Retention for Major Excavations

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ABSTRACT

Deep excavations present a number of interrelated challenges. Those that apply to Auckland's geology and the regulating environment include the risk of block slides, the effects of temporary works on nearby developed land and the consequences of groundwater drawdown on the surrounding buildings. Examples of recently completed developments in the Auckland CBD are the 22 m deep basement for the Sky City hotel and casino, the water tight basement in recent marine sediments and reclamation fills for the Britomart Transport Centre and the 5-level basement for the Auckland University Business School on sloping land affected by historical block slides on clay seams. Design processes, predicted displacements and drawdown effects are described. Results of monitoring are summarised and lessons learned are presented.

1. INTRODUCTION

The construction of deep basements presents a range of challenges that arise mainly from major excavations in a built-up area often surrounded by existing buildings and underground services. The challenge presented to designers is to find an economic solution to two main issues, these being deflections of the retention system as excavation proceeds and the effects of groundwater drawdown.

Addressing this challenge requires consideration of:

(a) Geology: the conditions to be addressed may include weak/compressible soils (e.g. recent sediments), unfavourable jointing, weak bedding planes, expansive soils, vibration resonance, strong rocks;
(b) Hydrogeology: groundwater table(s) - unconfined phreatic surface or multiple water tables, artesian pressure(s), high permeability, risk of high inflows from leaking services;
(c) Drawdown: amount, duration and effects of drawdown on the surrounding land and nearby buildings (settlements to ground, buried services and buildings);
(d) Deflections: wall vertical and horizontal deflections on excavation need to be evaluated for each stage of construction, and the effects on buried services and buildings considered;
(e) Design options: "top-down" or "bottom-up", tanked or drained basement, palisade wall, secant piles or diaphragm walls, propped or anchor support, recharge trenches etc;
(f) Monitoring and contingency planning: an essential part of design is to monitor the effects to compare theses against design assumptions and predictions, and to allow mitigating/remedial action to take place, in an adequate time frame, to avoid damage or harm;
(g) Conditions of consent: these may be onerous or restrictive and need to be considered, accommodated and worked through.

Examples are presented below to illustrate the above issues and how the challenges were met.

2. SKY CITY DEVELOPMENT

2.1 Scope

Sky City included a 328 m communication and entertainment tower, two hotels, a casino and five levels of basement with an excavation depth to 20 m - 22 m. This development is surrounded by existing streets, underground services and buildings including the ASB Bank and St Matthews historical masonry church. The site covers a city block, some 65 m by 190 m.

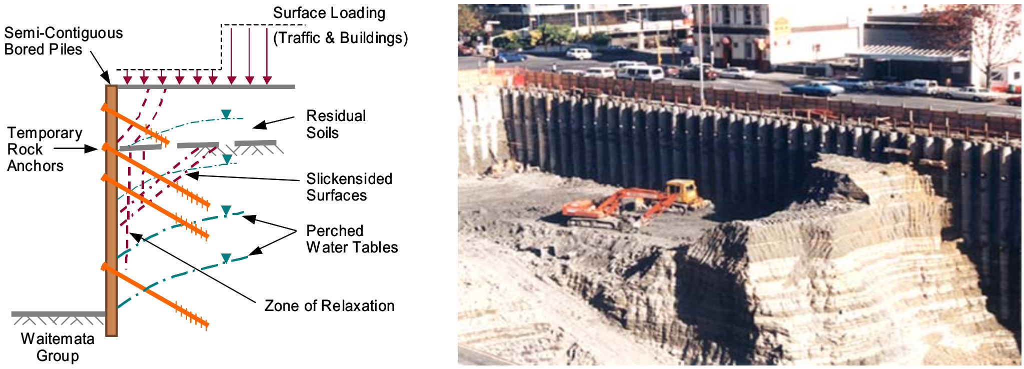


Figure 1: Sky City Ground Retention

2.2 Site Conditions

Table 1 below shows a simplified geological profile for the Sky City site. The main issues were the dipping bedding planes in the Waitemata Group (WG) soft Tertiary rock, the drawdown effects on the historic masonry church and the existence of a 15 m – 17 m deep in-filled gully to the east.

