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Adding new levels to old buildings: A case study in foundation assessment

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ABSTRACT: This paper presents a case study of geotechnical investigation and construction stage survey monitoring for the addition of a new level to a multi-level car park in the eastern suburbs of Melbourne. This building is of interest because of the variability in the subsurface conditions and footing type beneath its footprint and the availability of survey monitoring data during construction works. The western half of the building is built over deep uncontrolled fill and is supported by piles socketed into weathered rock. The eastern half is supported by pad footings in an area where the depth to the rock is relatively shallow. The practical constraints associated with the assessment and the methodology adopted to manage the risk of uncertainty in the ground and footing conditions are discussed. Additional settlement of less than 2 mm was measured following an approximate 25% increase in the footing loads, which provided validation of the geotechnical methodology adopted.

1 INTRODUCTION

As buildings in urban centres approach the end of their design life there is increasing pressure to modify or refurbish them (rather than demolish and re-build them), which can change the loads acting on the building footings. In many cases, information regarding the initial footing design and construction (including records of geotechnical investigation and inspections undertaken during construction) is not available and there can be significant uncertainty in how the building footings will respond to additional loads. There are also practical constraints on geotechnical investigation and assessment of existing building footings.

This paper presents a case study of geotechnical investigation and construction stage survey monitoring for the addition of a new level to the multi-level Car Park 6 (CP6) building at the Burwood Campus of Deakin University in the eastern suburbs of Melbourne. The geotechnical works discussed in this paper include a desktop review of the available construction drawings and site history information, a visual assessment of surface conditions and an intrusive investigation (comprising the drilling of boreholes using a low-headroom drill rig within the building adjacent to pile footings and through pad footings). The approach taken to managing the risk of uncertainty in how the footings were built and the results of survey monitoring obtained during construction of the CP6 extension are also discussed.

Golder Associates Pty Ltd (Golder) was engaged by Deakin University to provide geotechnical consulting services for the CP6 extension project. 4D Workshop Pty Ltd (4D Workshop) was the project structural engineer. Kane Constructions was the building contractor. The geotechnical investigation was performed in 2015 with the CP6 extension project completed in 2016.

2 PROJECT SETTING

2.1 *Location*

The Burwood Campus of Deakin University is located about 15 km east of the Melbourne CBD. A watercourse (Gardiners Creek) flows through the central part of the Burwood Campus. The CP6 building is located within the northeast part of the Burwood Campus, south of Gardiners Creek.

2.2 *Geology*

The Geological Survey of Victoria 1:63,360 scale ‘Ringwood’ mapsheet indicates that the CP6 building is underlain by Silurian age Andersons Creek Formation materials comprising interbedded siltstone and sandstone overlain by residual siltstone soils (typically clay). The geological map indicates that Quaternary age alluvium may be present near Gardiners Creek. Fill materials of variable nature and thickness are present over the natural materials at the CP6 site.

2.3 *History of development*

Construction drawings for the original CP6 building were provided by Deakin University and indicated that it was built in two stages in about 1997 (referred to herein as CP6 East) and 1998 (CP6 West). The original CP6 building comprised four suspended levels over a ground floor at grade.

A review of historical aerial photographs (refer Section 3.3) indicates that earthworks including the placement of fill were undertaken at the site between about the 1960s and 1980s. Prior to the construction of the CP6 building, the site appears to have been occupied by a sports oval (eastern half) and an at-grade car park (western half).

2.4 *CP6 Extension*

The CP6 extension project comprised the addition of one car park level over the original four-level building. 4D Workshop indicated that the additional level would increase the maximum column working load from about 2220 kN to about 2780 kN, an increase of about 25%.

3 GEOTECHNICAL ASSESSMENT

3.1 *Assessment approach*

A geotechnical assessment was performed to assess the redundant capacity of the existing footings of the CP6 building and the potential to increase the applied loads. This assessment included the following tasks, which are discussed in more detail in the following sections:

- A review of available construction drawings and other documents relating to the existing building.
- An intrusive geotechnical investigation.
- A visual assessment during a site walkover.

Assessment of structural aspects of the existing footings was performed by 4D Workshop concurrently with the geotechnical assessment. Structural aspects are not discussed in this paper.

