ASSESSMENT OF IMPACTS OF GROUND MOVEMENTS ON EXISTING STRUCTURES ADJACENT TO EXCAVATIONS

Jeff Hsi
SMEC Australia Pty. Ltd., Australia

ABSTRACT

A cut and cover tunnel is being constructed in soft marine clay as part of the 12 km long Kallang and Paya Lebar Expressway in Singapore. The width of the excavation ranges approximately between 40 m and 60 m and the maximum depth is up to 25 m. Such extensive excavation inevitably causes substantial ground movement in the surrounding areas. The structures identified to be affected by the excavation induced ground movements include the bored piles beneath the base of the excavation, several multi-storey buildings supported on deep foundations and a cluster of warehouses on shallow foundations. This paper presents the methodology and analysis techniques employed for the prediction of ground movements and assessment of impacts on the adjacent structures. The role of monitoring is discussed and selected field data is included.

1 INTRODUCTION

Contract 421 (C421) of the 12 km long Kallang and Paya Lebar Expressway (KPE) in Singapore involves the design and construction of a 1.5 km vehicular tunnel from East Coast Parkway (ECP) to Nicoll Highway. The key project features comprise a 4 lane dual carriageway within a tunnel box, a river crossing including cofferdams during construction, a ventilation building, the KPE/ECP interchange, etc. The project layout plan of KPE C421 is shown in Figure 1. The tunnel is constructed by the cut and cover method using sheet piles and internal bracing for support of the excavation. The bored piles installed below the excavation are designed for both the permanent loads from the tunnel structures as well as for temporary forces induced during construction. Tension piles are used to assist the tunnel structure against floating. Both the top-down and bottom-up construction methods have been adopted.

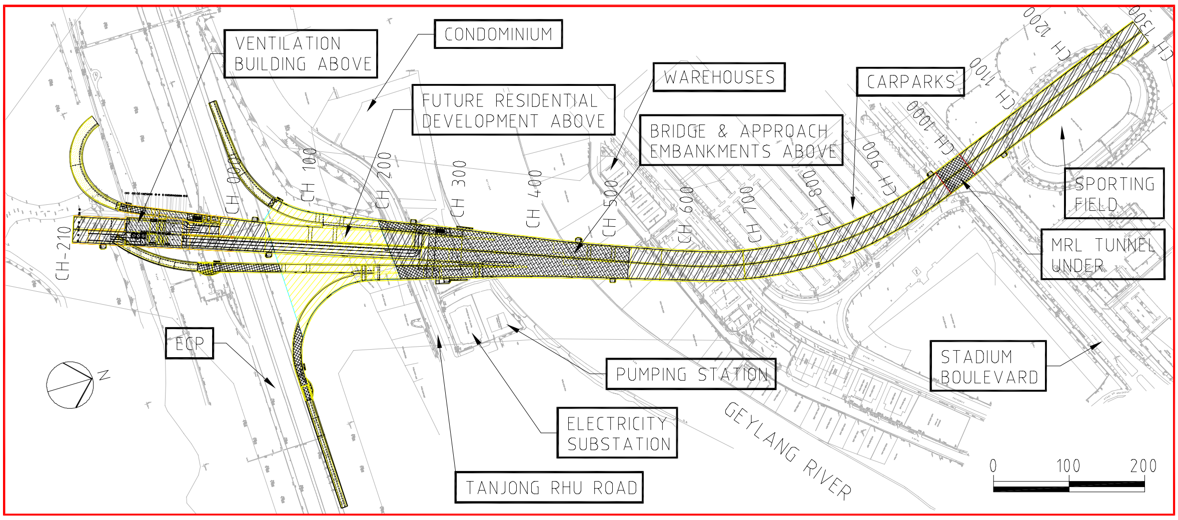


Figure 1: Project layout plan of KPE C421.

The project site is located on recently reclaimed land which is still settling. The fill is underlain by a series of soft marine clays and fluvial/alluvial deposits up to 50 m depth before reaching dense sands. Excavation is generally carried out in soft marine clays which presents particular challenges to the designer and the contractor for maintaining the stability of the excavation and limiting the ground movement in the surrounding area. The width of the excavation ranges approximately between 40 m and 60 m and the maximum depth is up to 25 m.

The tunnel generally traverses flat terrain in a greenfield site with several permanent structures all located within 50 m of the main excavation. These include a 20 storey Camelot condominium, a 5 storey electricity sub-station, and a single storey pumping station, all supported on pile foundations. There is also a cluster of warehouses supported on shallow foundations. Figure 2 shows the location of these structures in relation to the cut and cover KPE tunnel.

