DEEP BASEMENTS IN MELBOURNE SILTSTONE

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ABSTRACT

With the growth in renewal of city developments, basements in the Melbourne CBD environs are getting deeper in response to limitations imposed on building heights, surface infrastructure and space constraints, and planning scheme controls. Increasingly, many sites are being re-developed in close proximity to heritage listed or movement sensitive assets, hence the performance of ground retention systems in controlling displacements is of paramount importance. Because of the cost premium of below ground works, construction of deep basements is a significant consideration in any project costing and economic imperatives are driving alternative design solutions, but what are the risks?

1 INTRODUCTION

The Melbourne Mudstone is the dominant lithology that impacts on underground works in the wider city area. The geotechnical properties of the rock substance have been studied in detail over many years through University research programmes and complemented by several large scale field testing trials. However, there are relatively few case histories on the performance of deep basements in fractured weathered rock, outside of some notable enclosed underground works such as the Underground Rail Loop (1970's construction) and the CityLink tunnel (late 1990's).

This paper provides details of two case histories of deep basements constructed in "Melbourne Mudstone", a fractured and folded rock mass, with particular focus on design and performance aspects, together with data on the magnitude and pattern of wall movements adjacent to high value assets. One of the sites has perimeter boundaries defined by a whole city block, 200 m x 100 m, which afforded an uncommon opportunity to undertake full face mapping and structural logging to develop a geological defects model during basement earthworks and installation of the shoring walls.

While basement walls provide both temporary and permanent ground support, they clearly form part of Temporary Works and designs should ensure that all risks, where foreseeable, are appropriately considered. Wall designs in fractured rock typically involve analysis for kinematic stability of critical defect sets and of a continuum model where failure surfaces follow a step-path through the rock mass. The practical limitations and uncertainties in dealing with the latter are discussed. The statistical determination of rock strength and judgemental assessment of parameters for geological features without laboratory testing is examined. Methods for assessing stability risks are also suggested.

2 LITHOLOGY AND STRUCTURE OF MELBOURNE FORMATION

The Silurian age Melbourne Formation comprises a monotonously interbedded sequence of siltstones (dominant), mudstones and thin sandstones. The siltstones consist predominantly of angular quartz and occasional feldspar grains, and the depositional alluvial environment was that of a turbidite fan. Structurally, the strata have been tightly and isoclinally folded and faulted by several phases of mainly east-west tectonic compression. The broad structural axes are typically upright and strike NNE-SSW although the multiphase folding has resulted in a high degree of structural complexity at a local site-scale level. Intrusive igneous dykes are not uncommon and these can significantly affect retention and foundation designs.

In the wider Melbourne area, and outside Quaternary age flooded valleys, the Melbourne Formation has weathered under a chemically oxidising environment that has penetrated to several 10's of metres depth. Deeper zones of weathering occur along major fault lines and other structural defects.

3 QV SITE REDEVELOPMENT

QV is the redevelopment of Melbourne's historic Queen Victoria Women's Hospital site. In 1846 the foundation stone for the Queen Victoria Women's Hospital (at the time named the Melbourne Hospital) was laid. The hospital was closed in 1987 and the site was undeveloped until 2001 when the present QV development commenced. The only remaining building of the original hospital is the heritage listed Women's Centre, a five storey Edwardian brick tower, constructed in 1910.

The site has plan dimensions of 100 m x 200 m and occupies an entire city block – see Figure 1. The development incorporates the original historic laneways as well as the Women's Centre. The development comprises a high density, mixed use precinct containing retail, business and apartments, with high rise towers and deep basements extending for up to 22m below the surrounding street level. The basement excavation was one of the largest and deepest in Melbourne, requiring excavation of more than 320,000 m^3 of material. Immediately adjacent to the site across Little Lonsdale Street is the Victorian State Library, another heritage listed building, incorporating the landmark domed Latrobe reading room, opened in 1913.

