The Role of Rock Modulus in Assessing the Impact of New Buildings on Existing Tunnels

S T Srithar
BSc (Hons), MASc, PhD, MIEAust, CPEng
Associate, Golder Associates Pty Ltd, Melbourne, Australia
N D Benson
BSc (Hons), MSc, DIC, FGS, CGeol
Associate, Golder Associates Pty Ltd, Melbourne, Australia
C M Haberfield
BE (Hons), PhD, FIEAust, CPEng
Associate, Golder Associates Pty Ltd, Melbourne, Australia

Summary: The construction of a new twenty-six level apartment building with a five level basement in the Melbourne CBD had the potential to impact on the existing underground railway tunnels of the Melbourne Underground Rail Loop (MURL). The four tunnels are located in Silurian age siltstone at approximately 25 m and 35 m below the surface. Approval for the project required minimal impact on the tunnels, with a particular criteria for the decrease of the hoop stress in the tunnel concrete lining. Analyses indicated that the assessed impact of the development on the tunnel lining depended on many factors including ground conditions, rock mass modulus and the cross-sectional properties and condition of the tunnel lining. This paper describes the project and the ground conditions, the assessment of rock mass of Young's modulus and the analyses carried out to assess potential impact on the tunnels. Comparisons are also made between assessed and measured movements.

INTRODUCTION

Redevelopment of the site of the former Victorian Police Headquarters at 336 Russell Street in Melbourne involved the construction of a new twenty-six level residential tower with a 5 level basement. The site is bounded by Russell Street to the west, LaTrobe Street to the south and Mackenzie Street to the north. Heritage buildings and facades were retained along Russell Street to LaTrobe Street (existing 12 storey building) and Mackenzie Street (existing 1880's 3 storey building). Melbourne Underground Rail Loop (MURL) tunnels (Caulfield, Burnley, Clifton Hill and Northern Loop) are located beneath LaTrobe Street on the southern boundary. The redevelopment of the site included a major five level basement, ramping up to one and a half basements at the LaTrobe Street boundary.

There was potential for the development to impact on the tunnels due to stress relief during basement excavation and reloading from construction of the tower. This was a significant design issue. Approval for the project relied upon minimal impact on the tunnels. In particular, there was a requirement to demonstrate that the decrease in the hoop stress in the tunnel linings resulting from the development would be less than 350 kPa (which equates to about 5 microstrain). The stress relief resulting from the basement excavation would cause a temporary reduction in stress in the tunnel liners which would be reversed, and additional compressive stress imposed, by the construction of the tower.

DETAILS OF THE PROJECT

The site of the redevelopment was originally occupied by several buildings of 3 to 12 stories in height, built prior to the construction of MURL tunnels. Parts of the existing buildings along the frontages, (which are Heritage listed), were retained and integrated into the new residential tower development. The remaining structures were demolished. A site plan showing the tower footprint is presented in Figure 1.

The new tower was supported on a combination of pad footings and six large diameter rock socketed piles for major structural columns located close to the LaTrobe Street boundary of the site. Ground retention for the basement construction comprised soldier piles, shotcrete infill panels and temporary ground anchors. The eastern boundary was formed against an existing three storey building.

