of adjacent piles. Most of the precast piles were restruck following installation of the H-piles and before pouring of the pile cap to minimise the effects of pile heave. Measured heave for piles that were restruck varied between 0 mm and 16 mm. However, at columns 55 and 56 it was not possible to restrike the piles as the pile cap had already been poured.

Figures 11 to 13 show comparisons of total measured and estimated settlements with time for selected columns since completion of Level 9 of Tower 2. The estimated settlements were based on estimates of column loads at various stages during construction (at the end of construction of Levels 9, 12, 17 and 21 [roof]) provided by the structural engineers. Figures 11 to 13 show reasonable agreement between estimated and observed settlement rates. The settlement increase per Tower level in the most heavily loaded columns was less than about 1.5 mm per level. The estimated column settlement increases per level generally agree well with the observed values. However for some column locations (e.g. columns 55 and 56) observed settlement increases per level are slightly higher than estimated.

Monitoring of settlements also occurred at Tower 3. In general the measured settlements up until Level 17 (after which monitoring data was no longer available) agreed closely with our estimated values. Measured settlement rates varied between 0.1 mm per level for the lightly loaded columns to 0.7 mm per level for the most heavily loaded columns.

Figure 11: Measured versus estimated settlement at Raft 6 and 7.

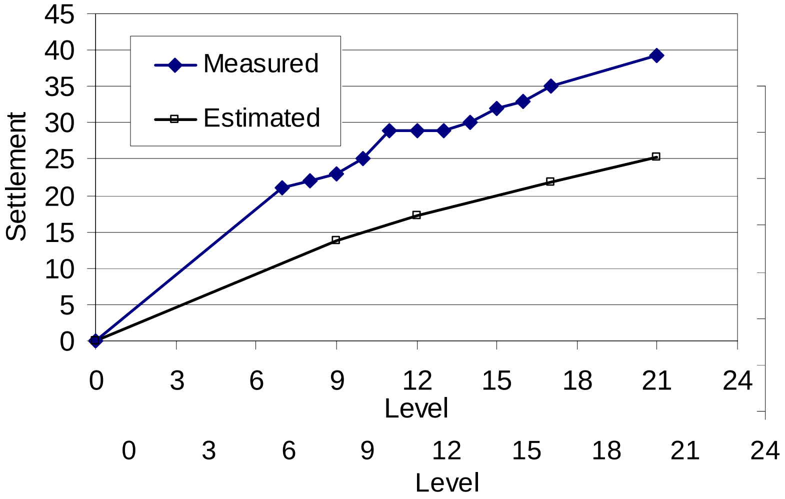

Figure 12: Measured versus estimated settlement at Column 98.

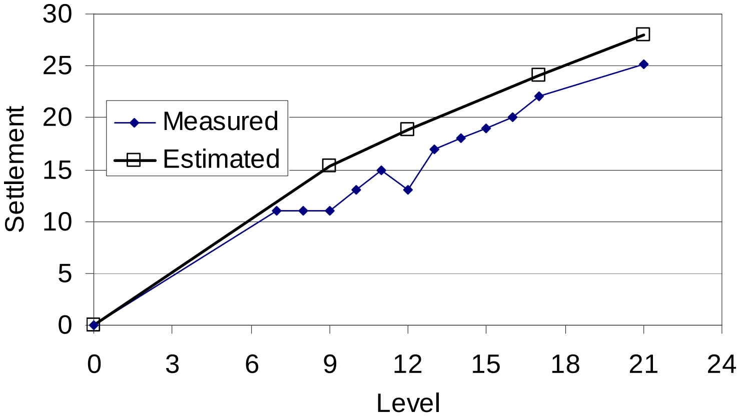

Figure 13: Measured versus estimated settlement at Column 56.

6 SUMMARY

This paper presents a settlement case history of two residential towers that experienced problems during pile installation. The problems were identified following completion of the foundations but prior to construction of the tower structures. Accurate settlement analyses were required to assess whether remedial measures were necessary to stiffen up the foundations to offset the potential risk of unacceptably high settlements. The settlement analyses carried out were based on a fundamental understanding of the issues which may have led to the increased settlements and were successfully modelled using a combination of standard engineering tools which were able to model the interaction between different foundation elements, the consolidation and creep behaviour of the soil and the three-dimensional aspects of the problem.

On the basis of these settlement estimates, additional piles were installed for one of the towers. Measurements of settlement during construction indicated settlements that were close to estimated values, thereby justifying the decisions that were made.

7 REFERENCES

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