Geotechnical Engineering for Condor Tower

D. P. Stewart[1], FIEAust.

[1]Golder Associates Pty Ltd, 1 Havelock Street, West Perth, WA 6005; Ph (08) 9213 7600; email: dpstewart@golder.com.au

ABSTRACT

The main apartment tower of Condor Tower at 22 St Georges Terrace in Perth comprises 19 new floors constructed on top of a pre-existing nine-storey fire-damaged building. The pre-existing building was supported on shallow pad footings. A complex foundation solution and construction sequence was required, including micro-piling using a low-headroom piling rig, grout underpinning of the existing footings, cutting of the footings to significantly reduce their width, excavation to below footing level and construction of a raft that was tied into the footings. An associated 11-storey commercial building was built within an adjacent excavation that had been abandoned during a previous stage of development about 20 years prior. The construction of this building was complicated by the presence of existing angled struts that had been used to support the diaphragm walls when the previous development was abandoned and the limited structural strength of the old diaphragm wall. The design of the foundation and retaining solution and the construction methodology adopted in the various parts of the project are described. A back-analysis of pile load test results from within the basement is presented together with the results of pile load monitoring from during the tower construction period.

Keywords: piled raft, underpinning, pile load test, pile load monitoring, sustainability

1 INTRODUCTION

Condor Tower at 22 St Georges Terrace in Perth is an award-winning urban renewal project that was completed in 2008 after commencing design in 2003. The main apartment tower on St Georges Terrace comprises 19 new floors constructed on top of a pre-existing nine-storey fire-damaged building that was supported on shallow pad footings. In order to retain the basement of the pre-existing building, a complex foundation solution and specific construction sequence was required. In addition, an associated 11-storey commercial building was built within an adjacent diaphragm-wall-supported excavation that had been abandoned during a previous stage of development about 20 years prior.

The development site extends from St Georges Terrace through to Hay Street. Prior to the Condor Tower development, the St Georges Terrace frontage was occupied by a nine storey building, known as the Oakleigh Building. The building was H-shaped above the ground floor, with a central service core joining the northern and southern sections of the building. A single basement level covered the site, with a floor level generally about 1.5 m below St Georges Terrace level. The building was supported on shallow pad footings founded within sand just above groundwater level at about 1 m below the basement floor. It was necessary to retain and refurbish the existing building while extending the southern section of the building upwards.

The northern part of the Oakleigh Building was adjacent to an approximately 7 m deep open excavation that extended through to Hay Street. The excavation was retained by old 0.5 m thick diaphragm walls supported with a series of diagonal props extending down to strip footings within the base of the excavation. The diagonal props had been installed after a previous development had been abandoned in about 1989 and before cutting of the temporary ground anchors that had been supporting the wall up to that time. Two deep barrettes had also been constructed within the area of the excavation, for the previously abandoned project. The base of the excavation was at a level roughly corresponding to groundwater level and had vegetation growing within it.

Design of the new structure to make use of the old structure involved a significant number of geotechnical challenges, as summarised in this paper and structural engineering challenges (Psaltis and Chidgzey, 2010 and Tyler, 2012).

2 SUBSURFACE CONDITIONS

Completion of a geotechnical investigation at the site was difficult due to space and headroom constraints around and within the site. A number of cone penetration tests, a flat plate dilatometer test and shear wave velocity measurements were carried out using low-headroom equipment, plus several boreholes where feasible. This data was used to supplement information from previous investigations at the site. The subsurface conditions at the site are illustrated in Figure 1. The conditions are relatively typical of conditions encountered elsewhere within the Perth CBD, except that the strength and stiffness of the upper clay unit (s_u ~130 kPa) was found to be about half that generally observed further to the west along St Georges Terrace. This clay unit was present about 3 m below the base of the existing shallow footings below the tower footprint. The alluvial soils within about 15 m below the base of the existing footings were found to be highly variable, as evident at a number of other sites further to the east along Adelaide Terrace, the extension of St Georges Terrace. The clayey soils were found to have softened below the base of the abandoned excavation (s_u ~40 to 100 kPa).