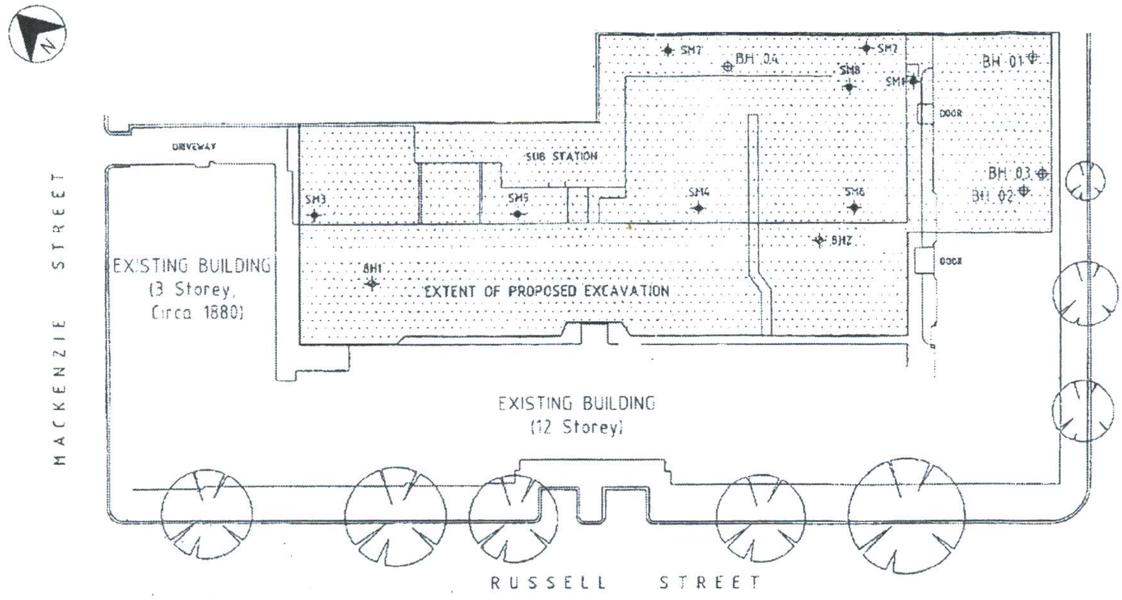


Figure 1 : Site Plan

MURL TUNNELS

The four MURL tunnels near the site were constructed in the 1970's using a Tunnel Boring Machine. The lower tunnels (Burnley and Northern) were constructed prior to the upper tunnels (Caufield and Clifton Hill). The tunnels are circular, of about 6 m internal diameter, with springlines at about 24 m depth and 35 m depth respectively for the upper and lower tunnels. The lateral offset from the site boundary to the closest outer edge of the upper and lower tunnels (Caufield and Burnley) is 4.5 m. The tunnels were excavated with temporary steel rib supports at 0.75 m centres in a zone parallel to the LaTrobe Street frontage of the project. The permanent liner of the tunnels is a 400 mm thick concrete liner with nominal reinforcement. Construction records indicate that some overbreak occurred in this area due to poor ground conditions.

A dilapidation survey was carried out to assess the condition of the closest tunnels prior to any construction activity. The survey indicated significant cracking of the lining and water seepage through the cracks at a number of locations.

GROUND CONDITIONS

The subsurface profile at the site comprises a thin layer of silty clay, underlain by variably weathered siltstone and sandstones of the Silurian age Melbourne Mudstone formation. Neilson (1970) developed the weathering system using zoning numbers which was utilised during construction of the MURL tunnels. The zoning numbers are now rarely used. The siltstone is extremely weathered at shallow depths grading to highly to moderately weathered at the invert level of the lower tunnels. The lower tunnels were logged during construction as Zone 3 to 2 (moderately to highly weathered) Melbourne Mudstone with highly and moderately weathered intrusive materials (e.g. quartz porphery) and associated shear zones. The upper tunnels were logged as Zone 3-2 and Zone 2 (highly weathered) Melbourne Mudstone, again with a complex of shear zones and extremely weathered intrusive materials. The intrusive and shear zones are parallel to bedding which is steeply dipping and strikes north-north-east. The projected orientation of the intrusive and major shear zone indicated that it was expected to cross the south-east corner of the site.

An idealised subsurface profile at the south-east corner of the site near the MURL tunnels is as follows:

- Very stiff clay – 0 to 2 m depth
- Extremely Weathered (EW) siltstone – 2 m to 6 m depth
- Extremely to Highly Weathered (EW-HW) siltstone – 6 m to 13.5 m depth
- Highly Weathered (HW) siltstone – 13.5 m to 30 m depth
- Highly to Moderately Weathered (HW-MW) siltstone – below 30 m depth

This profile represented a deep weathering profile associated with the main intrusive and shear zones. Significantly higher quality material exists in other parts of the site away from the intrusive and shear zone.