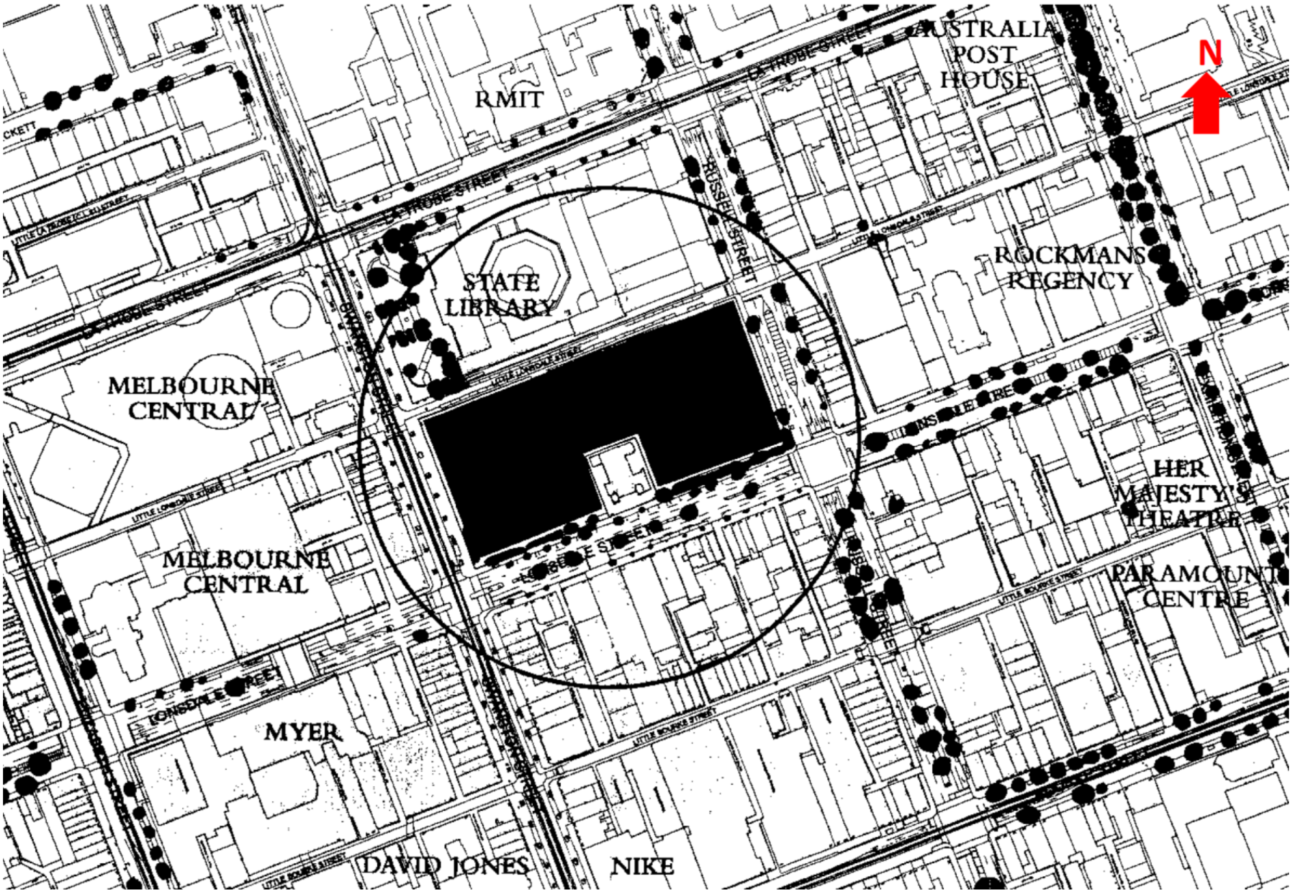

Figure 1: Site Plan QV Project Melbourne

3.1 SITE AND GEOLOGY

The site is bordered by busy city streets, with several vital services buried beneath the pavements close to the site boundaries. These services include a high pressure gas main along Russell Street and a large telecommunications tunnel running parallel to the southern boundary within 14m of the site perimeter.

Numerous geotechnical investigations had been carried out previously, revealing variably weathered Silurian age siltstone and sandstone of the Melbourne Formation (Melbourne Mudstone or MM) beneath a thin cover of miscellaneous site filling. Groundwater was found at about 11-12m below the surface. The investigations provided considerable detail about the variability of strength and weathering of the MM but only limited information on the orientation of predominant joints and bedding planes. The MM had extremely weathered material of extremely low strength towards the surface, with medium to high strength material towards the base of the excavation. The orientation, continuity and character of these defects were expected to dominate the basement retention designs.

Some bores over the eastern portion of the site revealed extremely low strength dyke material, clearly identified on the logs as pale coloured intrusions through the orange and brown oxidised MM. These dykes are prevalent in the MM throughout Melbourne and usually occur as swarms, with individual intrusion thicknesses commonly up to 2m. In one site bore the combined thickness of dyke material was logged as being up to 9.3m within the basement zone.

In view of the limited site data on defects and the prevalence of the weak dyke material, additional investigation was proposed to better define the likely influence of these features. Six inclined bores were drilled over the site and excavator pits were performed where bores had shown shallow depth to dyke material. In each of the bores, camera imagery (RaaX system) was employed to gain as complete an image of the defect orientation and character as possible. Borehole imaging and analysis was carried out using BIPS (Borehole Imaging and Processing System) which gives comprehensive information and graphical images along each bore, including the ability to rotate images to obtain 3-dimensional views. Such testing is relatively expensive but proved invaluable, faster and superior to any of the core-orientation and impression techniques locally available. The defect plots established from these surveys were used together with the rock strength data to establish design profiles for each basement wall elevation.

The excavator pits revealed dyke exposures up to 7m in length in the south eastern portion of the site, much more extensive as a single unit than expected from the bore information and from other exposures in the MM. The other aspect noted with respect to this dyke exposure was a definite jointing pattern, the jointing being very smooth and tight and lacking in cohesion. When wet and disturbed the dyke material tended to slurry and loses strength completely. Excavated blocks of the dyke material fell apart readily at the joints.

3.2 BASEMENT RETENTION CONCEPTUAL DESIGN

Based on the geotechnical investigation information, conceptual retention designs for temporary (construction) support were produced for each basement wall. Permanent retention primary support would be provided from floor propping. Along Swanston Street, bedding dipped directly into the site at angles to the horizontal of 30°-80° but most commonly 40°-50°. On this basis it was determined that the western excavation faces would require cable anchor retention as the friction angle along bedding planes in the MM is typically taken as 23°-25° (Johnston,1992), with the critical failure mode being 2-D block sliding along the bedding.

For the other excavation faces temporary options included cable anchors and nails. Of concern was the presence of larger than usual dyke intrusions in the eastern portion of the site and uncertainty as to the shape and extension of these features.

