AGS VICTORIA 2016 SYMPOSIUM

Excavations and slope stability in Melbourne geology: experiences and recent developments

Wednesday, 16 November 2016, 12:00pm – 7:00pm
Engineers Australia, 600 Bourke Street, Melbourne

AUSTRALIAN GEOMECHANICS SOCIETY
VICTORIA CHAPTER

WELCOME

The Victorian chapter of the Australian Geomechanics Society (AGS) is pleased to welcome you to this half-day symposium titled “Excavations and slope stability in Melbourne geology: experiences and recent developments”.

Since the publication of the “Engineering Geology of Melbourne” in 1992, both the geotechnical profession and Melbourne has undergone significant change. Urban sprawl over the past few decades has seen increasing development in the hillside areas in the Dandenong and Mornington Peninsula regions. This coupled with changes to the regulatory environment and the introduction of the Landslide Risk Management Framework by the AGS in 2007 has changed the way in which local and state government as well as geotechnical practioners manage and assess slope stability.

In addition to development in hillside areas, significant development in the inner parts of Melbourne has posed many challenges for excavations not just in the soft soils of the Yarra Delta but also the weak rock of the Melbourne Formation.

This symposium seeks to bring together practioners from consulting, construction and academia to share and discuss their experiences on the separate, but related, topics of excavation and slope stability. Best practices, case histories and innovative solutions for dealing with these challenges will be presented and discussed, with a particular emphasis on local geotechnical issues.

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RECENT ADVANCES IN THE GEOTECHNICAL DESIGN OF DEEP BASEMENTS IN MELBOURNE

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ABSTRACT

This paper presents a summary of the authors' recent experience and involvement in the design and performance monitoring of several deep basements constructed in Melbourne. Advanced methods of design and analysis of basement retention which provide a better understanding and allow refinements to the more traditional methods of basement retaining wall design are outlined. The results of monitoring of some basement support walls are presented. Recommendations for geotechnical investigation and design of deep basements are presented and some problems that have been encountered in recently constructed basements are outlined.

Keywords: Melbourne geology, basement design, deep excavations, soil structure interaction, ground support, retaining wall analysis.

1 INTRODUCTION

The design of basements in Melbourne has traditionally been based on the information presented in the seminar of Engineering Geology of Melbourne (1992) commonly known as the "blue book". In particular, the papers in the proceedings by Morgan (1992) and Macleod (1992) are often referred to for basement design.

Since 1992, the analytical tools available for basement design and our knowledge of the performance of deep basements constructed in Melbourne have increased significantly.

This paper presents recent Victorian experience and performance monitoring from several deep basements in which the authors' company has been directly involved.

2 TRADITIONAL DESIGN APPROACH

Based on the Morgan (1992) paper presented in the blue book, temporarily anchored drained basement retaining walls have typically been designed to resist lateral ground pressures in the range of 4 to 8 times H in kPa, where H is the total height of the excavation in metres. Where movement of adjacent structures induced by the proposed basement excavation is of concern (e.g. adjacent near surface supported buildings), then 6 to 8 times H is often recommended to provide a stiff wall likely to limit horizontal deflections of the wall and associated settlement outside of the wall to acceptable values.

The above design pressures exclude ground water pressures given that the majority of basements in Melbourne tend to be constructed, where possible, as drained basements.

Drained basements are generally constructed below the standing ground water level using temporary dewatering during construction. A permanent drainage blanket is installed below the lowest basement floor slab to prevent the potential build-up of water pressure beneath the floor which can lead to hydrostatic uplift. Where practical, strip drains are installed behind the basement walls such that additional loading of the basement walls due to ground water pressure does not occur.

This type of solution works well as long as the volume of ground water flowing into the strip drains and sub-floor drainage blanket is manageable with respect to off-site disposal.

In cases where the rate of groundwater inflow into the subfloor drainage system is too high due to horizontal groundwater flows through more permeable granular soils a continuous section retaining wall (e.g. a diaphragm or a secant pile wall) is extended a sufficient depth below the excavation level or down into less permeable material to achieve lateral cut off of groundwater flows. In addition groundwater flow into the drainage layer should not lower the ground water table outside the basement such that excessive settlements occur. This is an important consideration in Southbank and South Melbourne where significant deposits of compressible Coode Island Silt are present.

In the event that a drained basement cannot be practically constructed, a more expensive essentially waterproof or tanked basement will be required.

