Nailed Support for Basement Retention in Jointed Rock

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Summary: For basement construction in weathered mudstone in Melbourne, soldier piles and tensioned anchors are the usual support method. However, when the bedding is favourable, the highly jointed mudstone can often be excavated to form near vertical faces which remain relatively stable. The paper describes the support of a 20m deep basement excavation using both conventional support and also rock nails and shotcrete facing. A jointed rock model was used to confirm rock mass strength parameters adopted for design of the rock nails. Unfavourable bedding and the presence of a massive weak dyke structure prevented general use of the rock nail support system. Nevertheless, sufficient lengths of wall were successfully supported by the rock nails to demonstrate that this alternative means of retention is viable in highly jointed rock, given suitable conditions.

INTRODUCTION

In Melbourne, most deep basements for multi-storey buildings are constructed using a tensioned tieback system in which soldier piles are initially installed, followed by sequential excavation and anchor installation. Once the anchors have been stressed at each level, wall panel infilling with sprayed concrete over steel mesh is completed before continuing excavation to the next level. This retention system generally provides good control over excavation movements, with wall pressure design diagrams being specified by the geotechnical engineer for proportioning anchors, soldier piles and infill panels. The effect of surcharge loading from adjacent structures is usually accounted for with some form of simplified Boussinesq lateral pressure distribution although there is some conjecture as to whether the Boussinesq pressures should be used directly or doubled to limit horizontal movements (Terzaghi, 1954).

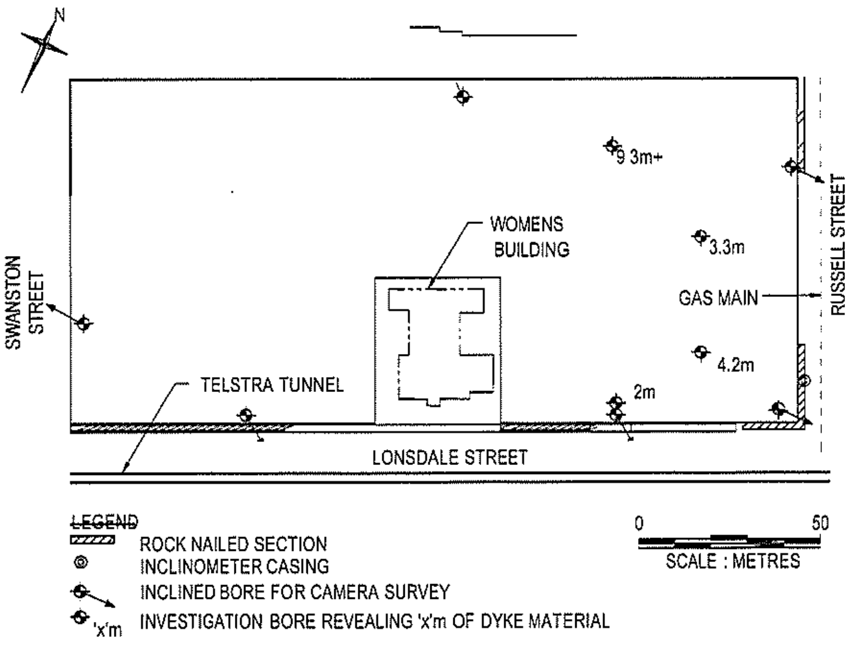

Figure 1. Site Plan

SITE GEOLOGY

The site geology comprises variably weathered siltstone, claystone and minor sandstone of the Melbourne Silurian rocks and are generically referred to as Melbourne Mudstones (MM). These rocks vary widely in strength from very low to high strength and at this site are oxidized to a yellow-brown colour. Iron staining along joints is common, joints may be clean, tight and rough or coated with a thin clay veneer. When groundwater seepage is present, the strength along clay coated joints can become critical if the jointing is unfavourable, releasing planar slabs or three-dimensional rock wedges. Extremely weathered dykes are found within the MM and usually occur in swarms, with individual dykes commonly of the order of a few centimetres thick to two metres thick.

The groundwater at the site is at a depth of about 13m and the rock mass Coefficient of Permeability established by bore recovery tests was around $1x10^{-7}$ m/sec. Basements in the MM are designed as drained and long term groundwater inflow rates are low, typically less than 2l/sec for a site of this size.