Other than for the western faces, preliminary nail designs were developed in which Mohr-Coulomb strength parameters related to the different strength and weathering profiles were adopted. This approach had been used successfully elsewhere in Melbourne for retention designs in the MM. For the anticipated dyke areas, lower strengths were adopted based on triaxial testing carried out on undisturbed tube samples. Table 1 shows the strength parameters adopted initially for nails designs.

Table 1: Nail Design Preliminary Material Strength Parameters

Material	Effective Cohesion c' (kPa)	Effective Friction Angle φ' (°)	Bulk Unit Weight γ (kN/m^3)	Ultimate Grout Bond c_a (kPa)
Dyke	5	20	19	120
EW rock	30	25	19	75
HW rock	100	23	22	250
MW rock	200	23	22	350

The computer program STARES (Balaam) was used to perform preliminary equilibrium stability analyses to establish nail lengths and spacing. A minimum Factor of Safety of 1.4 was adopted for material strengths along with a Factor of Safety of 2.0 on nail bond strength.

Calculations were made using the Hoek-Brown (Hoek, Brown 1988) method to provide corroboration of the above strength values but considerable variation in strength values so determined were found due to variations in assumptions embodied in the method.

Due to the risk involved in such deep basements with surrounding critical assets, it was decided to employ a more detailed approach to independently estimate suitable material mass strength values for use in the basement wall equilibrium stability analyses. Furthermore, comparative numerical continuum analyses were performed for both cable anchor and nail designs to determine the relative wall and ground movements associated with each retention type. This was considered particularly important for the north wall of the basement, which was adjacent to the Victorian State Library building across Little Lonsdale Street.

3.3 ROCK MASS STRENGTH PARAMETERS

The data required for this analysis included the rock weathering and strength as logged, confirmed by point load index and moisture content testing. Extensive laboratory and field testing of the MM over the years has shown reasonable correlations between saturated moisture content and both strength and stiffness of the rock material (Johnston,1992). Detailed face mapping in the excavations at the start of basement construction provided additional information required for the analyses. Of major importance in the mapping was information on the continuity of joints, defect spacing, defect roughness and infilling as well as lengths of rock bridges between defects.

The excavation perimeter was divided into structural domains and the defect sets critically orientated for wall stability established. The domains were primarily distinguished on the basis of bedding orientation, with usually three dominant joint sets and up to three less common sets in each domain. A statistical assessment of laboratory data for $I_{S(50)}$ point load index tensile strength was undertaken in conjunction with a review of published relationships for weathering grade vs moisture content, strength vs moisture content and the unconfined compressive strength (UCS) to $I_{S(50)}$ strength ratio. This allowed a statistical shear strength model for the intact rock with respect to various weathering grades.

Using the site logging information, RQD and RMR rock mass classification indices for the site, a statistical shear strength model was developed for the rock mass with respect to failure across the structural fabric, not parallel to it. The STEPSIM4 computer program (Baczynski 2000) was used for the analyses for joint controlled “step paths” through the excavation walls for various weathering grades.

As most joints were relatively short and tended to terminate in rock (i.e. not “cut off” by other defects), a significant effective cohesion value was derived along most joints.

From the STEPSIM4 analyses, shear strength parameters for each category of rock weathering for joint-defined critical “failure step paths” for each section of wall around the excavation perimeter were established. These parameters apply to planar and tetrahedral wedge failures where the respective slip surfaces are joint defined (bedding plane failures excluded). Table 2 shows the deduced design parameters for such failures.

Table 2: STEPSIM4 Shear Strength Parameters for Planar and Tetrahedral Wedge Failures

Weathering Grade	Effective Cohesion c´ (kPa)	Effective Friction Angle φ´ (°)
EW to HW	80±20	29.5 ± 1.5°
HW-MW to MW	240±60	33.5 ± 2.0°
MW-SW to SW	430±90	36.5 ± 2.0°

The shear strength parameters derived using the STEPSIM4 approach were generally above those adopted for the preliminary wall stability designs, giving confidence that the equilibrium stability analyses for nailed walls were conservative.

The methodology for assessing stability risks and the reliability of calculations of planar and tetrahedral wedges was based on two approaches: (i) the Rosenbleuth method of statistical moments and (ii) Monte Carlo simulation. With the former, both the friction angle and cohesion are assumed as independent parameters and stability analyses are then undertaken for every combination of -1 and +1 standard deviation properties. Thus for tetrahedral wedges defined by two defect sets, 2^4 =16 calculations are needed.

3.4 NUMERICAL ANALYSES

In view of the proximity of the Heritage listed Victorian State Library building directly opposite the site and across Little Lonsdale Street, where the wall heights were greatest, comparative nailed wall and cable anchored designs were analysed using the finite difference program FLAC (Itasca, 1998). The analyses enabled the excavation and support sequences (including cable anchor stressing) applicable to both types of retention to be modelled. The wall and ground displacements determined from these analyses are shown in Figures 2 and 3. The analyses showed that both horizontal and vertical displacements for the cable anchored designs were about half of those for nailed support. Hence, cable anchors were specified for the northern basement wall.