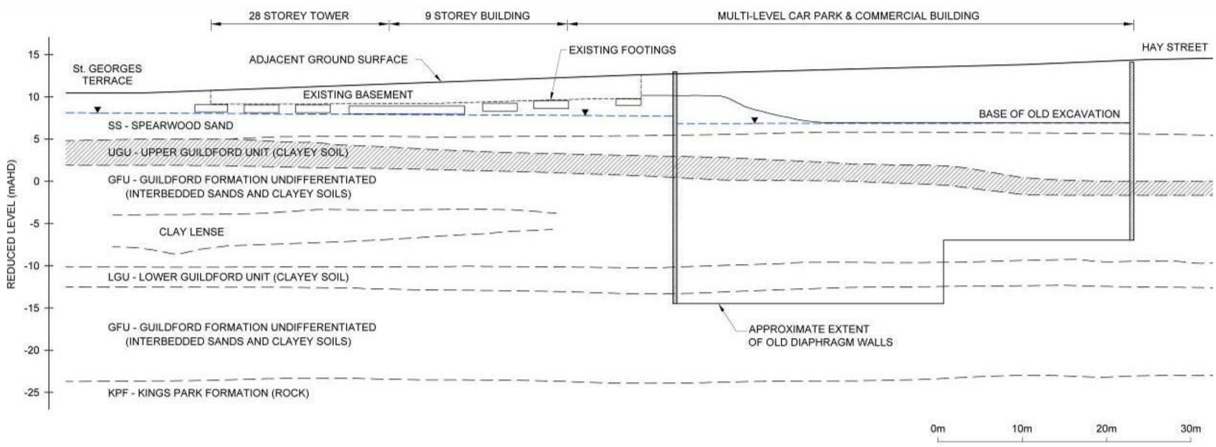

Figure 1. Schematic of subsurface conditions (not to scale)

3 FOUNDATION SOLUTION FOR TOWER

One of the main constraints for development of a foundation solution for the tower was that the basement must remain in the completed structure. This constraint dictated that a new raft footing could not be constructed by filling part of the height of the basement with a reinforced concrete raft. Options to support the new structure on a raft, piles or a piled raft were considered. Full support on piles was considered to be problematic due to the severe headroom constraints (2.2 m) that would restrict the size of piling equipment and the complexity of the work associated with developing a structural connection between the new piles and the existing columns within the space constraints. A raft alone was considered in some detail, although this was discounted primarily due to the difficulty of constructing a raft of the required thickness (about 2 m) and plan dimensions while supporting the building above. A further complicating factor was the relatively low strength and compressibility of the upper clay unit that led to some concern over differential settlement and the effect of relatively high raft bearing pressures.

A partially piled raft solution was developed to fit within the site constraints. The purpose of the piles was to provide additional support to the raft to reduce differential settlement and reduce the required thickness of the raft itself. This latter effect was of major importance so that the depth of excavation could be minimised, given the requirement to support the overlying building while the raft was constructed. Illustrations of the existing shallow pad footings and the arrangement of the new raft are shown in Figure 2.

To construct the raft it was necessary to cut back the existing pad footings to nearly half their width and then excavate to a level below the base of the footings within the space between the footings (Figure 3). This led to an increase in bearing pressure from a range of about 200-220 kPa to about 450-550 kPa. This work was enabled by first carrying out microfine cement permeation grout underpinning below the sections of each footing to be retained. A grouting trial was carried out in advance of the main program of work to enable the grout penetration and strength to the measured and to confirm the required spacing and grout injection rate. A grout column of about 0.6 m diameter (increasing with depth) was readily formed. The measured uniaxial compressive strength of the grouted sand was about 2 MPa. The grout spears were inserted on about 0.5 m spacing through about 25 mm diameter holes cored through the footings and typically extended to 1.6 m below the underside of the footings. The outer part of the footings was then diamond-wire cut and removed before excavation to the underside of the raft, typically to about 0.4 m below the underside of footings.