ROCK MODULUS

Rock mass moduli were initially assessed from historical data, rock mass classification systems and information from earlier boreholes drilled on or in the vicinity of the site. This was confirmed by additional investigation with pressuremeter testing. The historical data comprised empirical relationships based on intact rock strength, plate load test results, pressuremeter test results, and shaft only, base only, and full pile load test results. Where possible load and unload moduli from the same test were also assessed. Further details of the assessment of the data on rock mass modulus of the Melbourne Mudstone are set out in Benson and Haberfield (2003).

The results of this assessment indicated that the compressive rock mass modulus:

- Increased with decreasing saturated water content of the intact rock.
- Increased with increasing uniaxial compressive strength, q_u, of the intact rock, with most values falling in the range 50 q_u to 400 q_u.
- Tended to decrease with increased fracture count, although no clear trend was apparent.
- Showed similar values across the range of assessment methods.

A compilation of historical data is shown in Figure 2 which plots Young's Modulus against water content. Trend lines of modular ratios (Young's Modulus divided by q_u) of 50, 100, 200 and 400 are also shown. On the basis of this assessment, a modular ratio for mass Young's Modulus of $E_m = 100q_u$ was adopted for compressive loading.

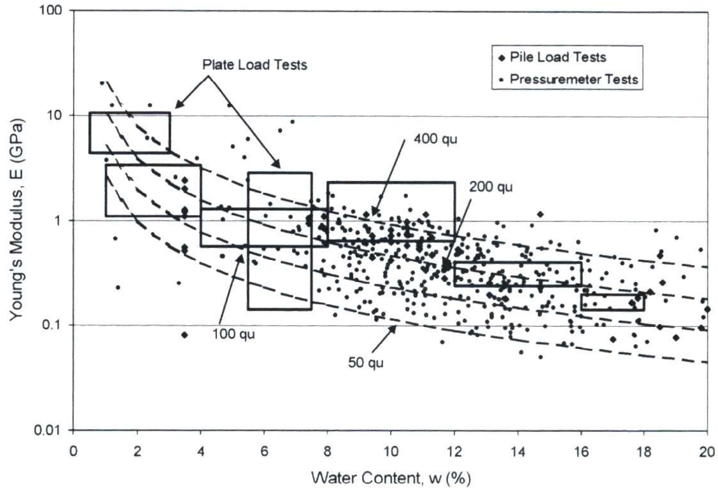


Figure 2 : Compression moduli

The results of historical pile load tests and pressuremeter tests also indicated that the unload reload modulus was between about two and ten times greater than the initial loading modulus. Comparison of initial and unload/reload modulus as assessed from the above tests is shown in Figure 3. The historical data also indicated that this ratio increased as the rock became more fractured (refer to lower values of modulus in Figure 2). On the basis of this assessment, a ratio of unloading modulus to initial modulus of three was chosen.

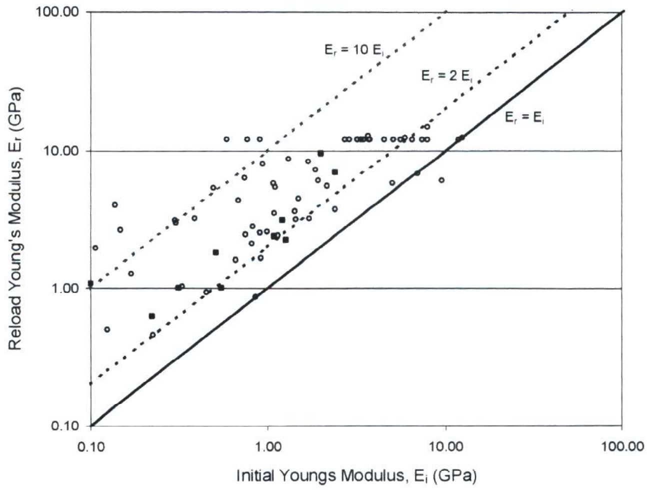


Figure 3 : Reload versus initial Young's moduli

On the basis of the data shown in Figure 2, the loading mass modulus values adopted in the FLAC analyses varied from 150 MPa for extremely weathered siltstone ($w \approx 16\%$) to 650 MPa for highly to moderately weathered siltstone ($w \approx 8\%$). Moduli values of three times the loading moduli were adopted for unloading situations.