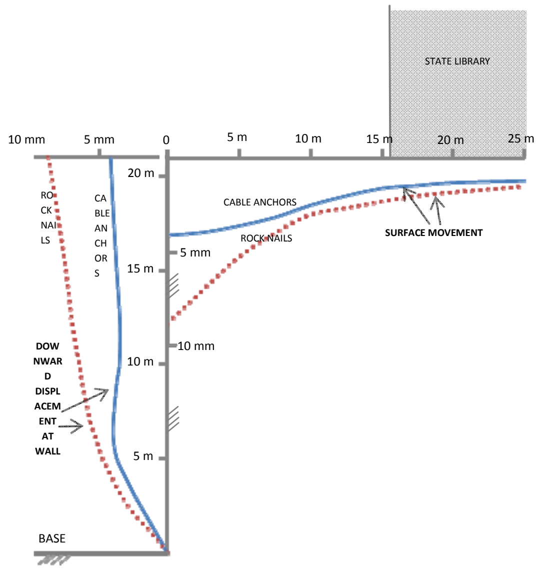

Figure 2: FLAC Analysis of Vertical Movements Cable Anchors vs Rock Nails

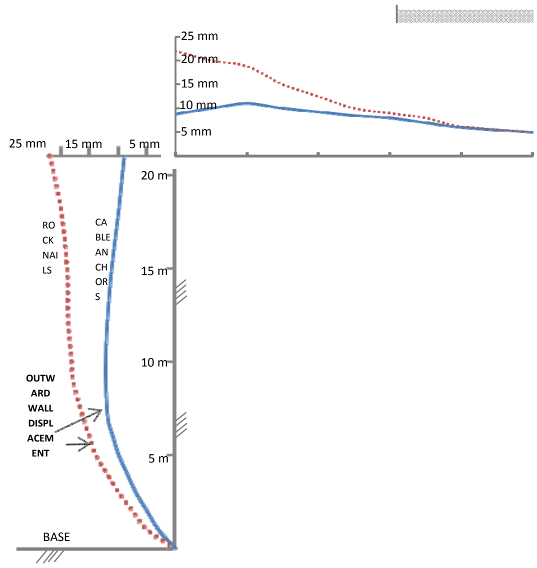

Figure 3: FLAC Analysis of Horizontal Movements Cable Anchors vs Rock Nails

Using a similar reasoning, cable anchors were specified for the excavation walls supporting the Heritage Women's Centre building fronting Lonsdale Street.

3.5 INFLUENCE OF DYKE

Based on the bore intersections and the test pit exposures towards the south-eastern portion of the site, the wide dyke structure was interpreted to intersect the south wall at the eastern end of the site. Accordingly, cable anchors were designed for this part of the south wall. However, early during excavation and support works, significant instability of the face was experienced due to the blocky, jointed nature of the dyke. These problems were magnified by the adopted "top-down" method of forming the wall soldier piles, initially proposed by the designers to shorten the basement construction program. Dyke material collapsed from beneath the shotcreted upper part of the wall, with no soldier piles yet in place to limit the width of wall vulnerable to such collapse.

Short grouted rock dowels in between rows of cable anchors were installed as shear keys to control face instability where these problems were most prevalent in the upper levels of the basement. Figure 4 shows the retention arrangement finally implemented and wall construction continued thereafter with minimal problems.