Table 1: Simplified Geology – Geotechnical Parameters – Sky City

Soil Profile	Description	Depth to top (m)	C' (kPa)	ø' (°)	Cu	E (MPa)	Calculated Deflections (mm)	Measured Deflections (mm)
Fill	Variable clayey soils with building rubble	0.0	0	28	50-100	30	10-20	10-30
Residual Soil	Interbedded clayey SILT and silty SAND	0.4-1.2	0	30	80-120	50	15-30	-
HW-MW WG	Extremely weak sandstone/siltstone	3.0-16.0	2	32	150	100	15-30	-
SW-UW WG	Very weak sandstone/siltstone	4.0-17.0	20-50	34	200-500	200-500	-	-

Note: Palisade bored pile = 800mm dia. at 1.35 m c/c; extending 4.0m below excavation into WG rocks.

2.3 Design Choice

A bottom-up construction was adopted with a semi-contiguous, anchored, 800 mm bored pile palisade wall, with piles at a spacing of 1600 mm (and some at 900 mm diameter at 2000 mm). The hydrogeology consisted of a series of cascading perched water tables each about 3 – 5 m deep with the regional water table just intersecting the base of the basement. With this groundwater condition a drained basement could be adopted, significantly reducing the cost of tanking and permanent retaining.

Commercial retaining wall software packages FREW and FLAC were used to analyse the wall performance. Allowances were made in the anchor system design for bedding planes dipping into the excavation over half the excavation. The geological model was updated as the excavation proceeded. Based on the detailed logging of the bedding plane angles it was possible to save a large percentage of the anchor costs. Only a small proportion of the bedding planes were actually dipping into the excavation thus reducing the extent of wall requiring the full sequence of anchors. Elsewhere only a set of nominal anchors were installed to control the degree of relaxation expected in the soft rock.

2.4 Monitoring

Monitoring included: ground settlements, precise building settlements, precise wall lateral deflections and regular logging of groundwater drawdown using multi-level piezometers. The uppermost perched groundwater level drawdown was found to be within the range of normal reductions associated with summer seasonal changes and the drawdown in the lower perched water tables in the tertiary rock had little or no significant effects in terms of measured ground settlements (i.e. ≤ 10 mm). Lateral deflections along the top of the palisade retaining wall were within the predicted range of movements (≤ 25 mm). Cracking in the pavement areas close to the retaining walls was within the extent predicted. The services and pavement surfacing were upgraded at the end of construction.

3. BRITOMART

3.1 Scope

Britomart is an underground train station some 16 m deep surrounded by historic buildings in the lower downtown area, covering two city blocks over an area of some 60 m by 225 m. The proposed Britomart development was plagued by objections and concerns from submitters. Work was to be carried out under close scrutiny with detailed monitoring of both movements and groundwater drawdown effects.