In this case the basement floor must be held down to resist uplift water pressure which normally requires a thicker basement floor slab and, in some cases, tension anchors between structural columns.

3 PROBLEMS WITH THE TRADITIONAL DESIGN APPROACH

The main problem encountered using the traditional approach in deep basements constructed in Melbourne with soil overlying rock, is that at depth, the design wall pressures become very large based on the 4 to 8 times the depth of excavation (H) wall horizontal pressure distribution.

Piling contractors routinely carry out the design of basement retaining walls using the commercially available software package WALLAP provided by Geosolve (2016).

This software offers a modulus of subgrade or a quasi-finite element analysis of a two dimensional basement retaining wall problem. The program provides calculated wall design actions (bending moment and shear force per metre run of wall), anchor forces and wall lateral deflections. Water pressures can be varied on either side of the wall and adjusted at the toe of the wall. Various excavation and construction stages can be included including surcharge loads, construction of berms, anchor and strut installation and removal and changes in soil parameters (for example changing from short term undrained to long term drained soil strength parameters). One significant drawback with this method of analysis is that it only provides a lateral wall deflection profile with no information on horizontal and vertical ground deflection profiles behind the wall.

A more refined approach is to use the currently available geotechnical finite element programs such as PLAXIS 2D and 3D, PLAXIS (2016) to assess wall actions and deflections and ground movements around the proposed basement. Various soil models including Mohr Coulomb, Hardening soil and soft soil creep can also be included in these analyses. These programs can also calculate steady state seepage ground water profiles and pore water pressures behind proposed retaining walls.

In rock excavations, joints and slip planes can also be modelled in PLAXIS or similar software (e.g. FLAC) to assess the sensitivity of the design and anchor loads to movements along joints that may be induced in rock during basement excavation.

The more advanced analytical methods can generate insitu horizontal stresses that can more accurately model existing stresses in the ground prior to excavation which is an important consideration.

4 EFFECT OF LOCAL GEOLOGY

Based on recently monitored basement excavations and more recent ground investigations which have included down-hole televiewer defect mapping, it is apparent that in some areas of Melbourne there is an observable trend of defects dipping into and or out of the east and west faces of basement excavations rather than into or out of the north and south faces.

Figure 1 presents an extract from the Ringwood 1:63360 geological map sheet. The fault axes shown in Figure 1 as thin solid black lines all trend about 10 to 30 degrees east of north as shown by the thicker arrow ended line. Significant bedding planes and joints generally trend east or west. Hence the east and west faces of basement excavations are more likely to encounter unfavourable slip planes and joints potentially dipping into rock excavations than the north and south faces.

Figure 1 an extract from the Geological Survey of Victoria Ringwood Geological map sheet

The potential presence of slip planes and consideration of the local geology can be included in PLAXIS models by modelling joint planes in the rock and increasing the coefficient of the at rest earth pressure (Ko) in soils from between 0.7 to 1.0 to better model the effects of regional geology.

The authors' experiences on some recent projects which have used the more refined approach outlined above are presented below.

5 RECENT BASEMENT EXPERIENCES

5.1 Failed basement cut off wall

A two stage site investigation was carried out by Golder for a two to three level basement for a multi-level development close to the waterfront in a large regional city in Victoria.

The site investigation encountered the ground conditions summarised in Table 1.

Table 1: Ground conditions encountered

Material	Depth Range (m)			
	BH1	BH2	BH3	BH4
Fill	0-1.5	0-1.2	0-1.5	0-1.1
Loose sand	to 4.0	to 4.0	to 2.5	to 3.8
Soft silty clay	to 6.4	to 6.4	to 4.0	to 5.4
Clayey sand	to 9.0	to 9.4	-	to 7.2
Stiff clay	to 20	to 16	to 19	to 40

In addition to boreholes a series of cone penetrometer tests (CPTs) were carried out across the site at locations shown in Figure 2. The CPT's confirmed the depths to stiff clay which are shown in red next to the boreholes in Figure 2.

The basement retention design was based on a drained basement with a hard-soft secant piled wall, temporarily anchored until the basement floors were cast. The perimeter basement wall was required to penetrate at least 2 m to 3 m into the underlying stiff clay to limit groundwater inflow into the basement in the short and long term. The estimated amount of inflow was assessed not to cause a significant drawdown of the ground water table around the site under adjacent buildings so a drained basement was considered to be acceptable.

Construction commenced with the installation of a hard/soft or hard/not so hard basement perimeter secant pile wall constructed using CFA piling equipment to a piling contractor's design and construct alternative design.