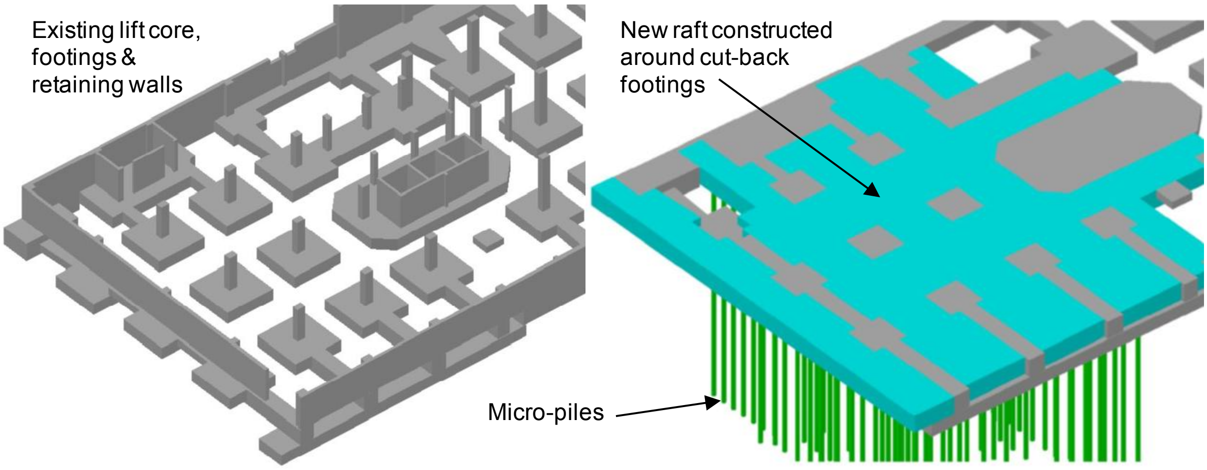

Figure 2. Existing shallow footings and new piled raft

The actual construction sequence was relatively complex, specified in detail and closely controlled on site so that strips of the raft running across the site were constructed in a defined sequence. This sequence was designed so that the footing cutback and subsequent excavation adjacent to any footing was only carried out along one side of each footing at a time to avoid overstressing of the footings and grout underpin and to minimise risk. The new 1.4 m thick raft was dowelled into all existing footings so as to effectively act as a monolithic raft when completed.

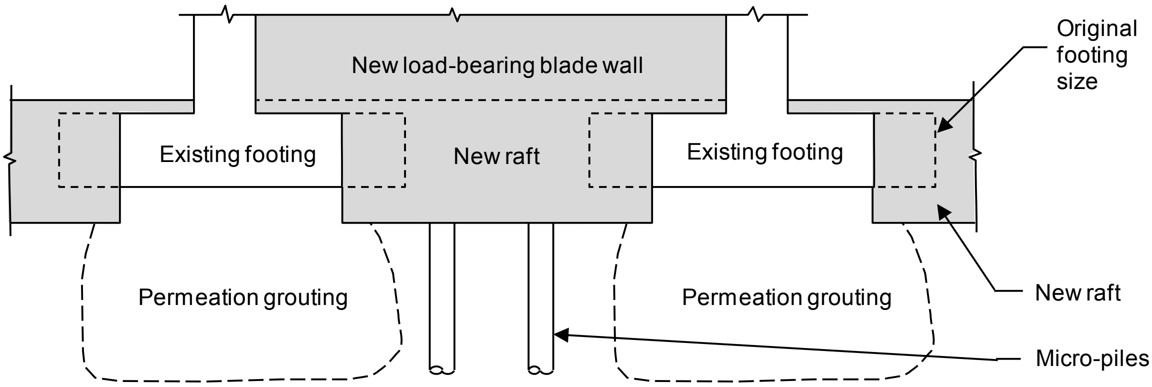

Figure 3. Detail section through new piled raft

Piles were installed using a specialist micro-piling rig held in position by jacking against the underlying and overlying floor slabs. The positions of the piles were selected to span across areas where several heavily loaded new blade walls would be constructed (Figure 4). To finalise the foundation design, a piling trial was carried out in advance of the production piling. Four piles, two 7 m long and two 10 m long, were subjected to static load testing. The piles were installed in two separate locations, so as to assess the variability in pile response across the site, given the variable ground conditions. Strain gauges were attached to the reinforcement cage to assess the split between base and shaft resistance. The results of the testing were valuable in providing confirmation of shaft and end bearing resistance and pile head stiffness and in assessing the practicability of the pile installation method. The results of load testing on the 10 m long trial piles are shown in Figure 5. Analysis of the load test data using various approaches indicated the parameters summarised in Table 1. Following assessment of the test pile results, 12 m long piles were specified for construction. About 90 reinforced grout-injected continuous flight auger piles of 300 mm diameter and 12 m length below underside of raft (13.2 m below piling platform) were installed.