ASSESSMENT OF IMPACT ON MURL TUNNELS

Methodology

The assessment of the impact of the excavation and building load on the tunnels was performed using the program FLAC. This is a two-dimensional explicit finite difference analysis program capable of analysing complex soil-structure interaction problems. Structural elements such as the tunnel liner were modelled as beam elements attached to soil/rock elements.

As the stress in the tunnel liner was considered critical, it was important to model the existing stress condition of the tunnel. Based on the available information on the construction of the MURL tunnels, the following construction sequence was adopted in the FLAC modelling:

- Excavation of tunnels in the sequence of Burnley Loop, Northern Loop, Caufield Loop and Clifton Hill Loop. The temporary steel rib supports were placed after allowing for some stress relief and the permanent concrete liner was installed in the bottom tunnels (Burnley and Northern Loops) prior to excavation of the upper tunnels. After the placement of concrete liners in all the tunnels, the temporary steel rib supports were removed due to possible deterioration due to corrosion. A 0.5 m soft zone with one third of the loading modulus was also assumed around the tunnels.
- Demolition of the existing building.
- Installation of site retention piles and anchors.
- Basement excavation within the site in stages.
- Application of new building load.

A number of FLAC analyses were performed to assess the sensitivity of the various parameters in the model:

- Horizontal stress in the rock – Horizontal to vertical stress ratios (K_0) of 1 and 1.5 were considered. Available information for Melbourne siltstone (Fitzgerald 1976, Brown et al 1990) indicates that a K_0 of 1.0 is reasonable. Values less than 1 were not considered as this case was not critical.
- Rock modulus – The loading modulus values as presented above were generally used for the analyses. The basement excavation and the application of new building load were also analysed using both unloading and loading modulus values.
- The calculated design moment of inertia (bending stiffness) of the intact tunnel liner was reduced by a factor of 4. The tunnel liner was constructed in eight segments and based on Wood (1975) a reduction by a factor of 4 is considered reasonable. However, this does not account for the current cracked nature of the tunnel liners. A further 50% reduction was considered reasonable to represent the current nature of the liners, and this was also considered in the analyses.

Results and Discussion

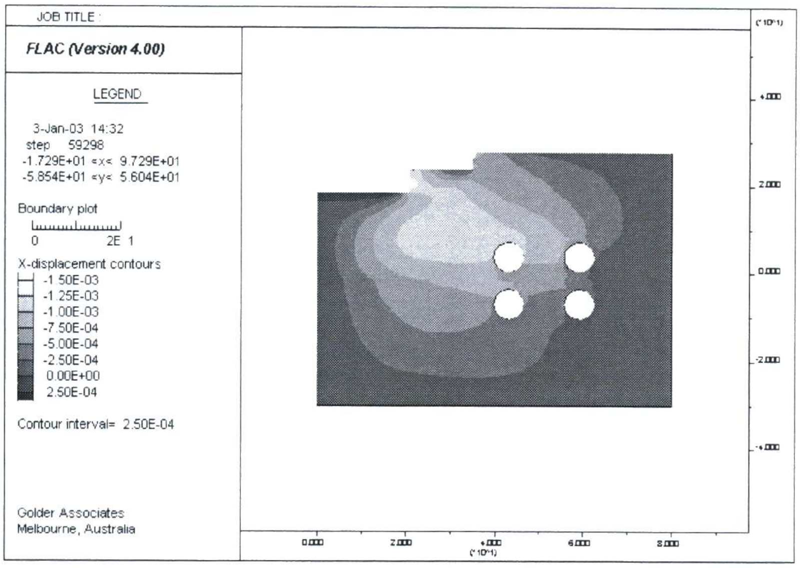
The most severe effect on the tunnels was assessed to occur after excavation. The calculated horizontal and vertical displacements around the tunnels after the excavation are shown in Figure 4. The calculated displacements in the vicinity of the tunnels were less then 2 mm. The axial (hoop) stress and the bending moment distributions along the tunnel liner for the lower tunnel close to the excavation, which was the most affected, are shown in Figure 5.