ROCK PROPERTIES

Much work has been carried out to establish the strength and deformation properties of the MM (Johnston et al 1985 and Wilson, 1960) and these have been extensively correlated with saturated moisture content. Correlations and site specific test data suggested that the ratio of unconfined compression strength to Point Load Strength Index (Is50) was between 6 and 12, most commonly taken to be 10 in the weathered oxidized MM. Typical moisture content and rock strength profiles are shown in Fig. 2. Significant intervals of weak, pale coloured dyke material were recorded in four bores. The locations of the bores intersecting dyke material are shown in Figure 1.

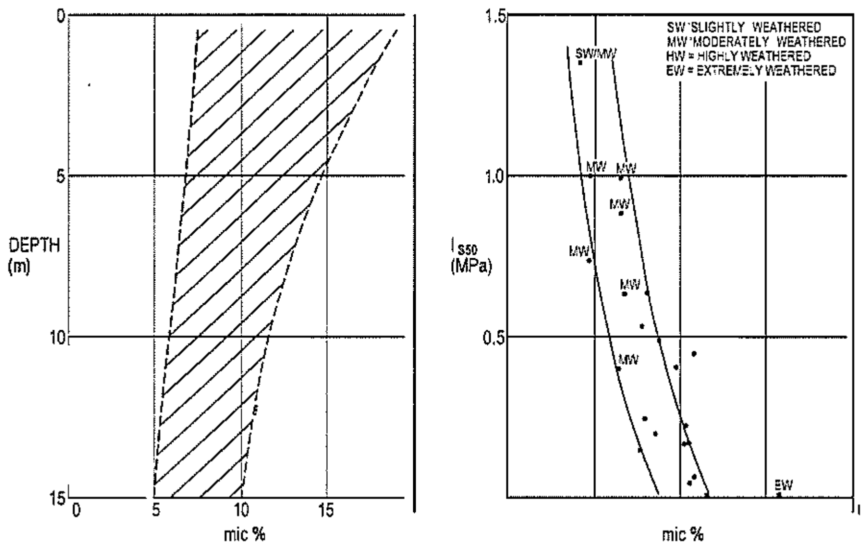

Figure 2. Moisture Content and Point Load Strength Index - Oxidized Mudstone

Joint spacing was found to be in the range 5cm – 50cm. Bedding plane data from a nearby site indicated the potential for unfavourably orientated bedding along the western basement wall.

CONVENTIONAL RETENTION SYSTEM DESIGN PRESSURES

For the Queen Victoria Village (Q W) basements the design pressures suggested by the investigation consultant are shown in Table 1. Similar recommendations have been employed for many other basements in the MM, because of the difficulty of rationally accounting for the mass strength properties of the jointed and variably weathered rock.

Table 1 : Wall Design Pressures

Wall Element	Adjacent Buildings	No Adjacent Buildings	Surcharges
Anchors	6H (kPa)	4H (kPa	K = 0.4 - 0.6
Soldiers	4.5H (kPa)	3H (kPa)	K = 0.3 - 0.45
Infill Panels	4D (kPa) not less than 20 kPa	4D (kPa not less than 20 kPa	Higher value when adjacent buildings

H = Excavation height (m), K = lateral pressure coefficient
D = Maximum anchor-to-anchor spacing vertically or horizontally (m)

These design values imply that the rock mass behaves as if it were a cohesionless material having a friction angle of 37" - 39°, depending on the material density assumed. Increasing the support factors by 50% where buildings are present is considered to restrict inwards movement of the excavation and is sometimes stated to represent "at rest" conditions, although this is not substantiated by rational analysis or field measurements. These design pressures do not apply where bedding is unfavourable, allowing planar failures into the excavation. Non-uniform surcharge loading from adjacent building foundations is additional to the above pressures.

Groundwater pressures were not considered to increase wall pressures. The basement is designed as fully drained and wall and floor drainage is provided to ensure that this assumption is realized in practice.