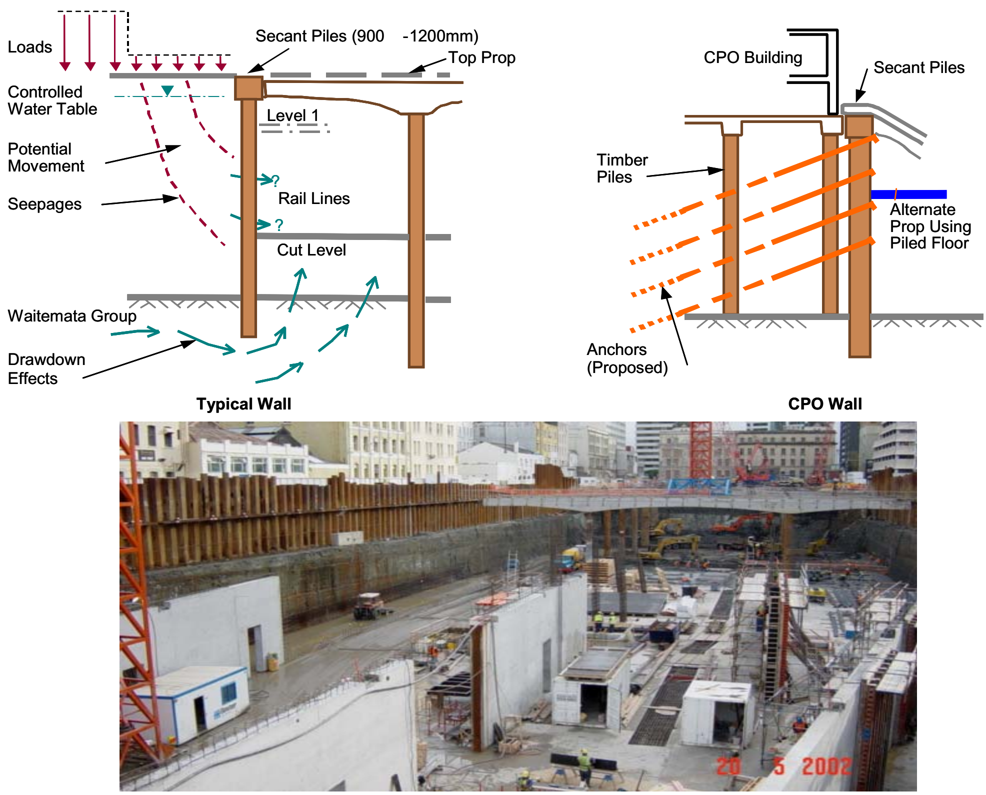


Figure. 2 Britomart Excavation

3.2 Site Conditions

Much of the station construction was to be in recent marine sediments to the west with a cover of historic hydraulic fill and building debris. The eastern end of the building was on WG soft rock.

Table 2: Simplified Geology & Geotechnical Parameters – Britomart Transport Centre

Soil Profile	Description	Depth to top (m)	C' (kPa)	ø' (°)	Cu	E (MPa)	Calculated Deflections (mm)	Measured Deflections (mm)
Fill	Variable clayey soils with rubble	3.0–4.0	2	28	20-100	20	10-20	≤20
Tauranga Group	Interbedded clayey SILT and silty SAND	3.5-5.5	0-8	26-28	30-150	5-100	10-30	-
W WG	Extremely weak sandstone /siltstone	3.0-14.5	10	32	150+	50	-	-
UW WG	Very weak sandstone/ siltstone	4.0-25.0	100	34	500	200-500	-	-

Notes: Secant pile = 900 – 1200 dia., overlap = 300 mm; 4.0 m into WG rocks.

Tanking the station was the only option due to concerns about drawdown effects, potential saline water intrusion and induced settlements on the surrounding historic buildings, some of which are 4 to 5 storey masonry buildings on shallow foundations. The water table is high and tidal.

3.3 Design Choices

A full 3-D groundwater model was created to evaluate the effects of various stages during construction. This was calibrated to groundwater level measurements and pumping tests. A secant bored pile retaining wall, taken down some 4 m into the bedrock (or 4 m below excavation level where WG rocks are exposed), was adopted to construct the watertight underground station. Bedrock exists at depths ranging from 3 m to 25 m below street level relative to the 16 m depth of excavation. Ground anchors and propping slabs were used to control lateral deflections. The secant piles were mostly 900 mm dia. embedded 4 m into rock, with a small number at 1200 mm diameter right next to the historic Post Office building; every second pile was reinforced.

3.4 Monitoring

Groundwater response to pile construction showed drawdown in the jointed WG rock to be rapid and widespread. A recharge trench assisted in controlling the groundwater levels in the upper more compressible marine and alluvial soils. Some minor leakages through the joints between the secant piles persisted for some time, and needed repeated grouting and sealing to achieve cut off.

Settlements were small, all within the 10 mm or less across the street, as required by consent conditions. No significant settlement effects from drawdown were measured on the surrounding buildings. Wall deflections were within the predicted 25 mm - 30 mm. The groundwater levels returned approximately to the pre-construction levels except for the now constant level produced by the recharge trench, which also acted as a drain.

4. BUSINESS SCHOOL

4.1 Scope

The Business School is a major development for the University of Auckland and covers a plan area about 150 m by 150 m on a broad sloping ridge of weathered WG soft rock. This development includes a 5 level basement surrounded by existing university buildings and roads.

4.2 Site Conditions

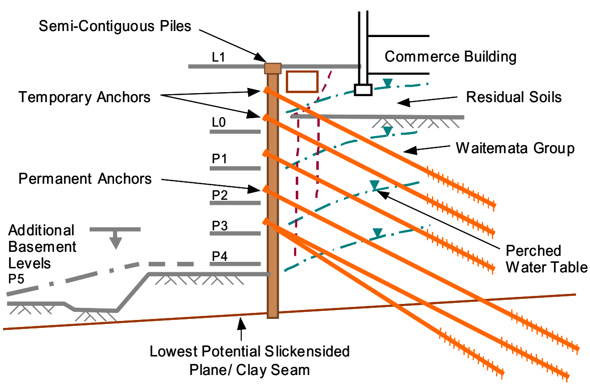
The site is located mostly on a broad ridge terminating on a steep bank above a deep gully that has been filled and used as an arterial route to the port.