The sensitivity analysis indicated reasonably small changes in stresses and displacements with changes in input parameters. The assessed change in hoop stress in the tunnel liner after excavation was assessed to be less than 250 kPa. However, when considering the change in bending moment, the maximum change in extreme fibre stress was assessed to be about 600 kPa. The resultant stresses in the tunnel liner were nevertheless compressive even after the above reduction in stresses. It should be noted that for the requirement of 350 kPa change in tensile stress to be adopted for the extreme fibre stress, it would only require a change in bending moment of 3.7 kNm. The change in curvature caused by this bending moment would result in diametrical change of about 0.6 mm of the tunnel, which is very small. The modulus values used in the analyses were also conservative when compared to those measured on site by insitu pressuremeter testing. Higher modulus values would result in less impact on the tunnel lining.

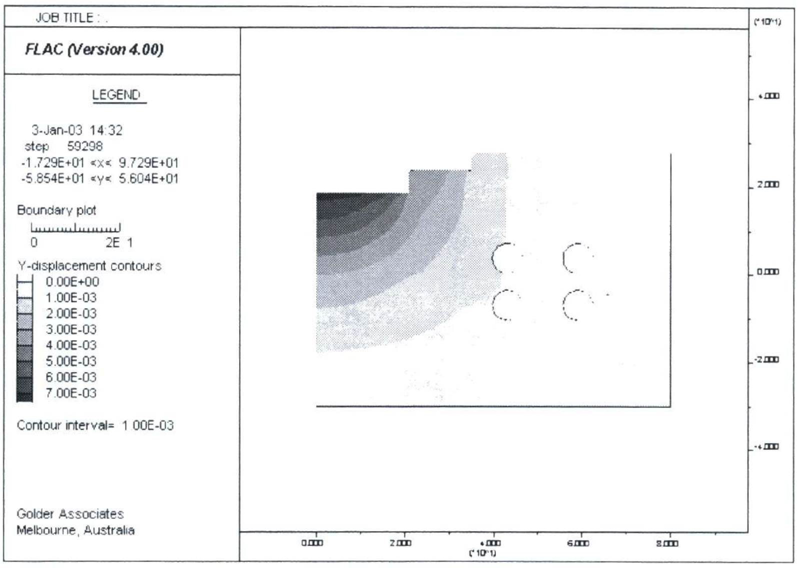
Considering the above, approval for the project was granted with a condition of monitoring the conditions at the site to confirm the analysis predictions. As the real change in stress in the tunnel liners is difficult to measure, a displacement based criterion was selected.

BORED PILE DESIGN

The original design at tender stage involved the installation of six 50 m long, 1.5 m diameter piles with working loads of between 12MN and 22 MN. The perceived purpose of the piles was twofold: to act as passive tension anchors to reduce displacement due to excavation of the basement, and to act as support for the building tower. By adopting the unloading modulus during analysis of the excavation stage, it was demonstrated that the piles would provide very little restraint to displacement during excavation. As a result, following pressuremeter testing and further pile analysis using the program ROCKET, on site logging during construction and adoption of a serviceability approach, piles were designed to take the building loads above and constructed 1.2 m diameter and 14.0 m to 27.6 m in length. Significant savings were therefore achieved.



(a) Horizontal displacements (m)



(b) Vertical displacements (m)