ALTERNATIVE RETENTION SYSTEM

Douglas Partners had previously designed rock nailed support for relatively low height basement walls and were approached by the excavation sub-contractor to consider a design for this form of alternative basement support for the Q W project. The advantage of rock nails was that excavation and wall support could begin early in the construction program without initial delays associated with the installation of soldier piles around the site boundaries. Vertical perimeter columns or ribs would be progressively cut into the face at 2.4m centres as the excavation deepened. These were required for the permanent wall support because of large floor-to-floor spans of about 8m in many areas. Such spans resulted in significantly larger wall panel pressures than would normally be required for conventional spans of about 2.7m for basements because of reduced arching effects. The rock nail system was only to provide temporary excavation support because any permanent support could not extend beyond the site boundaries.

The feasibility and risks associated with a rock-anchored system for wall support were considered. For the purposes of the sub-contractor's alternative tender, preliminary rock nail designs were prepared using a soil nail analysis program, with rock strength properties based on equivalent cohesion and friction angle parameters that had been employed satisfactorily on designs for lower height walls. A residual friction angle of 23" was adopted, together with a cohesion value equal to one fifth of the peak cohesion for each weathering grade. A summary of strength parameters adopted for the preliminary tender designs is given in Table *2*.

Table *2* : Design Parameters for Queen Victoria Village Basement Walls

Material	Preliminary Strength Values		Density kN/cu.m	E MPa	STEPSIM4 Strength Values			
	c' kPa	Ø' Deg			Joints only		Joints + Bedding Planes	
					kPa	Deg	kPa	Deg
DYKE	5	20	18	40-50				
EW	30	25	18	100				
EW-HW	50	23	20	150	80 +/- 20	29.5 +/- 2	45 +/- 15	29 +/- 2
Hw	100	23	21	250				
HW-MW	150	23	22	400	240 +/- 60	33.5 +/- 2	125 +/- 35	31 +/- 2
MW	200	23	22	600				
Mw-SW				800	430 +/- 90	36.5 +/-2	220 +/- 50	32 +/- 2
SW				1000				

There was concern that the passive rock nail system would not provide sufficient stiffness restraint to the excavation around the historic Women's Centre and the alternative tender allowed for a conventional soldier pile and anchor system around this building. Tender provision was also made for more extensive geotechnical investigation and rock defect mapping through the project to validate the design parameters adopted for the alternative design. The presence of weak dykes was not considered to represent a significant risk because the dykes in the MM are usually of the order of 1-2m in thickness and adjustment to nail spacing and size could be readily made to accommodate dyke occurrences of this magnitude.

FURTHER GEOTECHNICAL INVESTIGATION AND DESIGN

Five additional bores were drilled at selected locations around the site, angled at around 15" from the vertical. A high precision borehole imaging camera survey of each hole was carried out to provide comprehensive data on rock defect spacing and orientation using the RAAX™ system. This proved to be invaluable and quickly revealed a bedding plane related problem along all of the Swanston Street (West) wall. The bedding dipped at 40" directly into the excavation and immediately ruled out the use of rock nails for this wall. Other walls did not contain unfavourable bedding and rock nailing appeared to be feasible for these walls.

Further testing of the MM core from these bores was undertaken to augment the original site investigation data. In addition, two deep excavator test trenches were dug in the eastern part of the site. One of these towards the south eastern corner of the site caused concern because it revealed a dyke structure approximately seven metres wide.

Finite difference (FLAC; **1998)** numerical analyses were undertaken for both a stressed anchor support system and the rock nail system for the wall adjacent to the historic State Library building. The models were structured to take into account progressive excavation and anchor (or nail) installation. Rock mass moduli were based on a combination of pressuremeter test information and relationships that have been developed over many years for the MM between weathering, moisture content and rock mass stiffness. Results of these analyses are shown in Fig. 3. Indications were that the rock nail walls would deflect horizontally about twice as much as the walls supported by a stressed anchor system. On the basis of these analyses and uncertainty about using a hitherto untried support system for such a deep basement adjacent to the historic State Library, the decision was taken not to risk the untried rock nail support alternative. Accordingly, stressed anchors were employed along this wall.