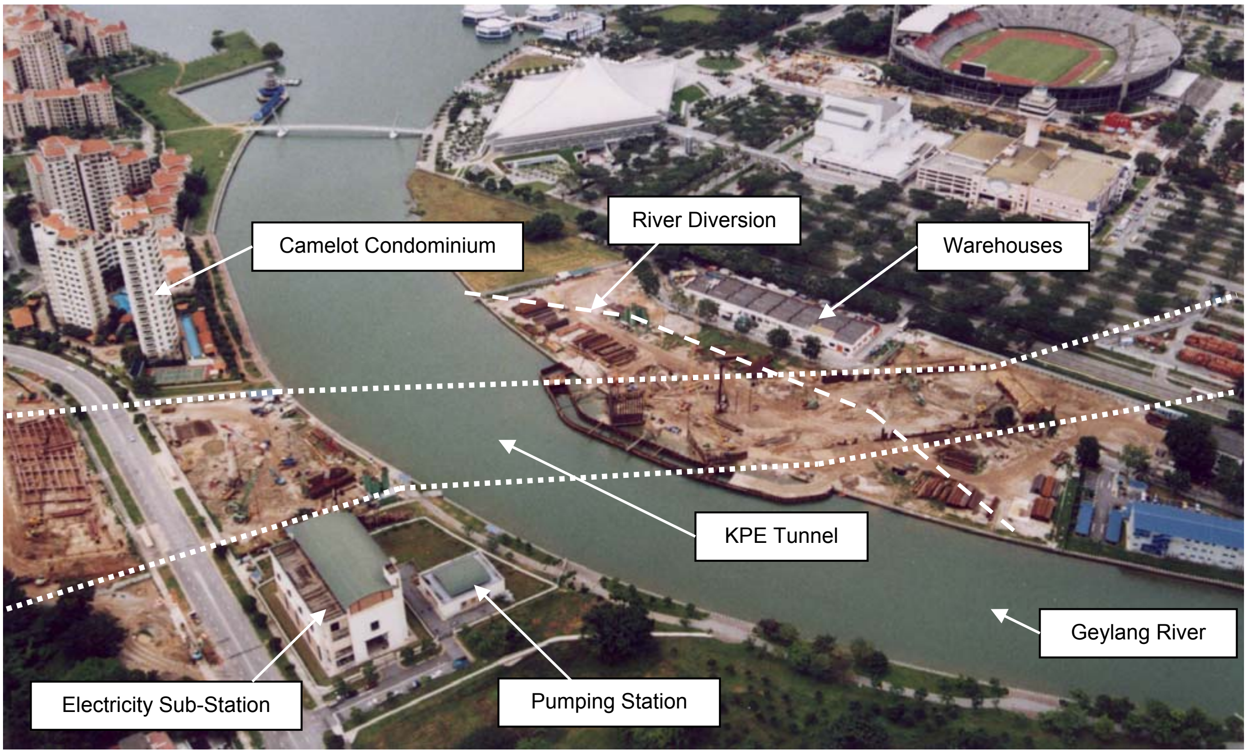


Figure 2: Aerial photo of structures adjacent to KPE C421 alignment.

Extensive numerical modelling has been used for the design of the temporary support systems, to ensure that the ground movements due to the excavation will not adversely impact the buildings adjacent to, and the bored piles below, the excavations. This paper focuses on the methods adopted for the prediction of the excavation induced ground movements and the assessment of impacts on the adjacent structures. The instrumentation and monitoring plan for critical structures are discussed and selected field monitoring results are presented.

2 SUBSURFACE CONDITIONS

The project site is located in an area where bedrock is inferred to be at a depth of greater than 70 m. The soils underlying the area have been divided into three main units comprising fill, overlying Kallang Formation, then overlying Old Alluvium. The measured groundwater levels are generally within 1 m to 1.5 m below the ground surface level.

Fill is present over the entire site except on the riverbed of the Geylang River. This fill is part of the reclamation works over the former low lying swampy land adjacent to the river. The material varies from sandy to clayey. The sandy fill is generally uncompacted with densities ranging from very loose to loose and medium dense. The clayey fill is generally soft to firm in consistency.