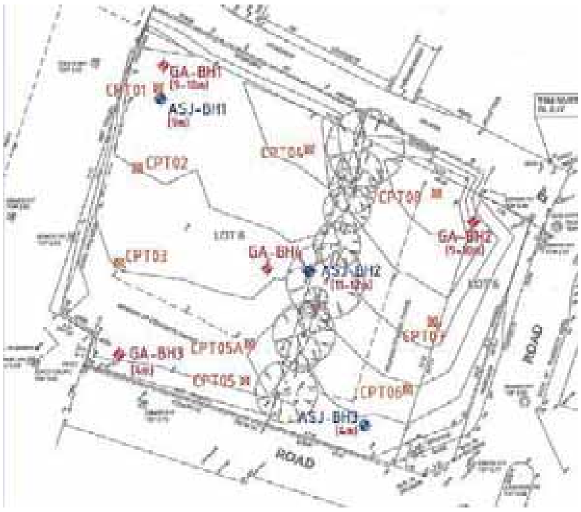

Figure 2. Site investigation layout – red figures indicate depth to top of stiff clay

Once the walls were installed, a perimeter capping beam was cast over the top of the CFA piles and bulk excavation to about 2.5 m was commenced to allow the first row of ground anchors to be installed.

After the first row of ground anchors were installed, excavation continued to the second level of anchors. Some problems were encountered in achieving the design anchor load along the left hand side boundary (boreholes GA-BH1 and GA-BH3) shown in Figure 2, when the soft to very soft, silt clay layer was found to increase in thickness outside of the site. The temporary anchor drilling technique was changed from cable anchors with air flush to augers and finally to drilled and grouted hollow Dywidag stress bar anchors to achieve the required retention force for the left hand side basement wall.

When excavation proceeded below the second level of anchors on the left hand side wall, increased water flow into the excavation was encountered and the soft silty clay and clayey sand in the vicinity of borehole GA-BH1 was observed to lose strength and to have an upward flow of water to the excavated surface.

Additional boreholes drilled outside of the site adjacent to borehole GA-BH1 indicated that the top of the stiff clay layer into which the CFA piles were to be embedded 2 m to 3 m dipped down outside of the site and it was likely that the CFA pile wall had not been constructed to achieve the required level of penetration into the stiff clay.

This was concluded to be the cause of the upwards flow of water into the excavation and the loss of strength of the soft silty clay and clayey sand exposed in the base of the excavation, as shown in Figure 3 below.

Figure 3. Water flow into basement excavation

Fortuitously, a basement ramp was to be constructed from basement level 3 (the deepest level) to Level 2 along the basement wall adjacent to the left hand side of the basement shown in Figure 2.

To allow construction to continue, and to provide a long term reduction in inflow into to achieve the design level of inflow, a sheet pile retaining wall was designed and installed under the ramp adjacent to the wall where the CFA piles had not encountered the stiff clay.

The sheet piles were installed inside the basement wall and incorporated into the ramp floor slab which was keyed into the basement wall to resist uplift pressure on the basement wall side of the sheet piles.

To allow the ramp slab to be constructed, relief wells were installed between the sheet piles and the perimeter basement wall to control seepage water pressures until the ramp slab was cast and able to resist uplift pressures. At this time the wells were capped and sealed. This allowed basement construction to continue to completion.

5.2 A moving deep basement east wall

In the north of the Melbourne CBD, Golder was involved in the investigation and basement retaining wall design for a 5 level (15 m deep) basement beneath a 40 level apartment block. The site was bound by lane ways and Little Lonsdale on three sides and a two storey warehouse apartment on the fourth east side.

Ground conditions at the site comprised 1.5 m of fill over 6 m to 8 m of residual clay overlying Silurian age siltstone and sandstone. Insitu pressuremeter tests were carried out in the rock as well as UCS tests on recovered core. The ground water level was present at a depth of about 11 m. The calculated rate of water inflow was about 1litre/sec and a drained basement was adopted.

The proposed basement retention system comprised a 300 mm thick diaphragm wall installed to grab refusal on the siltstone supported by three levels of temporary post tensioned (P/T) anchors. Below the bottom of the diaphragm wall, excavation was continued in rock in 1.5 m drops with the exposed rock supported by passive rock bolts installed on a 1.5 m square grid and shotcrete facing.