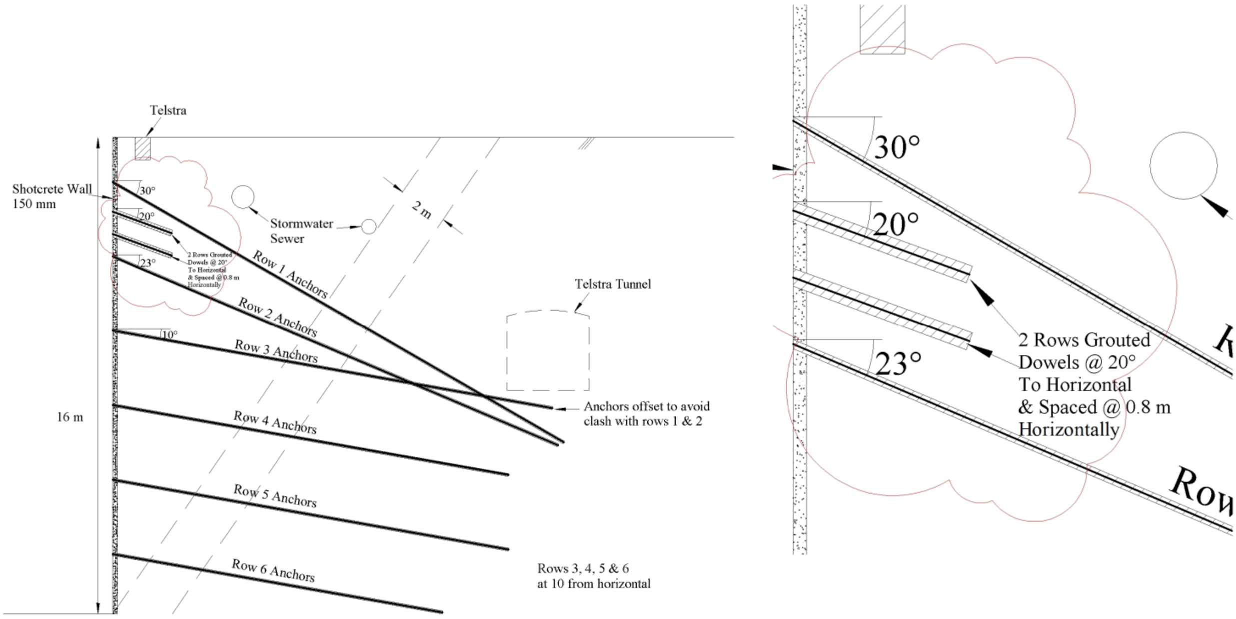

Figure 4: Lonsdale Street East Retention Arrangement to Limit Wall Displacements

A more major problem was encountered along the eastern basement wall as the dyke structure was found to distend and widen further out beyond the wall. This was unexpected as most dykes found in the Silurian siltstone around Melbourne are of limited width and tend to exist as reasonably linear features. This particular dyke structure was entirely dissimilar, instead resembling a widening plug above an intrusive lower shaft. The plug had a very irregular shape, only fully understood as the excavation progressed and the face and inclined drill holes carefully mapped. Figures 5 and 6 show the shape of the dyke on the eastern wall. The maximum width of the dyke, about 30 m, is the largest encountered in Melbourne in the authors' experience. The face collapse problems encountered on the south wall also occurred on the eastern wall and due to the jointing pattern within the dyke were more difficult to contain. The integrity of a high pressure gas main parallel to the east wall was a concern as wall movements associated with the initial face collapses were higher than observed elsewhere around the excavation and some pavement cracking outside the excavation had begun to develop. A combination of shorter excavation stages of limited width and the installation of soldier piles in front of the wall over the lowest basement level were adopted to limit face instability.