The Kallang Formation consists of sediments of fluvial or marine origin. The fluvial sediments can be sub-divided into cohesive and non-cohesive soils. The non-cohesive fluvial soils (F1) comprise sandy silts, silty sands and clayey sands. The F1 soils are loose to medium dense. The cohesive fluvial soils (F2) comprise sandy clays, silty clays and clayey silts. The F2 layer, which separates the two marine clays, is of high plasticity and is firm to stiff in consistency.

The two units of high plasticity marine clays have high compressibility, medium to high sensitivity and very low permeability. The upper marine clay (AuM) forms a near continuous layer along the tunnel route and ranges from 4 m to 18 m thick. The AuM is generally very soft to soft in consistency. The lower marine clay (ALM) forms several discontinuous layers ranging from 2 m to 18 m thick. The ALM is generally soft to firm in consistency. The field and laboratory test results indicate that the undrained shear strength (S_u) of the marine clays increases with depth. For design purposes S_u is taken to range between 10 kPa and 60 kPa. Due to the soft consistency, the marine clays have particular significance to the design and construction of the KPE tunnel. Typical material properties are shown in Figures 3, 4 and 5.

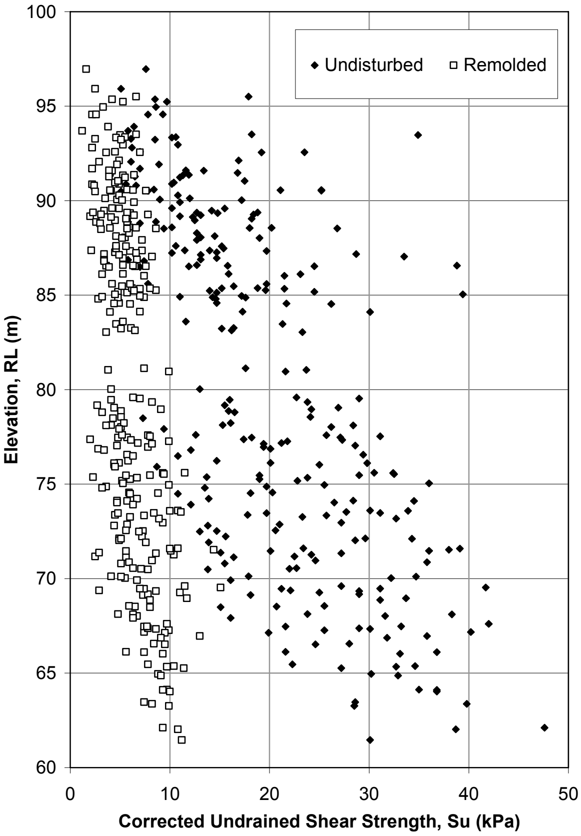


Figure 3: Undrained shear strength of marine clays from vane shear tests.

The Old Alluvium (OA) is a deposit of over-consolidated and cemented silty clays and silty sands, which range from medium dense or stiff, when weathered, to very dense or hard, where cemented. The depth of weathering is irregular varying from 0 to 21 m from the top of the unit. The OA layer is further sub-divided into a weathered unit OA-W1, two slightly weathered units OA-SW1 and OA-SW2 and the cemented unit OA-CZ.

The interpreted subsurface profile along the KPE C421 alignment is shown in Figure 6, and the geotechnical parameters adopted for the analysis are given in Table 1.

3 BORED PILES BENEATH EXCAVATION

3.1 GENERAL CONDITIONS

Permanent bored piles are installed under the base of the tunnel excavation to support the tunnel box, including overlying fill. The bored piles are typically 1 m diameter reinforced concrete piles, except at sections with heavier loads where 1.2 m diameter piles are used. In general, pile spacing of 3 m c/c or 6 m c/c is adopted to minimize the group effect. Piles in pure compression require reinforcement for the top 12 m only. Piles are installed prior to excavation.

3.2 DESIGN CONSIDERATIONS

The reduction of overburden pressure as excavation progresses results in heaving of soils at the base of the excavation which introduces tensile forces in the piles. Further, excavation causes lateral movement of the temporary sheet pile walls as well as lateral movement of soils below the excavation. The piles are subjected to additional lateral forces resulting from the lateral soil movement.