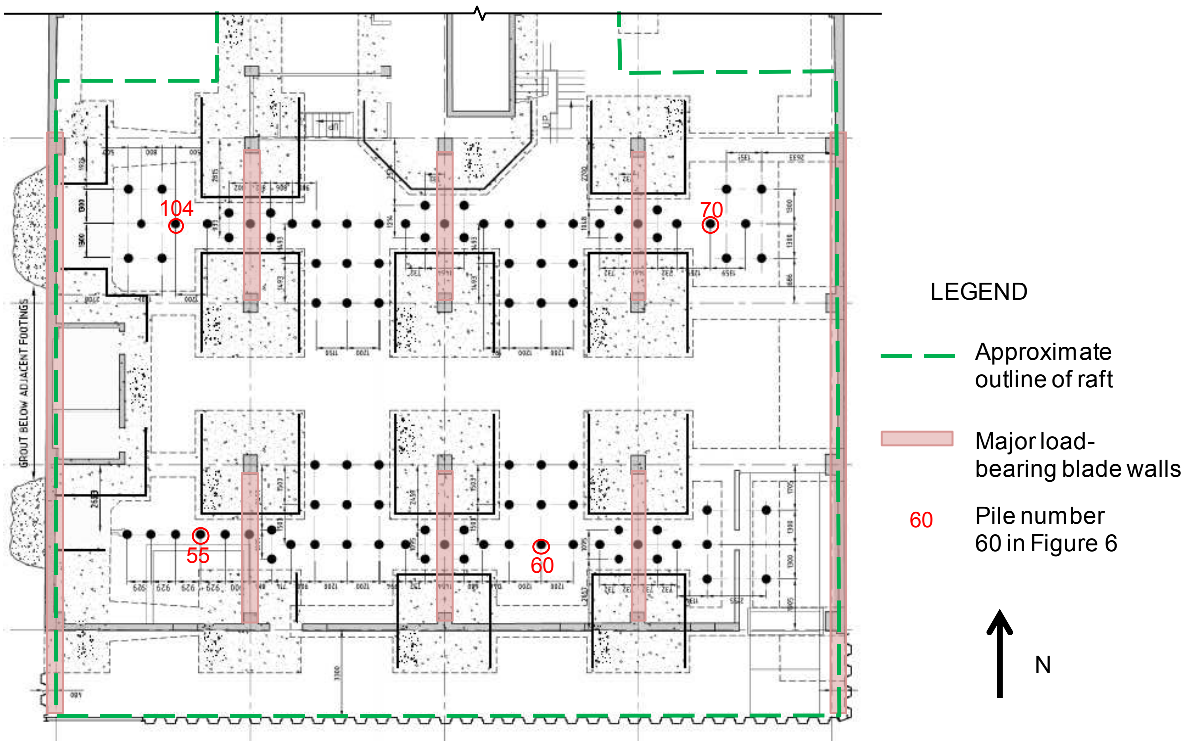

Figure 4. Micro-piles below new piled raft

Table 1: Summary of pile load test results

No.	Area	Length (m)	Load at 10% defl. (kN)	Average $f_{s.ult}$ (kPa)	Average q_c over shaft length (MPa)	Mean q_c near pile toe (MPa)	Ratio $q_{c.ave}/f_{s.ult}$	Initial pile head stiffness (kN/mm)
1	South	10	750	50	7.5	4	150	300
2	South	7	430	40	7	2	175	150
3	North	10	870	77	7.9	2.2	103	300
4	North	7	675	78	8	2.5	103	300