Figure 4 : Displacements from FLAC analyses after excavation

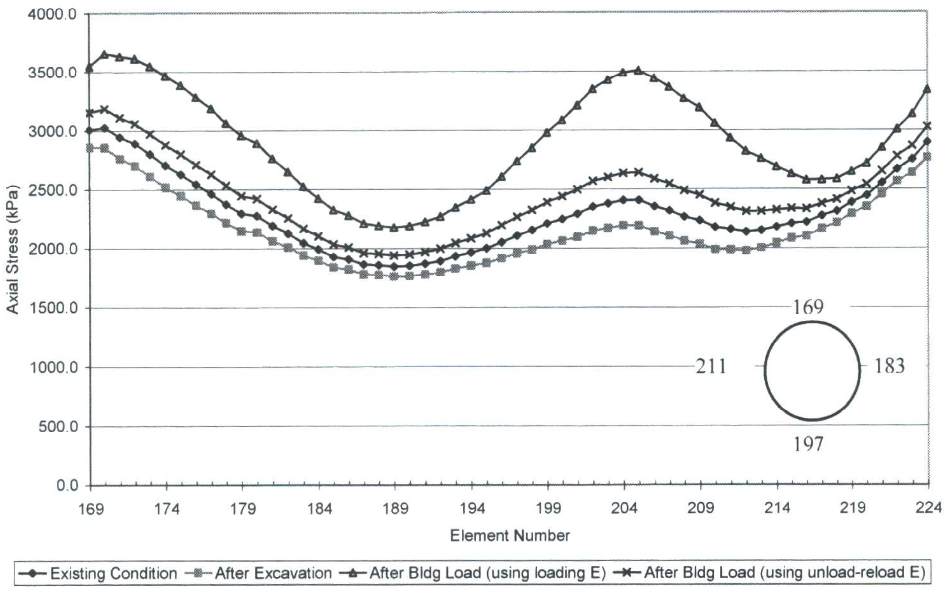
MONITORING OF MOVEMENTS

The monitoring of movements at the site included:

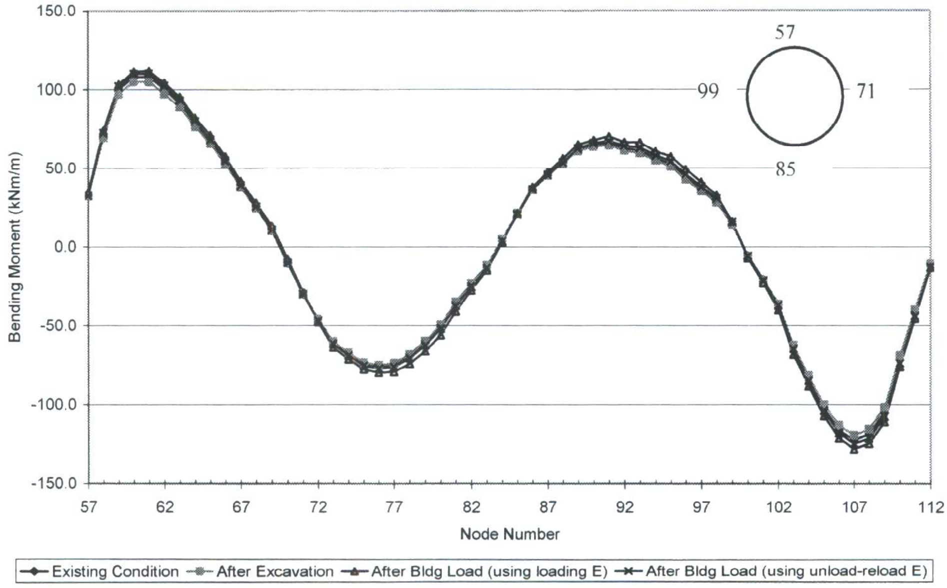
- A 40 m deep inclinometer to monitor lateral movements with depth.
- A wire extensometer (anchors at 4m intervals from 7 m to 35 m depth) to monitor vertical movements.
- A number of line and level survey points around the site to monitor the movements at the ground surface.

Plots indicating the measured movements in the inclinometer and the extensometer over the 5 months of basement excavation works are shown in Figures 6 and 7. The inclinometer results show no apparent trend of movement - the readings observed were within the reading accuracy of the inclinometer probe. The movement of 8 mm is inconsistent with reading taken before and after and is thought to be an anomaly.

The movements observed in the wire extensometer were generally less than 0.5 mm. The distinct 1 mm to 2 mm movements observed around 1 February 2003 and 14 April 2003 are due to heavy machinery working near the extensometer causing movements of the head assembly.



(a) Variation of axial stress around circumference of liner



(b) Variation of bending moment around circumference of liner

Figure 5 : Structural actions in liner of Burnley Loop Tunnel calculated by FLAC

The surface movements observed at the survey points were less than 2 mm, which are within the reading accuracy of the survey. In summary, the measured movements were less than those assessed from the analyses.