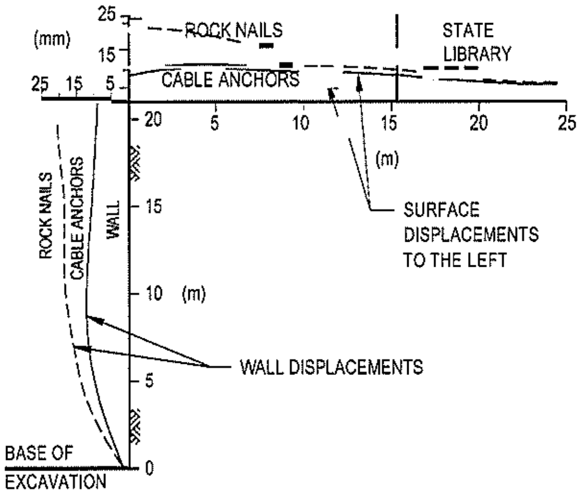

Figure 3. Predicted Horizontal Deflections (FLAC Analysis) - North Side of Excavation

The STEPSIM4 (Baczynski, 2000) software was employed to confirm the validity of the rock mass strength parameters used in the tender designs. The program inputs were derived from the borehole imaging surveys, face mapping and peak and residual rock strength parameters from tests on rock core and from moisture content correlations. Initially the faces were divided into like domains in terms of (primarily) bedding, face orientation and weathering. The computer program then evaluates the most probable failure path through adjacent cells in a particular domain. Failure may be through intact rock or along joints or bedding planes. It was found that, apart from bedding planes, defect lengths were of the order of 0.5m as was the average length of rock "bridges" between discontinuities. The discontinuities were assigned strengths on the basis of type and degree of any infilling and roughness. Approximately *25%* of joints were found to be clay infilled during mapping of the upper excavation levels. This was considered to represent "worst case" conditions because weathering and strength generally improved with depth.

The computer program STEPSIM4 assumes that the predominant failure mode is one of sliding along a two-dimensional surface and the global strength parameters determined for each domain were input into conventional wedge and circular slip stability analyses. Reinforcement was included using the STARES stability program developed at the University of Sydney (circular slip surfaces) and also in simple wedge analyses in which the resolved reinforcing force normal to the potential slip surface did not increase normal force on the plane of sliding. A typical wall section showing one of the circular stability analyses is shown in Fig. 4. A target Factor of Safety of 1.4 was adopted for the analyses. Table 2 also shows the equivalent rock mass strengths established from the STEPSIM4 analyses compared with the parameters adopted for the tender designs.

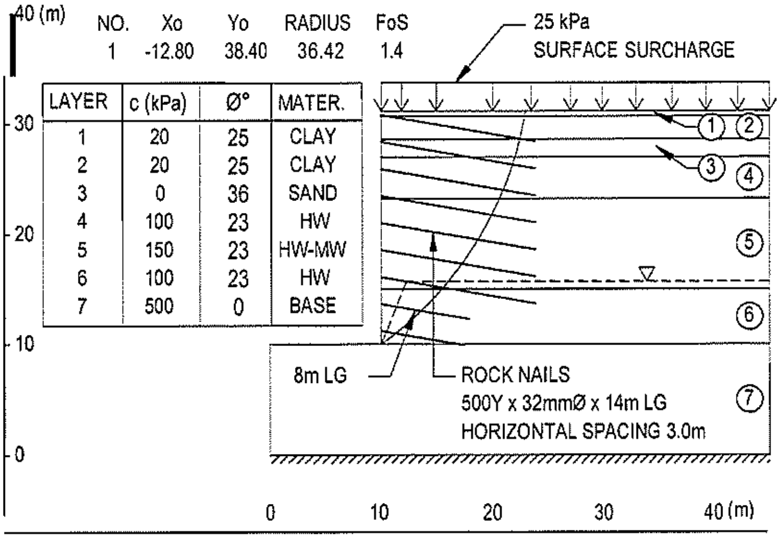

Figure 4. STARES Stability Model - North Wall

DYKE OCCURRENCES

Excavation commenced in the south-west corner of the site and shortly thereafter excessive wall movements were observed. On inspection, a massive weak dyke structure was discovered that extended for a distance of 35m along the Lonsdale Street wall. The material had the consistency of a hard silt, was nearly white in colour and was transected by relict joints. These joints proved to be particularly troublesome, precipitating progressive face collapse, especially where seepages were encountered from high level service trenches just outside the site boundaries. Further investigation and laboratory testing showed that rock nails would be inadequate for wall support in this area and redesign was carried out for a fully tensioned anchor system. Construction stability was still problematic during each excavation stage before anchors could be installed and tensioned but fortunately the dyke broke into a number of discrete "fingers" below a wide higher level cap and stability improved at the deeper levels.