Table 3: Simplified Geology & Geotechnical Parameters – UOA Business School

Soil Profile	Description	Depth to top (m)	C' (kPa)	Ø' (˚)	Cu (kPa)	E (MPa)	Calculated Deflections (mm)	Measured Deflections (mm)
Fill	silty CLAY	0	0	28	50-100	10	10-20	5-10
CW to HW WG	silty CLAY	0.5-4.5	1	29	80-120	12	15-30	5-20
Clay seam	silty CLAY	various	0-1	8-9	20	3	-	-
MW WG	Extremely weak sandstone/ siltstone	1.8-9.4	2	32	150+	50	-	-
SW WG	Very weak sandstone/siltstone	4.5-11.4	60	33	300	200	-	-

Note: Palisade bored piles = 900mm at 1.8 m –2.0 m; 7+ m below base of excavation to create shear sockets.

The main difficulty in terms of geology is the existence of very weak bedding parallel clay seams dipping towards the steep bank below the site. There have been large historical landslides to the two adjacent gullies, a previous landslide into a basement excavation above the site and a recent small slip on a clay seam during the arterial route bridge construction below the site. Groundwater was found to consist of a sequence of cascading perched water tables.



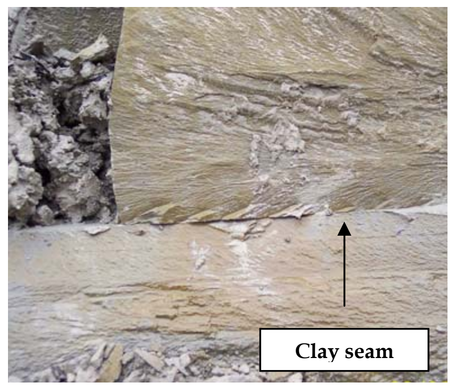


Figure 3: Business School round Retention

4.3 Design Selected

A mixture of temporary, permanent anchors and propping from the carparking floor diaphragms restrained the basement walls. The perimeter retaining walls were semi-contiguous 900 mm dia. bored piles spaced at 1.8 m – 2.0 m, with drains in the space between the piles augmented by drilled inclined drains behind the wall. Water tightness was not required due to the series of perched water tables that existed in the WG siltstone/sandstone sequence. The seismic forces above the lowest clay seam are resisted by a combination of deep pile sockets, permanent anchors and shear resistance from the side palisade walls via floor diaphragm action.

4.4 Monitoring

Monitoring consisted of regular groundwater level measurements that confirmed the cascading nature of the hydrogeology. Lateral and vertical movements were found to be well within the range of predicted (10 mm – 30 mm) wall displacements due to the high level of temporary pre-stress induced in the anchors.

5 CONCLUSIONS

The lessons we have learned from the projects described herein include:

- It is important to understand the geology of the site and address the risks that may be triggered by the excavation;
- The goals of the design are to limit deflections and thereby displacements or any damage to adjacent structures, and to control the effects on the groundwater table(s);
- In order to avoid surprises it is critical to plan and efficiently execute the monitoring work; and
- It is vital to have the authority to address the unforeseeable in a timely manner.

To put this in context, four workers died when a 33 m deep excavation for a new mass transit rail line in Singapore collapsed in 2004 (News 2005a & 2005b). The failure of the propped temporary diaphragm retaining walls was considered by the judge (in the Committee of Inquiry appointed) to be attributed to inappropriate design and warnings from the monitoring not being adequately addressed (Magnus et al, 2005):

- Overestimated soils strengths and inappropriate model;
- Errors in support design and inappropriate monitoring back analyses;
- Excessive wall deflections and surging inclinometer readings;
- Whaler beams buckling and stiffeners buckling;
- Support system overstressed by 50%/Not responding appropriately to violent ominous sounds.

The lack of clarity in the management structure and organisational failings on the Singapore project was deemed to have compounded these issues.

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