3.2 *Construction drawings and documents*

Key information from the review of construction drawings is summarised below:

- The CP6 East columns are supported on concrete pad footings designed based on a maximum allowable bearing capacity of 300 kPa. The internal columns are typically supported on 2.9 m square footings; based on the loads calculated by 4D Workshop the bearing pressure applied to those footings prior to the extension would have been about 265 kPa, increasing to 330 kPa post-extension. Smaller pad footings (1 m to 2.6 m square) are located around the perimeter of the car park.
- The CP6 West columns are supported by reinforced piles which are typically 1.5 m in diameter, designed based on a skin friction of 60 kPa and allowable base resistance of 1200 kPa in ‘distinctly or less weathered mudstone’, and a minimum socket length of 1.8 m. Based on these design parameters the working load capacity of the minimum socket length would be about 2630 kN, which is above the design working load calculated by 4D Workshop for conditions prior to the extension, but about 95% of the post-extension design load. Contours of ‘minimum pile depth’ were indicated on the construction drawings.

The design drawings included reference to a geotechnical investigation for the original CP6 building. However, a copy of this report could not be obtained prior to the geotechnical assessment. A copy of the report was obtained after the extension project had been completed. Had the report been available earlier, it would have been of great value in the initial assessment and review of subsurface conditions. The design parameters indicated on the construction drawings were consistent with the geotechnical report recommendations.

Other notable information obtained during the review of construction records related to documentary evidence of construction of CP6 East in Bennetts et al (1999). Figure 1 shows a photograph from this publication which appears to confirm that the columns are supported on pad footings.

Based on the available information it was considered likely that the existing CP6 footings would have sufficient redundant capacity to accommodate the proposed load increase, taking into consideration the relatively small increase (less than about 10%) in the post-extension footing loads compared to their original design capacity.

Figure 1. CP6 East during construction (Bennetts et al, 1999).

3.3 *Intrusive geotechnical investigation*

The intrusive geotechnical investigation comprised the following works:

- CP6 East (pads): Drilling of boreholes at two locations (adjacent to internal and perimeter columns) to depths of about 3.3 m to 3.8 m. NMLC coring techniques were used to advance the bore-holes through concrete pad footings into the underlying founding materials.
- CP6 West (piles): Drilling of boreholes at two locations (adjacent to internal and perimeter columns) to a depth of between about 9.8 m and 10.5 m. Auger techniques were used in fill and natural soils and NMLC coring techniques were used in weathered rock. It was not considered practical or desirable to attempt to drill through the piles beneath the CP6 West columns.

The boreholes were drilled from the ground floor of the CP6 building using a specialist low headroom drilling rig. Headroom restrictions meant that a conventional drill rig would not have been able to operate within the carpark. Other practical considerations associated with the investigation included the coordination of drilling works with Deakin University within an operating carpark. These practical restrictions are common for investigation of existing buildings where modifications are proposed. Hence, intrusive investigation works for these projects need to be carefully planned recognising the limitations on site access and the requirements of the owners and tenants of the subject site and potentially adjoining sites. The productivity of a low headroom drill rigs is usually less than for a conventional drill rig hence investigation cost (per metre drilled) is usually higher. Note that the excavation of footing exposures was not a preferred investigation approach because it would not be practical for the piles, they would be more disruptive to car park operations than boreholes, and reinstatement of footing exposures in the car park would be more problematic than for small diameter boreholes.

There had initially been some uncertainty as to how successful drilling through the pad footings in CP6 East would be, due to concerns on drill bit wear, particularly should steel reinforcement be encountered. However, the investigation was successful at recovering continuous samples of the concrete footing and underlying founding materials, which varied from extremely to highly weathered siltstone in the two CP6 East boreholes. Figure 2 shows a photograph of the contact between the concrete footing and underlying weathered siltstone as observed in core samples from one of the CP6 East boreholes.

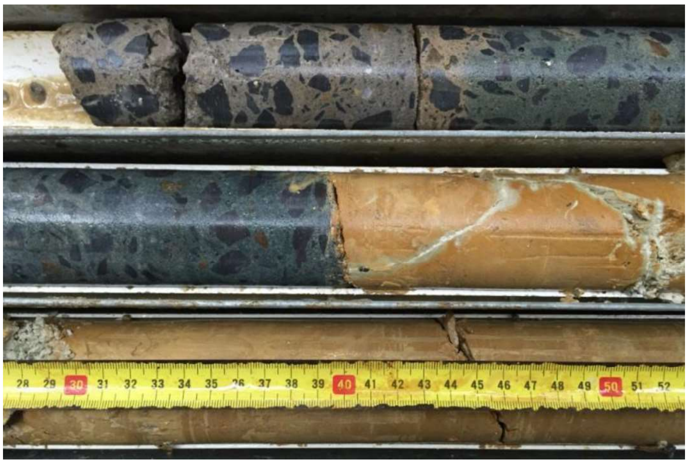

Figure 2. Core photograph showing the contact between the concrete footing and underlying weathered siltstone.