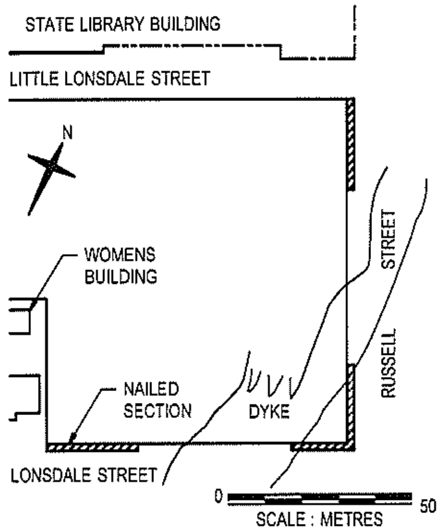

Figure 5. Plan of Main Dyke

A more serious problem was encountered along the east (Russell Street) wall where the dyke structure intersected the wall and passed behind it. Fig 5 shows a plan of the main dyke structure, monitoring of anchor drilling helping to establish the position behind the wall. It was determined that stressed anchors would be required wherever the dyke was within 10m behind the excavation face. Elsewhere, the rock nails could be used. Perversely, the dyke straightened up and ran northwards nearly parallel to the east wall after passing behind it. This resulted in a greater than expected length requiring stressed anchors. Further construction stability problems were encountered, often initiating at the external corners where the progressively excavated vertical rib slots had to be formed. As a result, lateral wall movements increased and a staged hit-and-miss excavation sequence together with additional soil nail support was required to maintain stability. Unacceptably slow progress with this construction sequence resulted in bored soldier piles and stressed anchors being installed for the lower two basement levels through the dyke area along the east wall.

Neither of these major dyke occurrences had been detected during the initial site investigations. The bore intersecting 9.3m of dyke near the north wall had passed down the "throat" of a dyke 1m wide which caused no problems during the retention works. These observations illustrate how unpredictable dyke occurrences in the MM can be.

EXCAVATION MOVEMENTS

Disproportionate wall movements were observed in dyke areas and increases in the capacity of the retention system were made in these areas. The magnitude and rates of wall movements from precise survey and inclinometer installations were used to determine the lateral extent of increased retention measures.

The inclinometer on the east wall (nailed location) showed a possible slip movement at 5.5m depth (see Fig 6a) which was considered a sympathetic response to the movements in the adjacent dyke section. With increased support given to the dyke area, the rate of movement did not increase and diminished rapidly after achieving full excavation depth. Inclinometer movements on the north wall (stressed anchors) increased reasonably uniformly with depth of excavation, reaching a maximum of 37mm at the full excavation depth of 22m (see Fig 6b).