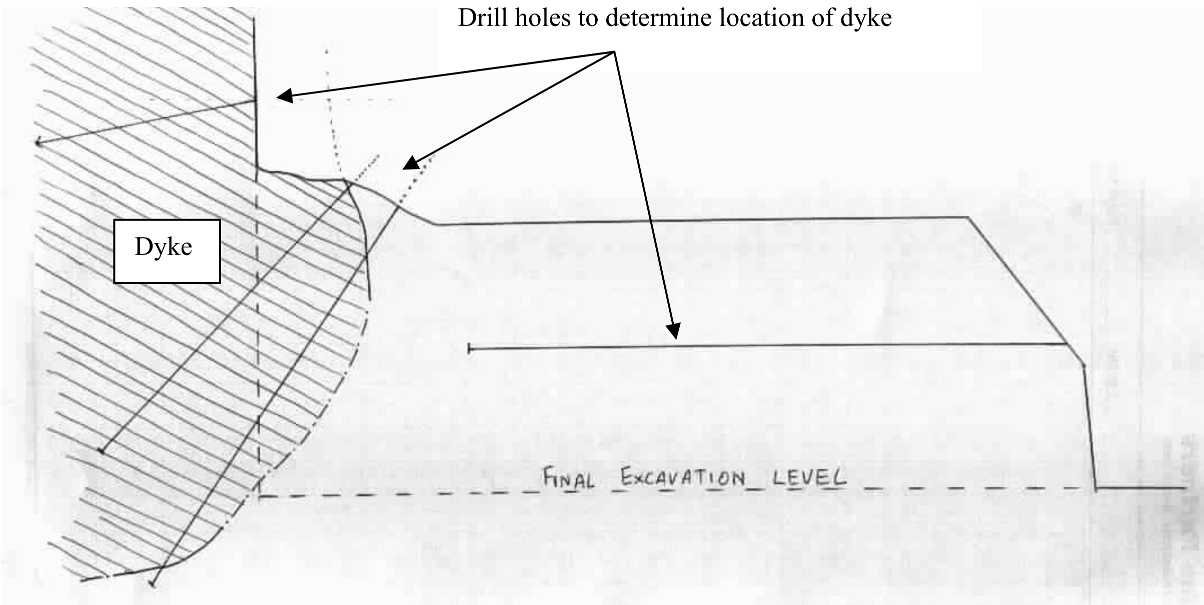

Figure 5: Section through Russell Street Wall Section A-A

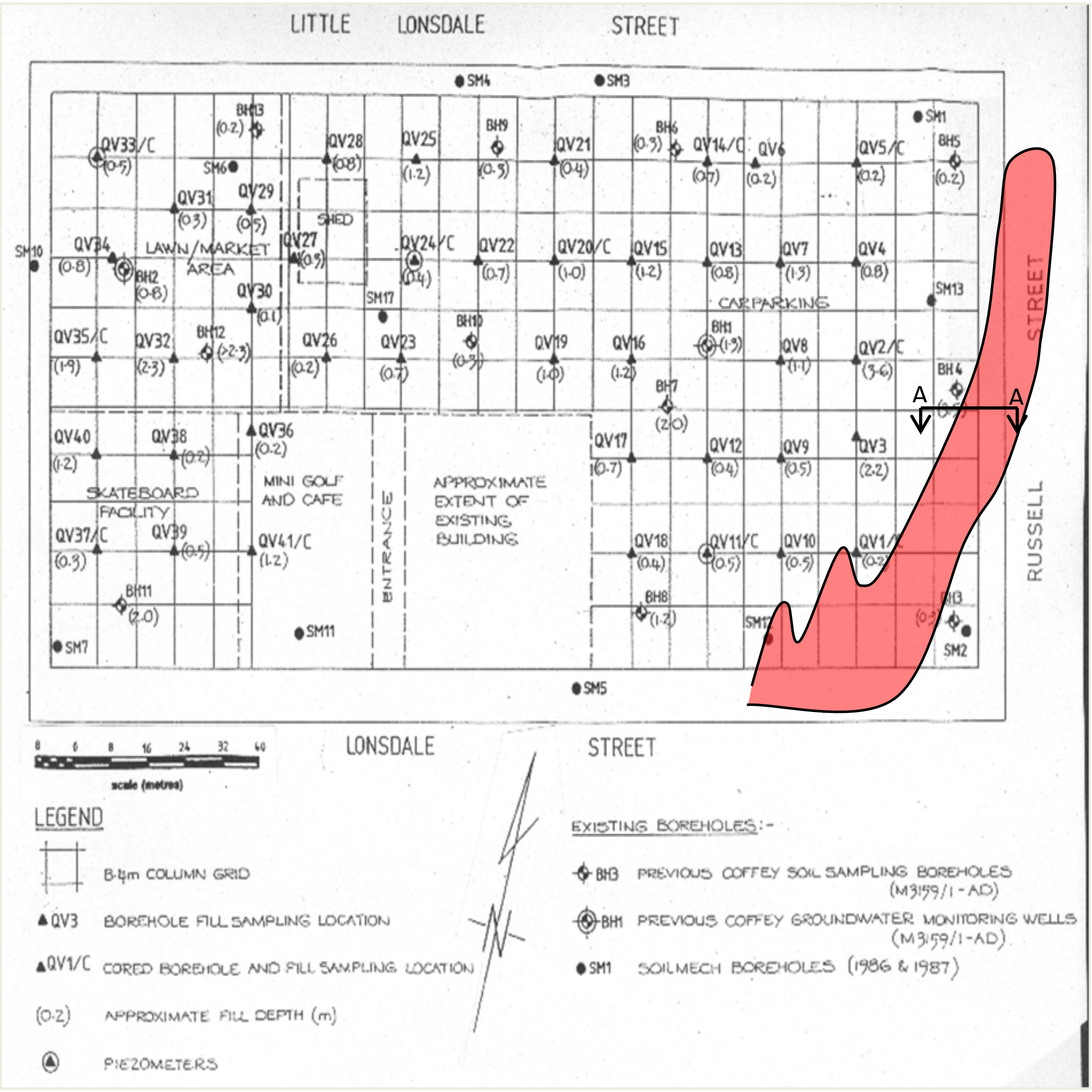

Figure 6: Site Plan Showing Major Dyke Intrusion – Mapped During Excavation and Hole Drilling

3.6 WALL DEFLECTIONS

Figure 7 shows the inwards movements of the top of the wall around the excavation at the time of pouring the lower basement slab. Interestingly, wall deflections in nailed areas were similar to those in cable anchored areas where there was no influence from the large dyke. The larger lateral wall movements along Little Lonsdale Street towards Russell Street are considered to be the result of a deeper weathering and strength profile at this end of the wall.

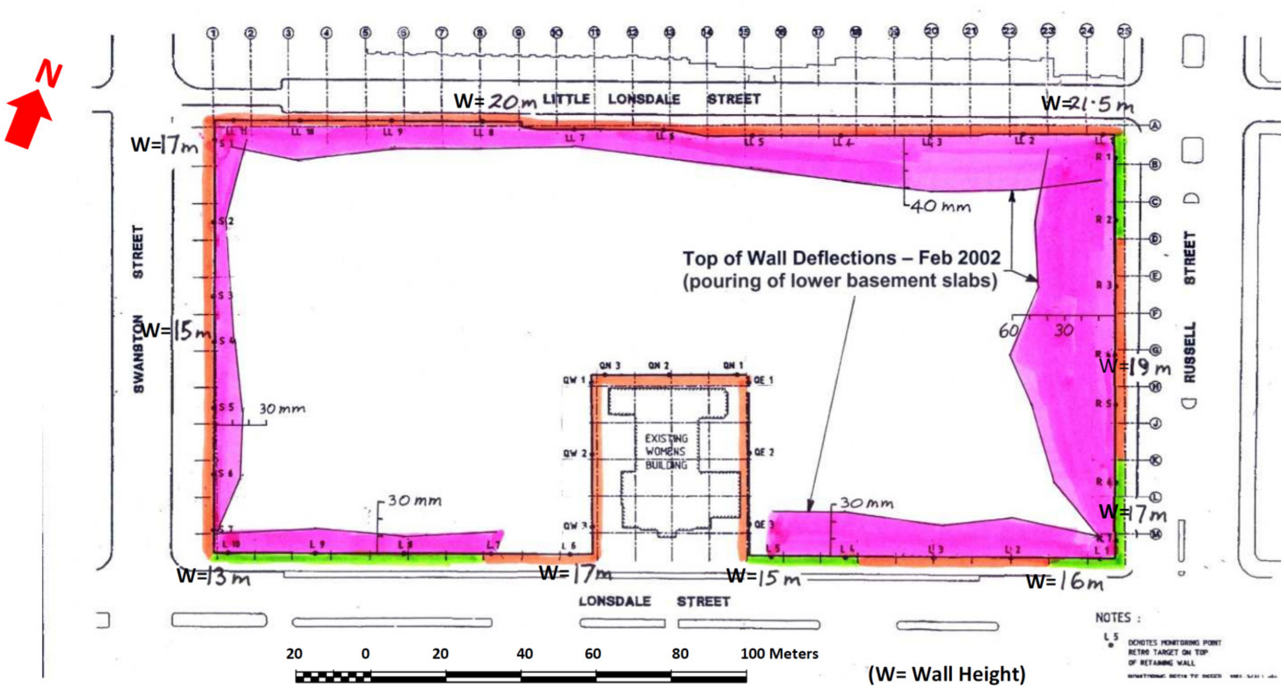

Figure 7: Top of Wall Lateral Movements at Start of Pouring Lower Basement Slab

4 RCH DEVELOPMENT

The new Royal Children's Hospital (RCH) is located in the inner northern Melbourne suburb of Parkville. The 165,000 square metres hospital is directly adjacent to the old RCH building. The hospital complex comprises a multilevel basement up to 18 metres deep and 5 – 6 above ground levels. Development required the construction of 600 lineal metres of anchored soldier pile walls around the perimeter of the site and abutting the old operating hospital with a sensitive laboratory facility that housed ongoing long-term experiments. The main structure is supported on pad footings. A plan of the basement footprint and surrounding features is shown in Figure 8.