SUMMARY AND CONCLUSIONS

Assessment of the potential impact of the excavation and building load was performed using the program FLAC. The Analyses indicated that the potential impact of the development on the tunnel lining depended on a number of factors, the most important of which comprised the ground conditions, the modulus of the siltstone and the cross-sectional properties and condition of the tunnel lining. The greatest impact on the tunnel lining was assessed to occur during the excavation stage.

Several methods were used to assess the modulus of the siltstone. These included historical data obtained for the original tunnel construction (plate load, seismic and laboratory test results), pressuremeter tests and assessment of modulus from correlation with saturated water content. As the excavation resulted in unloading of the siltstone, significant attention was focussed on the unload and reload moduli rather than the loading modulus. Adoption of different loading and unloading moduli resulted in considerably smaller displacements and significant cost savings in foundation installation. Monitoring during construction verified that adoption of unloading moduli was appropriate.

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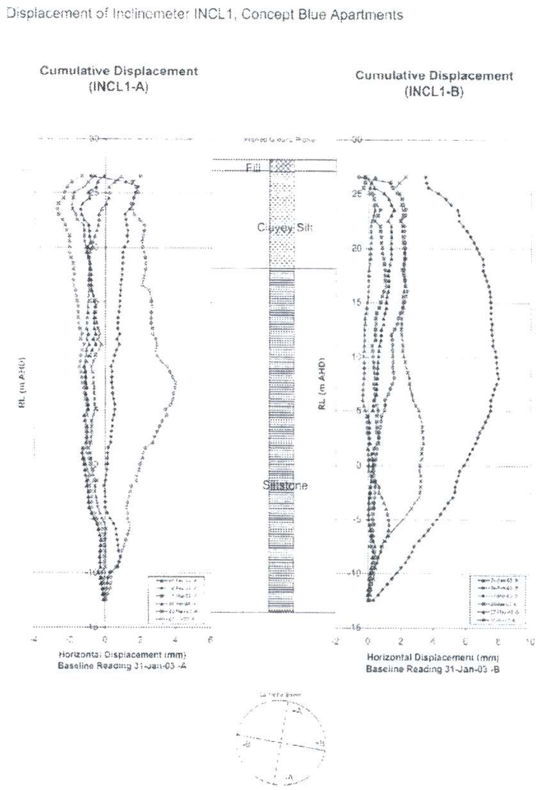

Figure 6 : Inclinometer measurements during excavation

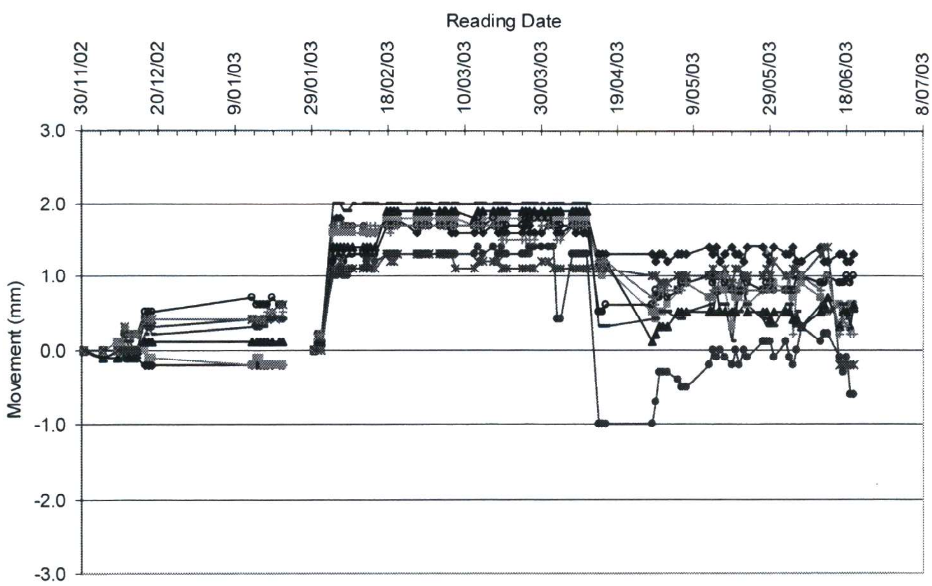

Figure 7 : Extensometer results during excavation