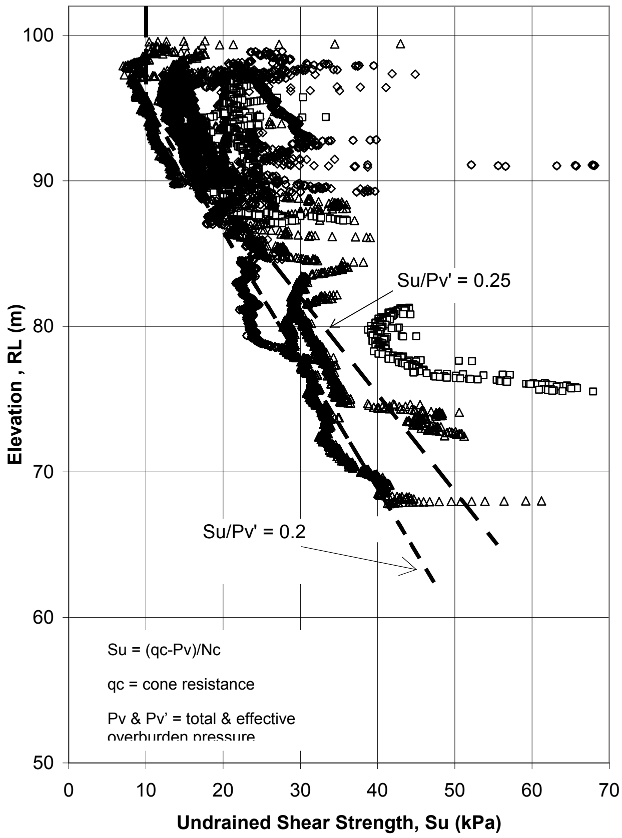


Figure 4: Undrained shear strength of marine clays from piezocone tests.

Table 1. Geotechnical design parameters.

Soil Unit	γ_t kN/m^3	Undrained Condition		Drained Condition			K_0	k cm/s
		S_u kPa	E_u kPa	c' kPa	ϕ' deg	E' kPa		
Fill (Clayey)	19	40	20,000	0	25	17,000	0.5	10^{-5}
Fill (Sandy)	19	-	-	0	30	10,000	0.5	10^{-4}
F1	19	-	-	0	30	10,000	0.7	10^{-3}
AuM	16	z≤6.7m, 10 6.7m<z≤40m, 10-60	z≤6.7m, 3000 6.7m<z≤40m, 3000-18,000	0	22	z≤6.7m, 2600 6.7m<z≤40m, 2600-15,600	1.0	10^{-7}
F2	19	50	15,000	0	26	13,000	1.0	10^{-6}
ALM	16	z≤6.7m, 10 6.7m<z≤40m, 10-50	z≤6.7m, 3000 6.7m<z≤40m, 3000-15,000	0	22	z≤6.7m, 2600 6.7m<z≤40m, 2600-13,000	1.0	10^{-7}
OA-W1	19	100	30,000	10	30	26,000	1.0	10^{-5}
OA-SW1	20	200	60,000	20	32	52,000	1.0	10^{-5}
OA-SW2	20	400	120,000	25	34	104,000	1.0	10^{-6}
OA-CZ	20	600	180,000	35	35	156,000	1.0	10^{-6}

where, γ_t = total unit weight of soil; S_u = undrained shear strength; E_u = undrained Young's modulus; c' = effective cohesion; ϕ' = effective friction angle; E' = drained Young's modulus; K_0 = at rest earth pressure coefficient; k= permeability; and z = depth below ground surface.

The possible additional lateral and axial tensile loads on piles as a result of unloading in the soil and the movements of soil during excavation are considered in the design.

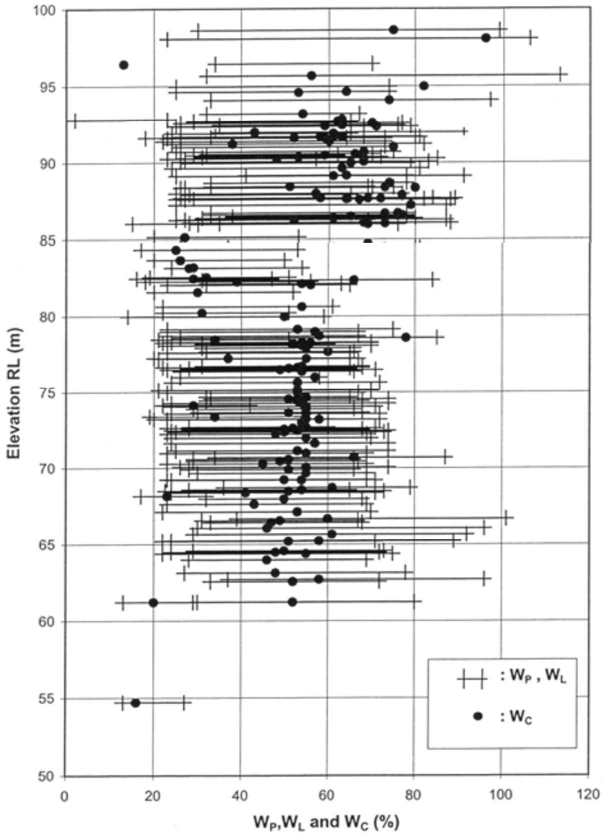


Figure 5: Plastic limit W_P, liquid limit W_L and moisture content W_C of marine clays.