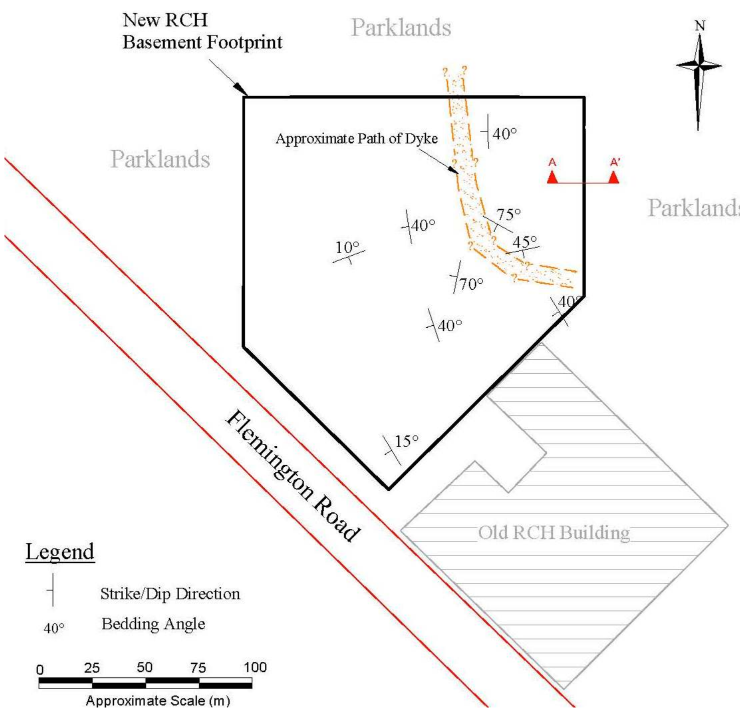

Figure 8: Site Location of RCH Development

The old hospital comprised a series of interconnected buildings. The building abutting the new basement had up to 3 above ground levels and was supported on spread footings. The main old hospital building, with 10 above ground levels supported on spread footings, was set back approximately 8 m from the new basement.

4.1 SITE GEOTECHNICAL CHARACTERISTICS

The site is underlain by Silurian age siltstone (Melbourne Mudstone) of varying weathering degree and strength which generally improves with depth. The ground profile comprised 3 m to 5 m of residual clay underlain by weathered siltstone. The siltstone was initially extremely weathered, extremely low strength, and became highly weathered to moderately weathered of very low to low strength. An intrusive leuco-granitic dyke highly decomposed to soil like material was encountered in the north eastern part of the site at about 15 m depth. The dyke material was identified at the investigation stage and its persistence was traced during construction. The dyke was associated with a sheared/fault zone and the rock material immediately above and below was more weathered. The approximate extent of the dyke material is shown in Figure 8. The siltstone was fractured and folded with varying strike and dip of the bedding and was disrupted around the dyke. Structural orientation of the strata is shown in Figure 8. No groundwater was encountered above lowest basement level.

A series of point load strength index (Is50) tests and saturated moisture content tests were undertaken on siltstone samples to estimate the rock strength. Strength estimation from saturated moisture content based on empirical correlations is commonly adopted for Melbourne Mudstone and has been found to provide broad but generally reliable trends, which is typical for mudrock suites that exhibit progressive weathering. The saturated moisture content typically ranged between 8% and 13%, corresponding to an unconfined compressive strength of about 2 MPa. A plot of saturated moisture contents with depth is presented in Figure 9. UCS tests performed on three dyke samples recorded strengths of between 0.8 MPa and 1.2 MPa.

4.2 BASEMENT RETENTION & CONSTRUCTION

The basement retention comprised a perimeter anchored soldier pile wall with shotcrete infill panels. The piles were non load bearing. The retention was designed based on uniform earth pressure distributions of 4H (no adjacent structures) and 6H (with adjacent structures), where H is the retained height.

The perimeter piles were typically 600 mm diameter bored piles installed at 2.4 m centre to centre spacing, reduced to 1.6 m for the section of retention adjacent to the old hospital building. Piles were embedded 2 m to 3 m below basement level. The piles were restrained by multiple rows of temporary prestressed cable anchors. The anchor holes were 150 mm diameter and were drilled by rotary percussion with air flush. The bonded length of the ground anchors started at least 1 m outside of a line taken up at 45º from the base of the lowest basement. Ground to grout adhesion values adopted for the different material types are shown in Figure 9.

During construction, logging of the shaft material was undertaken for some piles to assess the condition of the siltstone. Samples were collected for saturated moisture content testing to verify the field strength assessments. After completion of the perimeter piles, a staged bulk excavation was undertaken to enable progressive installation of anchors. The excavation was generally taken to not more than 0.5 m below each row of anchors. Appropriate proof load testing was undertaken to confirm sufficient bonding between the ground and the grout.