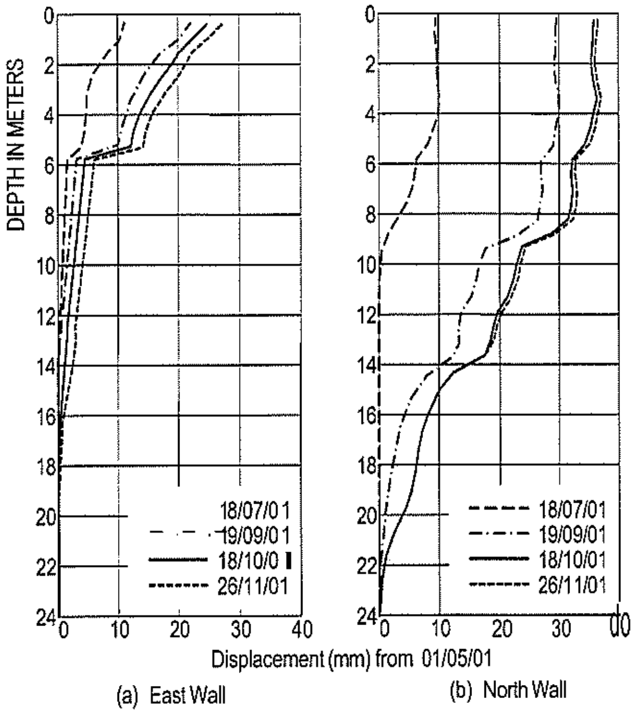

Fig 6. Horizontal Wall Movements

Fig 7 shows the final inwards deflection of the top of all walls around the excavation perimeter. A maximum deflection of 62mm occurred in the dyke area on the east wall. Table 3 shows the ratio of horizontal wall deflection expressed as a percentage of the wall height. Vertical settlements immediately behind the walls were of a similar magnitude to the lateral wall deflections. As expected, in the dyke areas deflection ratios were greatest but generally less than those reported by Goldberg (1976) for strong soils. The deflection ratios for stressed anchor walls were on average 30% less than the average for nailed walls, which was less of a difference than indicated from the FLAC analyses. The FLAC analyses gave acceptable horizontal wall deflections for nailed walls.

The FLAC model lower boundary was 10m below excavation level and probably too close to model excavation rebound accurately. This is considered to be partly responsible for the under prediction of wall movement.

Table 3 : Horizontal Wall Deflections

Conditions·	Horizontal Wall Movements*		
	Range	Average	FLAC
DYKE, Stressed anchors	0.11 - 0.33	0.21	
Mudstone, Stressed anchors	0.04 - 0.15	0.10	0.04
Mudstone. rock nails	0.09 - 0.21	0.14	0.10

* expressed as a percentage of wall height

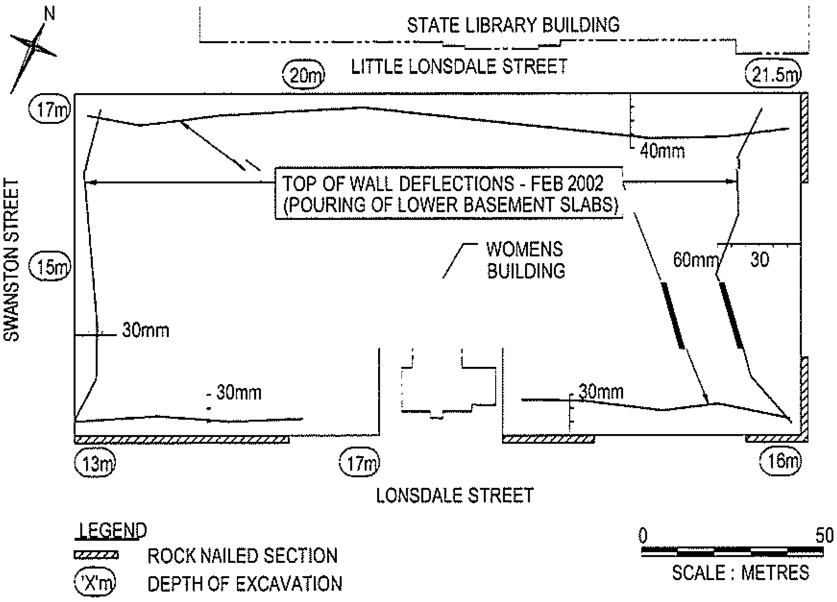

Figure 7. Final Top of Wall Horizontal Deflections

CONCLUSIONS

The unpredictability of dyke occurrences in the MM can have a substantial impact on construction progress, requirements for wall support and hence retention costs. The massive dyke features at this site were much larger than recorded elsewhere in basement excavations in the MM and probably represent a "worst case" scenario. Due to careful deflection monitoring, face mapping and redesign during construction it was possible to avert major potential wall instabilities.

Rock nails were shown to be a viable method of temporary support for basement retention design in highly weathered and closely jointed siltstones and mudstones. The STEPSIM4 software confirmed the adequacy of rock mass strength parameters adopted for wall stability design although rock mass friction angles were evaluated to be about 8" greater than used in the wall designs.

Rock mass stiffness values used for numerical excavation movement analyses appeared to be of the correct order, after accounting for excavation rebound effects that were not effectively modelled in FLAC analyses.

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