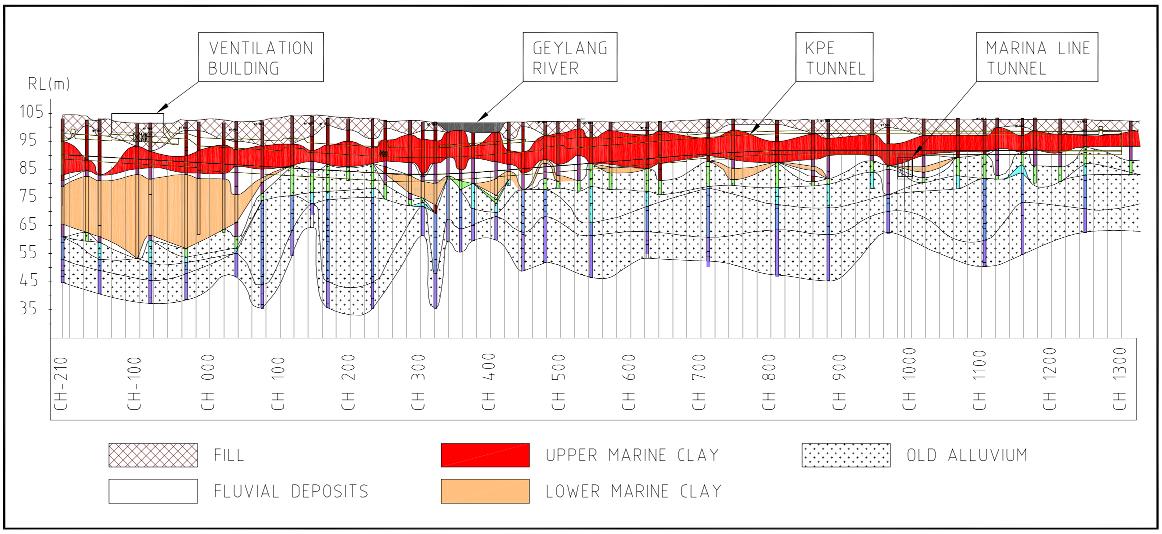


Figure 6: Interpreted subsurface profile along KPE C421.

3.3 ASSESSMENT METHODOLOGY

The methodologies suggested by Hull (1998), and Poulos (1989) are adopted for the assessment of bending moments and tension forces in piles associated with excavations.

A two dimensional finite element analysis program, PLAXIS (Version 7, 1998), is employed to calculate the ground movements without the presence of piles, i.e., the "greenfield" ground movements. PLAXIS considers the soil-structure interaction, the time dependent behaviour, the pore pressure response, the construction sequence and the appropriate constitutive soil model. The generation and dissipation of excess pore water pressure around the boundary of the excavation is simulated by PLAXIS in the form of consolidation using effective stress parameters and Mohr-Coulomb soil model. The rate of pore pressure dissipation is controlled by the permeability of the soil. Figure 7 is an output from the PLAXIS analysis for an excavation section at Ch +180 showing the displacement vectors when excavation reached its final stage. Note that the vertical lines below the excavation indicate the locations of the bored piles.

Figure 7 shows that the piles located under the centre of the excavation are mainly subjected to vertical soil movements whereas the piles located close to the edge of excavation are subjected to both vertical and lateral movements. The lateral soil movements are introduced in the program PALLAS (Hull, 1998) to calculate the induced bending moments in the piles. The vertical movements are entered in the program PIES (Poulos, 1989) to derive tensile forces in the piles. Both PALLAS and PIES allow soil to move past the piles, i.e. the soil and pile do not need to have identical movements.