4.3 MONITORING RESULTS

A series of survey monitoring points were established along the perimeter wall capping beams and at varying depths on selected perimeter piles prior to the start of the bulk excavation in February 2008. These points were surveyed on a monthly basis to assess performance of the retention system. The bulk excavation and basement retention was completed in July 2008 while construction of the basement floors began in November 2008.

From February 2008 to November 2008, a maximum lateral movement (towards excavation) of 8 mm was recorded for the points on the perimeter capping beam adjacent to the old hospital building. Away from the adjacent old hospital, lateral movements of between 5 mm and 15 mm were recorded at the capping beam over the same period. In all instances the recorded lateral movements were less than 0.1% of the retained height.

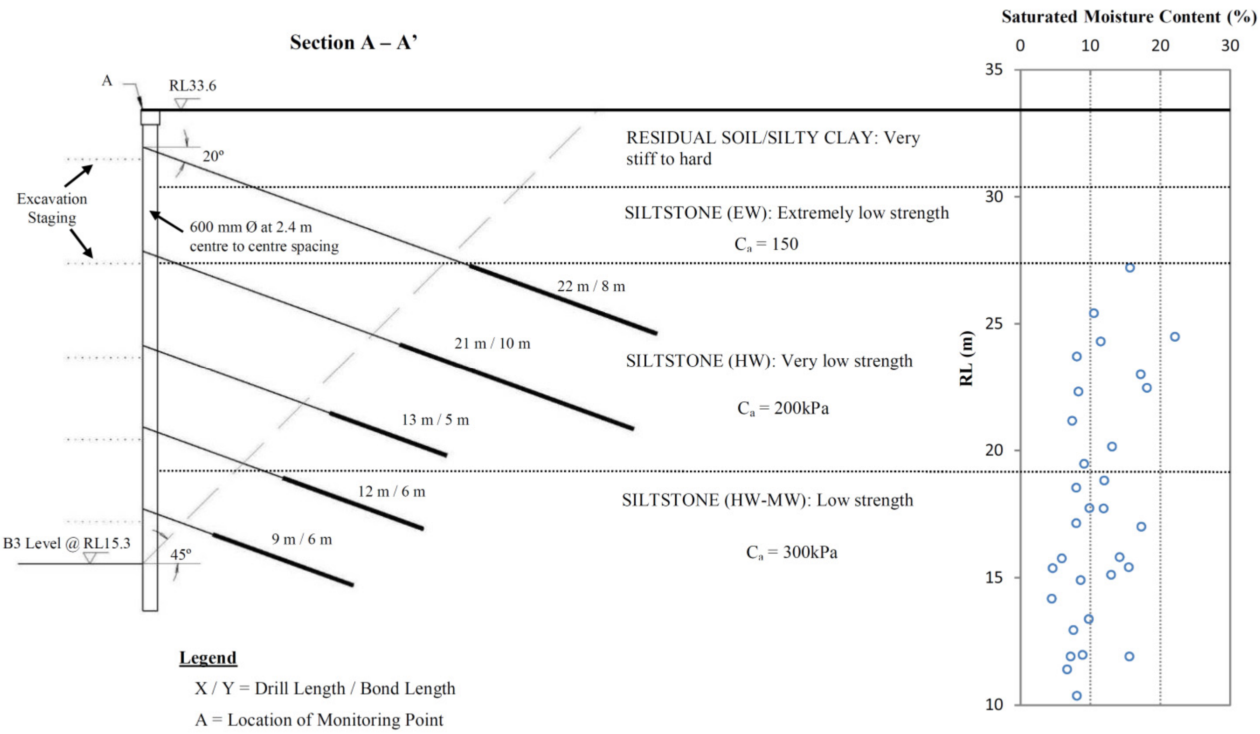

Figure 9: Ground Retention Details

5 CONCLUSIONS

The support design for the QV development was configured to provide lower risk of ground movement near Heritage Listed buildings on and adjacent to the site, with numerical analyses carried out to quantify ground displacement risks for different support options. The use of the STEPSIM4 statistical approach to establish rock mass shear strength parameters for planar and tetrahedral wedge type failures controlled primarily by defects confirmed that the preliminary design values were conservative.

The high variability of the strength and weathering of the MM necessitated a comprehensive program of drilling and defect mapping to enable appropriate retention measures to be implemented at different sections of the basements walls. The additional complication of an unexpectedly large dyke structure necessitated close monitoring and amendment to retention methods and staging to control wall movements around the excavation. Innovative techniques were adopted to control face stability in the dyke areas.

At the RCH site a conventional bored pile cable ground anchor combination was adopted over the full retained height to minimise adverse effects on an adjacent high-value very movement sensitive operational research facility. This method of retention was also chosen to negate foreseeable risks due to potential variations in the structural complexity of the strata and ill-defined orientation of an intrusive dyke (or cluster of dykes) traversing the site.

While the so-called observational method has many benefits and should be compulsory practice in complex ground conditions to ensure the integration of design, construction and monitoring, it is not necessarily an appropriate substitute for initial robust retention designs where adverse geological features requiring unscheduled augmentation of the retention may only become identifiable at late stages in deep excavations.

The experiences at the QV site emphasise the importance of a careful risk-based approach to site characterisation, support design, monitoring and support redesign to respond to unexpected conditions. The MM is a highly complex geological unit that should not be underestimated and provides many challenges for deep basement design and construction.

6 ACKNOWLEDGEMENTS

The STEPSIM analysis was undertaken by Norbert R Baczynski, Senior Geotechnical Consultant to Douglas Partners at the time.

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