Key findings from the intrusive investigation are summarised as follows:

- CP6 East (pads): At the two investigation locations, the pad footings were founded on extremely or highly weathered siltstone. There was no evidence of loose debris between the base of the footing and the founding materials at the two lo-cations investigated.
- CP6 West (piles): The fill thickness ranged from about 3.0 m to 7.5 m. Waste materials including plastic, cardboard, metal and glass fragments were encountered within the fill indicating it to be uncontrolled. Highly to moderately or less weathered siltstone was encountered in both boreholes below about 7.4 m to 8.0 m depth.

The results of the intrusive investigation provided further evidence of redundant capacity of the CP6 East pad footings, given that the design maximum allowable bearing pressure of 300 kPa was below what is typically adopted for founding materials that comprise extremely or highly weathered siltstone. The investigation results also helped to provide confidence that the original construction methodology included cleaning of the base of the footing excavations prior to footing construction.

Within CP6 West, the significant depth of the fill and presence of waste materials had not been expected prior to drilling the boreholes. Subsequent review of historical aerial photographs provided confirmation that significant filling works had taken place (probably associated with backfilling and levelling of the Gardiners Creek floodplain, including re-routing of the creek) between the 1960s and 1980s (Figure 3). The elevation below which weathered rock was encountered also agreed well with the 'minimum pile level' contours on the original construction drawings. This provided confidence that the contours had been generated based on a geotechnical investigation of the depth to rock and that piles had likely socketed into the rock. Once the original geotechnical report had been obtained it was apparent that seven boreholes were drilled to depths of about 5.7 m to 11.8 m as part of the original investigation for CP6 West, with results consistent with the contours on the construction drawings.