The basement retention was designed using 2D PLAXIS analysis of four representative cross sections through the basement walls. Figure 4 presents a PLAXIS screen capture of the east wall cross section showing the surcharge load from the immediately adjacent apartment building as blue arrows and the anchors and passive rock bolts as yellow lines.

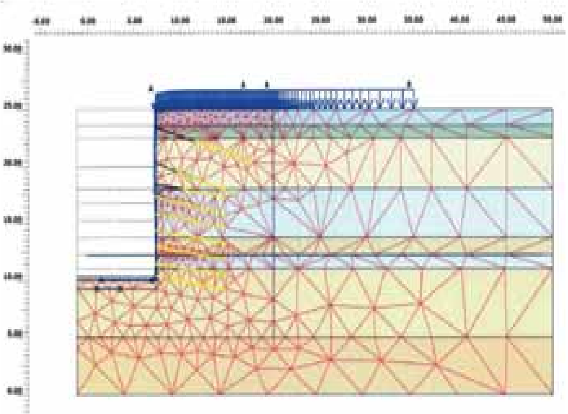

Figure 4. East wall PLAXIS mesh section

The PLAXIS analyses modelled the construction stages comprising:

- Establish initial ground conditions and apply building and/or traffic surcharge loads and zero deflections
- Install diaphragm wall (DW)
- Excavate to 2 m and install P/T anchors
- Excavate to 5 m and install P/T anchors
- Excavate in hit 1, miss 2 drives to 7 m and install DW passive toe anchor bolts
- Excavate rock in 1.5 m drops and install passive rock bolts on a 1.5 m square grid with a shotcrete facing.

The results of the PLAXIS analyses using the above staging indicated lateral deflections at the top of the DW wall of less than 10 mm. Vertical settlement behind the wall was calculated to be less than 5 mm.

Construction commenced and proceeded satisfactorily until the depth of excavation reached about 9m at which point deflection of the east wall was noted to have reached about 23 mm, exceeding the less than 10 mm estimated from the PLAXIS analysis.

Survey measurements indicated that the east wall of the excavation had moved relatively uniformly into the excavation without tilting from the top down. The other three boundary walls had moved less than the PLAXIS model estimates.

Melbourne City Council issued an order to protect the excavation in view of the observed movements. A series of steel tube struts were designed and installed to prop between the east and west walls as shown in Figure 5.

Figure 5. Steel tube struts propping east wall

PLAXIS calculations were undertaken to try and ascertain the cause of the additional movements observed in the east wall. The results of these analyses indicated that underestimation of the adjoining building surcharge load was unlikely to have caused the observed wall deflection.

The observed wall movements could however be replicated by modelling the east wall anchors as passive rather than prestressed. This could represent poor anchor installation including full grouting of the anchors to the back of the DW or a loss of anchor prestress.

Unfortunately no detailed anchor records were available and the anchor installation was not observed by Golder.

The observed wall movements stabilised at about the same time as the struts were installed. This is consistent with the wall moving and the anchors taking up load. Basement construction continued after strut installation to full depth and no further problems were encountered.

Lessons learnt from this project include the value of detailed monitoring of basement walls as excavation proceeds (we were not provided with wall movement records until significant deflections had occurred) and the benefit of observing installation and testing including proof loading of temporary anchors.

5.3 A moving deep basement wall

A new development was constructed in the north east of the CBD. The development included the construction of a three level drained basement supported by bored soldier piles, temporary P/T anchors and passive rock bolts and shotcrete infill panels adjacent to a heritage listed building.

Golder carried out a desktop study followed by an intrusive site investigation which included insitu pressuremeter testing and UCS testing of rock core.

The site investigation indicated the stratigraphy and material properties summarised in Table 2.

Table 2: Site ground conditions

Strata	Depth (m)	c' (kPa)	Φ' °	E' (MPa)
Fill	1	1	30	20
Residual/EW	2.5	10	28	30
HW Silurian	5	80	40	200
MW Silurian	>5	90	44	300

The basement retaining walls were designed as 600 mm diameter soldier piles at 1.8 m (3D) or 1.2 m centres drilled to below basement excavation level.

Temporary support for the upper fill and residual soil was provided by temporary P/T anchors and by 3 m long passive rock bolts installed in 125 mm diameter holes in rock.
Calculated lateral deflections during basement excavation varied between 5 mm and 10 mm for the three wall sections analysed. Figure 6 presents a screen capture of the PLAXIS model of the southern and deepest basement excavation section which was carried out immediately adjacent to the heritage building.