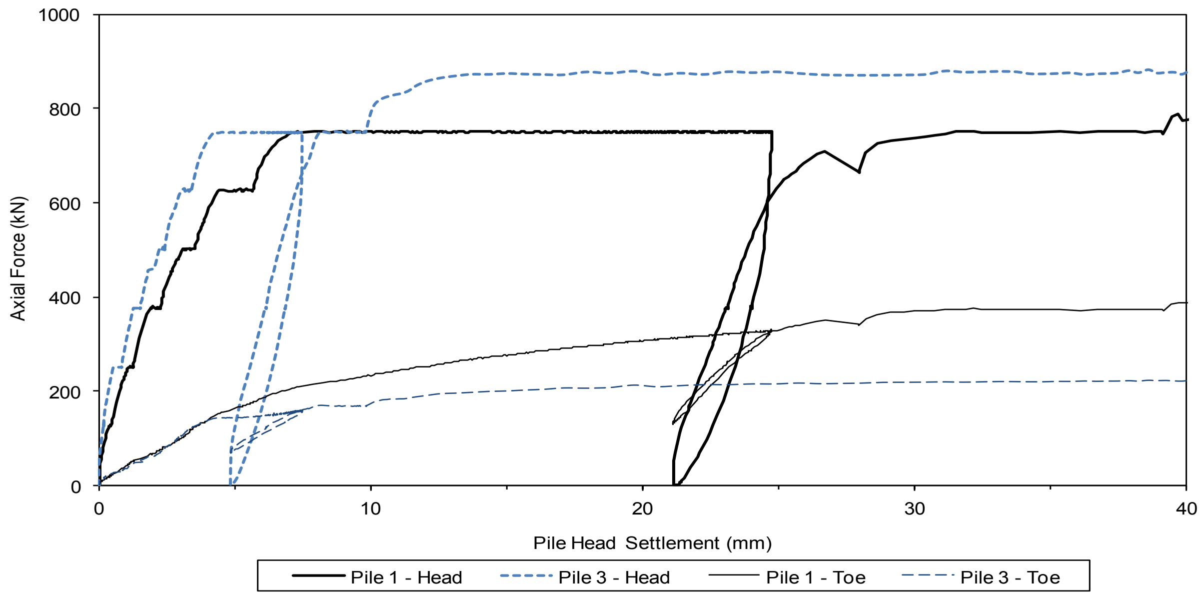

Figure 5. Pile load test results on 10 m long trial piles

A number of analysis approaches were used to aid in developing the design solution for the tower but these will not be fully described here. To assess the partially piled raft, analysis was carried out with the APRAF software developed at the University of Sydney and was supplemented with analysis of the piles using PIGLET, developed at the University of Western Australia. To overcome the limitations of the various analysis approaches, numerous sensitivity analyses were carried out to account for both variability and uncertainty in the geotechnical parameters and to account for factors that were not represented in the software. Based on the analyses, the 12 m long micro-piles were designed for an ultimate factored structural load of 1400 kN. Since the piles form part of a partially piled raft, the geotechnical strength of the piles themselves is of secondary importance to the pile head stiffness. The analyses were updated after interpretation and back-analysis of the trial pile load tests had been completed, to give greater confidence in the design solution.

4 PILE LOAD MONITORING BELOW TOWER

Strain gauges were placed on the reinforcing cages of eight piles just below the underside of the raft, to enable pile loads to be monitored over about a 2½ year period as the structure was built. The axial load in selected piles is shown in Figure 6. The range of measured pile loads compares favourably with the design estimates, which were generally in the range of 800 to 1100 kN. At the end of the monitoring period, it is unlikely that the full design live load had been experienced and therefore the long-term in-service pile loads would be expected to be slightly higher. Unfortunately no settlement monitoring data is available, despite efforts to collect this information.