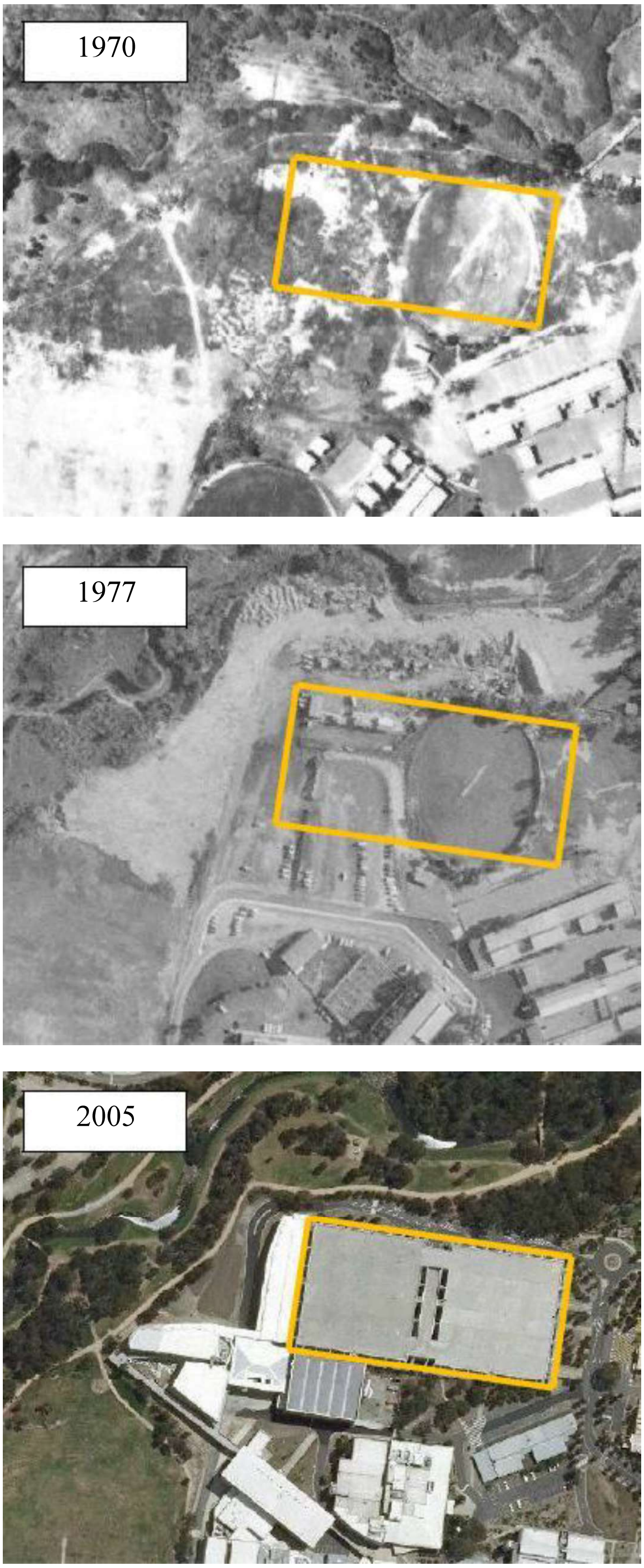

Figure 3. Aerial photographs dated 1970, 1977 and 2005 (Victorian State Government), with the approximate footprint of the CP6 building highlighted.

The presence of waste materials and deep uncontrolled fill, together with visual assessment of surface conditions (refer Section 3.4), meant that the potential for downdrag (negative skin friction) loads needed to be considered for the piles. It was not apparent if downdrag had been considered in the original pile design. However, the presence of typically highly to moderately weathered rock recovered by NMLC coring (the original investigation had been drilled using auger techniques only) meant that the original pile design parameters adopted were likely to have been conservative for these materials.

3.4 *Surface conditions*

The visual assessment of surface conditions contributed two main pieces of information to assist in developing design recommendations, as follows:

- Weathered siltstone materials were exposed in cut batter slopes to the east and south of CP6 East.
- Evidence of differential settlement was observed between the asphalt pavement surface and piles within CP6 West (Figure 4).

The observation of differential settlement within CP6 West provided further evidence of the nature of the uncontrolled fill materials and the potential for downdrag loads to develop. For the column shown in Figure 4, the difference in surface level across the cracks was about 20 mm to 40 mm, which was inferred to have developed in less than 17 years. The shape of the crack pattern (the circular shape was about 1.6 m to 1.7 m in diameter and centred on the column) also provided confirmatory evidence that the pile diameter was consistent with the construction documents.

Figure 4. Differential settlement between piled footings (CP6 West) and the surrounding asphalt pavement.

4 FINDINGS OF THE ASSESSMENT

The results of the geotechnical assessment indicated that the depth to weathered siltstone varied across the site, with a general trend of increasing depth to rock from south to north and east to west. This trend was consistent with the different footing types adopted for CP6 East and CP6 West and was later confirmed based on a review of the original geotechnical investigation reports for the building. Within CP6 West, the presence of deep uncontrolled fill had implications for pile design.

Based on the results of the assessment, it was considered likely that the existing footings had sufficient redundant capacity to accommodate the pro-posed increase in the applied loads, taking into con-sideration that the design parameters adopted generally appeared conservative for the weathered siltstone materials encountered and additional settlement rather than bearing capacity would be the key geotechnical consideration. However, it was recognised that only a small proportion of existing footing locations had been investigated, and there was the potential for variability in the original construction methods (e.g. cleanliness of pad footing or pile excavations) and ground conditions, including the potential for some pad footings to found on residual siltstone soils (typically stiff to hard clay) rather than weathered rock, particularly within the north and west parts of CP6 East. Hence, a program of construction monitoring was recommended so that the behaviour of the footings when loads were applied could be monitored and responded to, if necessary.

It was expected that an increase in the applied load by about 25% would increase settlement by a similar amount; i.e. assuming footings had originally designed for a maximum settlement of about 10 mm to 15 mm, the additional settlement should be less than 5 mm, which was considered acceptable by 4D Workshop. A trigger level of a maximum of 5 mm of additional settlement was recommended as requiring construction works to cease until further geotechnical and structural assessment could be under-taken, on the basis that if this settlement was observed it would indicate that founding conditions or loads differed from the assumptions, while there was still scope for remedial actions to be performed before the existing building was damaged.

Recommendations were also made regarding the staging of construction of the new car park level, so that for CP6 East the loads would progressively be applied from southeast to northwest, to allow the initial monitoring of footing performance to be made in the area where founding conditions were expected to be more favourable (weathered rock rather than clay).