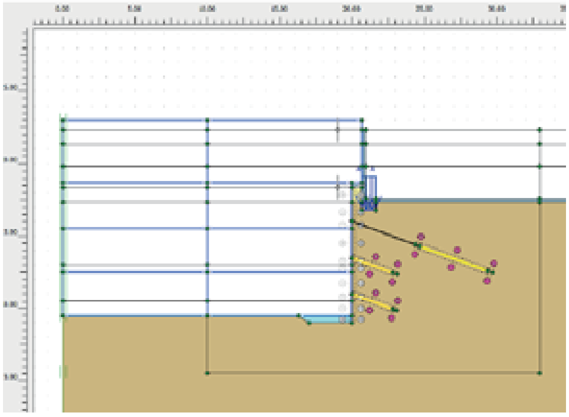

Figure 6. Southern wall PLAXIS model

As excavation was carried out the basement wall deflections were monitored by survey targets on the capping beam and by inclinometer survey of casings cast into bored piles on the south (adjacent to the heritage building) and the western boundaries.

Survey data was provided which showed wall deflection on all of the walls apart from the southern wall were within design expectations (less than 10 mm).

Measured deflections on the southern wall were observed to increase above the 5 mm to 10 mm design values towards the end of basement excavation.

The measured inclinometer lateral deflections from a retention pile in the western wall and from a pile in the southern wall are presented in Figures 7 and 8 respectively. Referring to the deflected shape of the wall shown in Figures 7 and 8 it appears that the upper post tensioned anchors in Figure 8 (south wall) were not engaged.

This was confirmed by further PLAXIS analysis from which we inferred that the southern wall movements were due to stress relief, and a combination of potentially softer anchor performance or higher lateral stresses and poorer ground outside the southern boundary of the site. Unfortunately Golder was not able to verify anchor installation and testing of the initial anchors installed on site, which may have provided more information as to the cause of the larger deflections.

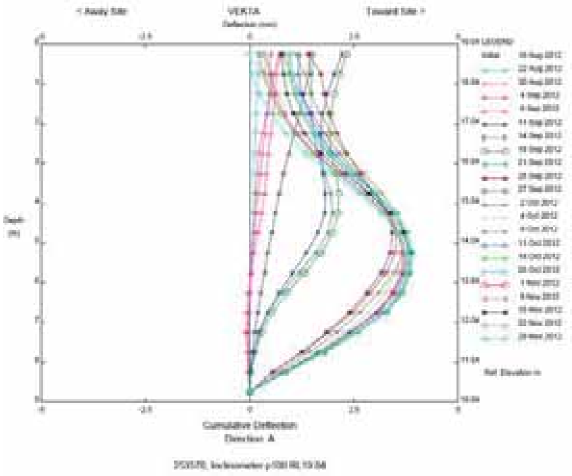

Figure 7. Western wall inclinometer data

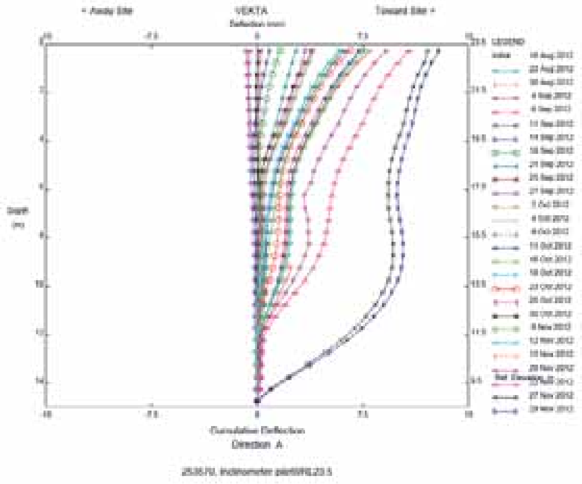

Figure 8. Southern wall inclinometer data

6 CONCLUSIONS

The results of several recent deep basement excavations carried out in and around Melbourne have been presented. The results demonstrate the need for monitoring during excavation and observation of anchor installation and testing.

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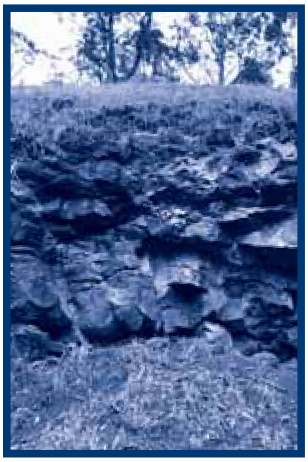

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