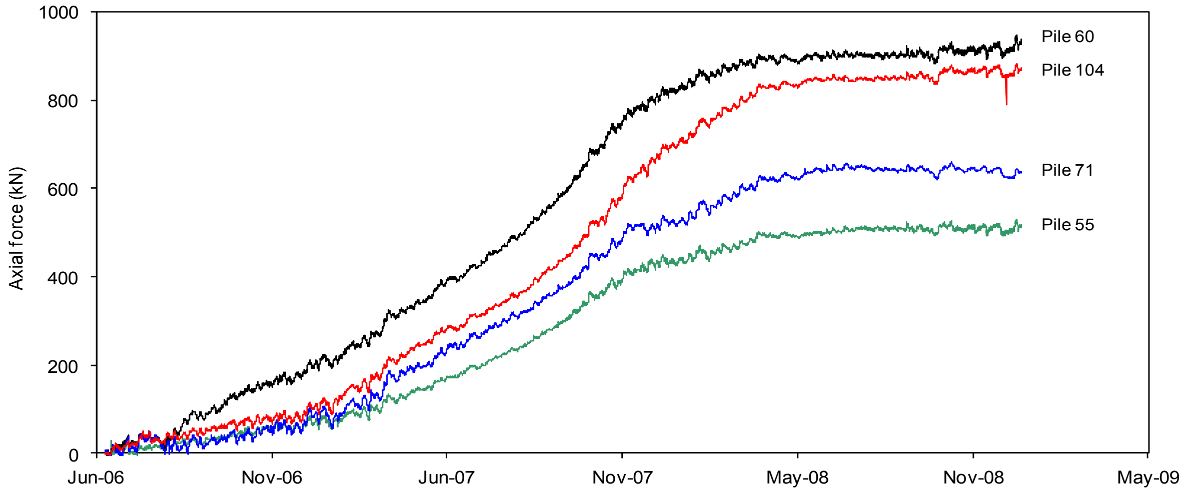

Figure 6. Pile loads measured during construction of the tower

5 CONSTRUCTION OF HAY STREET BUILDING

The Hay Street building was constructed within an abandoned open excavation supported by propped diaphragm walls, after abandonment of a previous development nearly 20 years earlier. The props were angled steel struts supported on strip footings founded within the base of the excavation and with tie beams running across the excavation base (Figure 7). Temporary ground anchors supporting the walls had been de-stressed after preloading the struts by jacking. The presence of the struts and footings complicated the development of the site because of the need to either maintain the struts in place while the new structure was built, or to provide additional support to the diaphragm walls before removing the struts. The initial development proposal was to extend the excavation a full basement level deeper to accommodate four basement levels. This initially seemed attractive since the old diaphragm walls extended relatively deep. However, it was found that the walls had inadequate bending strength to enable a deeper excavation to be supported. Following assessment of various options, a half-basement level deepening was designed over only the southern part of the site.

Within the southern part of the basement, a new section of 600 mm diameter secant CFA pile wall was constructed across the site, near the rear of the existing building. The interface between the secant piles and the diaphragm walls at each end proved problematic and required significant attention to remedial grouting to enable a reasonable joint to be achieved. Drilled and grouted ground anchors were then installed to support the southern section of the diaphragm walls to enable the steel struts supporting this section of the walls to be removed. The anchors were designed to avoid interference with the old de-stressed anchors remaining in the ground.

Figure 7. Steel struts supporting diaphragm walls; photograph taken after initial site cleanup

Within the northern part of the basement, 600 mm diameter CFA piles were installed to support some parts of the structure and as part of a piled-raft element in the base of the excavation in one area (to work around the old struts and footings). Micro-fine cement grout underpinning was carried out below the strip footings supporting the steel struts to facilitate adjacent excavation for pile caps and raft elements extending about 1 m below the underside of the footings. This grouting was carried out fully below the watertable and was successfully completed in most areas. However in one section up to three injection episodes were required before sufficient grout penetration was achieved.

The permanent basement floor slabs were constructed in the northern section of the basement with temporary blockouts in the floor slabs around the steel struts. After the permanent structure reached ground level, the steel struts were removed. This construction sequence had the advantage of minimising adjacent ground movements, resulting in no known damage to an immediately-adjacent 8-level building supported on shallow footings.

6 CONCLUSIONS

The Condor Tower development with its associated structures was a challenging project that required the application of relatively complex construction techniques under close supervision. The installation and load testing of micro-piles with only 2.2 m headroom was a key component of the foundation solution that made the construction of a thinner raft feasible. Micro-pile loads were monitored over a 2½ year period as the structure was built, indicating loads that were in-line with design expectations. Extensive use of micro-fine cement permeation grout underpinning enabled existing footings to be cut back to significantly reduce their thickness and excavate adjacent to below footing base level. The project is an excellent example of urban renewal of an old structure, leading to a lower cost structure and being construction time-neutral compared to demolition and fully new construction, Tyler (2012).

7 ACKNOWLEDGEMENTS

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8 REFERENCES

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