5 CONSTRUCTION STAGE MONITORING

Based on the ground model, it was expected that footing settlement in CP6 East would increase from south to north and east to west in line with the expected increase in the depth to highly or less weathered siltstone. In CP6 West, it was expected that similarly loaded piles would have similar socket lengths in weathered rock hence settlement under the new loads would be less variable than in CP6 East.

Construction phase monitoring was performed by Madigan Surveying on behalf of Kane Constructions, based on advice from 4D Workshop and Golder. Monitoring points comprised steel dome headed bolts installed into the exposed concrete surrounds at column bases. Level survey was performed relative to a control point located outside the CP6 building prior to the construction works, and on four more occasions over a period of about three months during and after construction. Table 1 summarises the maximum settlement measured at each of the monitoring points (compared to the initial reading) over the monitoring period. Survey measurements were reported to the nearest 0.1 mm. How-ever, the reported precision may overstate the level of survey accuracy presented in Table 1.

Table 1. Measured settlement during construction.

Location (footing type)	Monitoring Point	Settlement (mm)
CP6 West (piles)*	MP1	0.6
	MP2	0.8
	MP3	0.7
	MP4	0.6
	MP15	0.3
	MP16	0.5
	MP17	1.0
	CP106	0.7
CP6 East (pads)**	MP5	1.4
	MP6	1.6
	MP7	1.8
	MP8	1.1
	MP9	1.2
	MP10	1.0
	MP11	1.0
	MP12	0.9
	MP13	0.9
	MP14	1.0
	MP18	1.4
	MP19	0.7
	CP103	0.8
	CP105	1.0

* All CP6 West monitoring points were installed at columns supported by nominal 1.5 m diameter piles.
** Monitoring points MP5, MP10 and CP105 were installed at columns supported by nominal 2.6 m square pad footings. MP7, MP12 and MP14 were installed at columns supported by nominal 1.8 m square pad footings. All other CP6 East monitoring points were installed at columns supported by nominal 2.9 m square pad footings.

For the piled CP6 West, the maximum settlement measured was about 1.0 mm, with the measured settlement typically between 0.5 mm and 1.0 mm.

The measured settlement for CP6 East (pad footings) was generally higher than for CP6 West, with a maximum measured settlement of 1.8 mm and typical measurements in the range of 1.0 mm to 1.5 mm. The highest measured settlements were in the northwest and west parts of CP6 East, which was generally consistent with expectations based on the ground model, taking into consideration the relatively small magnitude of the settlements measured. The measured settlement was below the nominated 5 mm trigger level for further assessment.

Note that assuming an increase in the applied pressure of about 67 kPa beneath the 2.9 m square pad footings, the survey measurements are generally consistent with a foundation stiffness of about 40 kPa/mm to 95 kPa/mm, for founding materials that may range from residual clay to highly weathered siltstone.

6 CONCLUSIONS

Geotechnical assessment of existing buildings to assess redundant footing capacity and the potential to increase applied loads should consider a variety of information sources and techniques to manage the risk of unsatisfactory footing performance. Information sources may include, but not be limited to:

- construction drawings and historical geotechnical reports;
- construction records (e.g. reports of footing inspections) and photographs;
- historical aerial photographs;
- site observations;
- intrusive investigations; and
- construction monitoring.

Key observations from the case study that are more widely applicable to building modification/extension projects are summarised as follows:

- The availability of construction records and design documents has a significant impact on the assessment of redundant capacity and the level of confidence in the original construction methodology, as well as the scope and cost of additional assessment required to address knowledge gaps or uncertainty in these records. Hence, good record keeping (e.g. readily accessible electronic databases for current or future projects) should be a priority for all stakeholders including developers, building owners, contractors and consultants. The data should be considered as an asset with future value.
- Practical limitations on intrusive investigations for building modification/extension projects include access for drill rigs or other investigation equipment, and the potential for disruption to existing building operations. These limitations require careful planning and mean that investigation works for these types of projects may be costlier and take longer than conventional geotechnical investigations.
- It is unlikely to be practical to conduct investigation or other assessment works that eliminate un-certainty in footing conditions. Hence, construction monitoring is likely to be required for building modification/extension projects, with protocols for responding to monitoring results put in place prior to the commencement of construction works.

The results of construction monitoring performed during the CP6 extension works provided verification of the geotechnical assessment and risk management approach undertaken for this project.

7 ACKNOWLEDGEMENTS

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