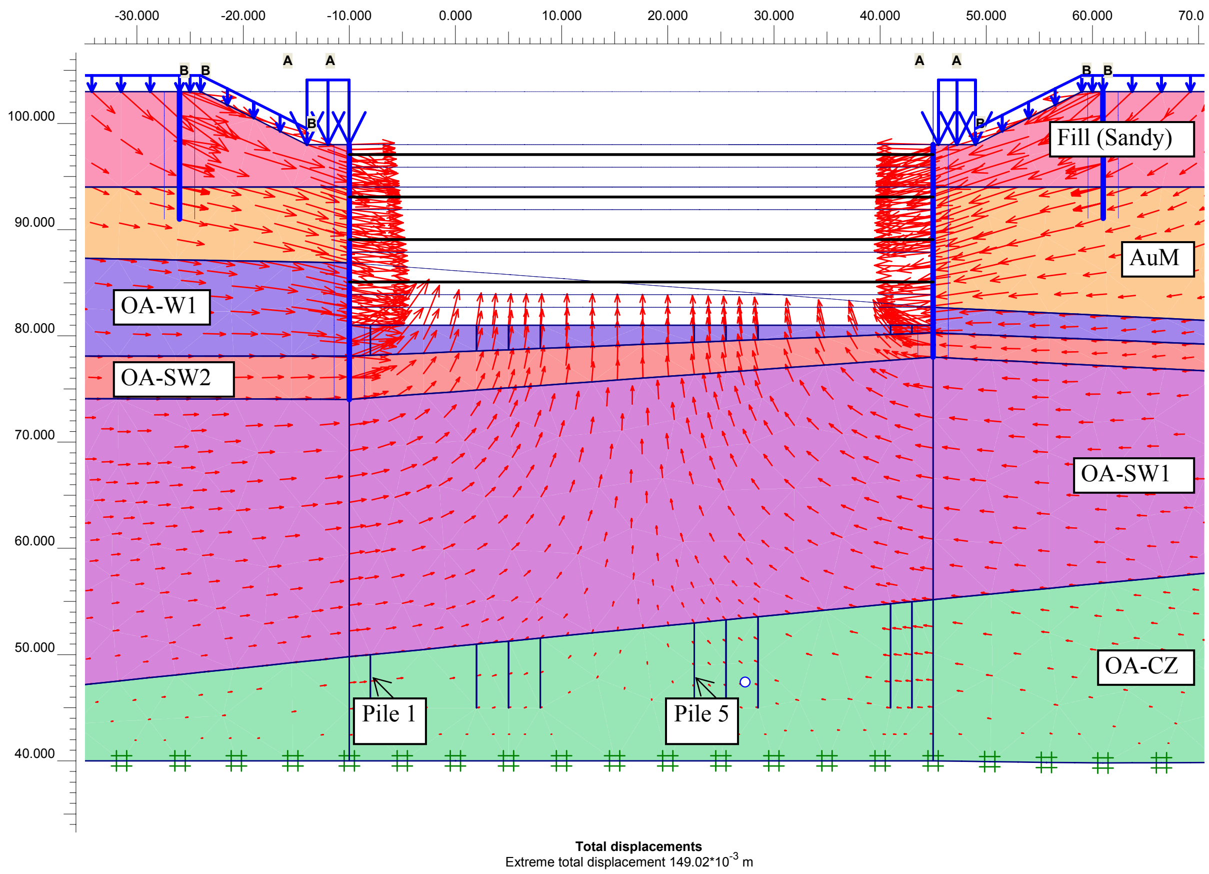


Figure 7: PLAXIS output at Ch +180 showing displacement vectors.

3.4 RESULTS OF ANALYSIS

The response of an edge pile at Ch +180 (Pile 1 on Figure 7) to lateral soil movement and the resulting bending moment induced in the pile are shown in Figure 8. The response of a central pile at Ch +180 (Pile 5 on Figure 7) to vertical soil movement and the resulting axial force induced in the pile are shown in Figure 9. These bending moments and axial forces are incorporated into the design of the bored piles. In Figure 9, the uniform pile has reinforcement extending to the toe of the pile, whereas for the non-uniform pile, reinforcement extends to the depth where axial stress is less than 1 MPa (tensile capacity of plain concrete), thus leading to a more optimized design.

4 BUILDING ON DEEP FOUNDATIONS

4.1 GENERAL CONDITIONS

There are three buildings located within 50 m of the tunnel excavation, which are supported on piles. The buildings include a 20 storey Camelot condominium, a 5 storey electricity sub-station, and a single storey pumping station. A similar methodology was adopted to assess the potential impacts the excavation may have on the foundation piles. In this paper, the assessment details and results for the most critical building, the 20 storey condominium, are discussed.

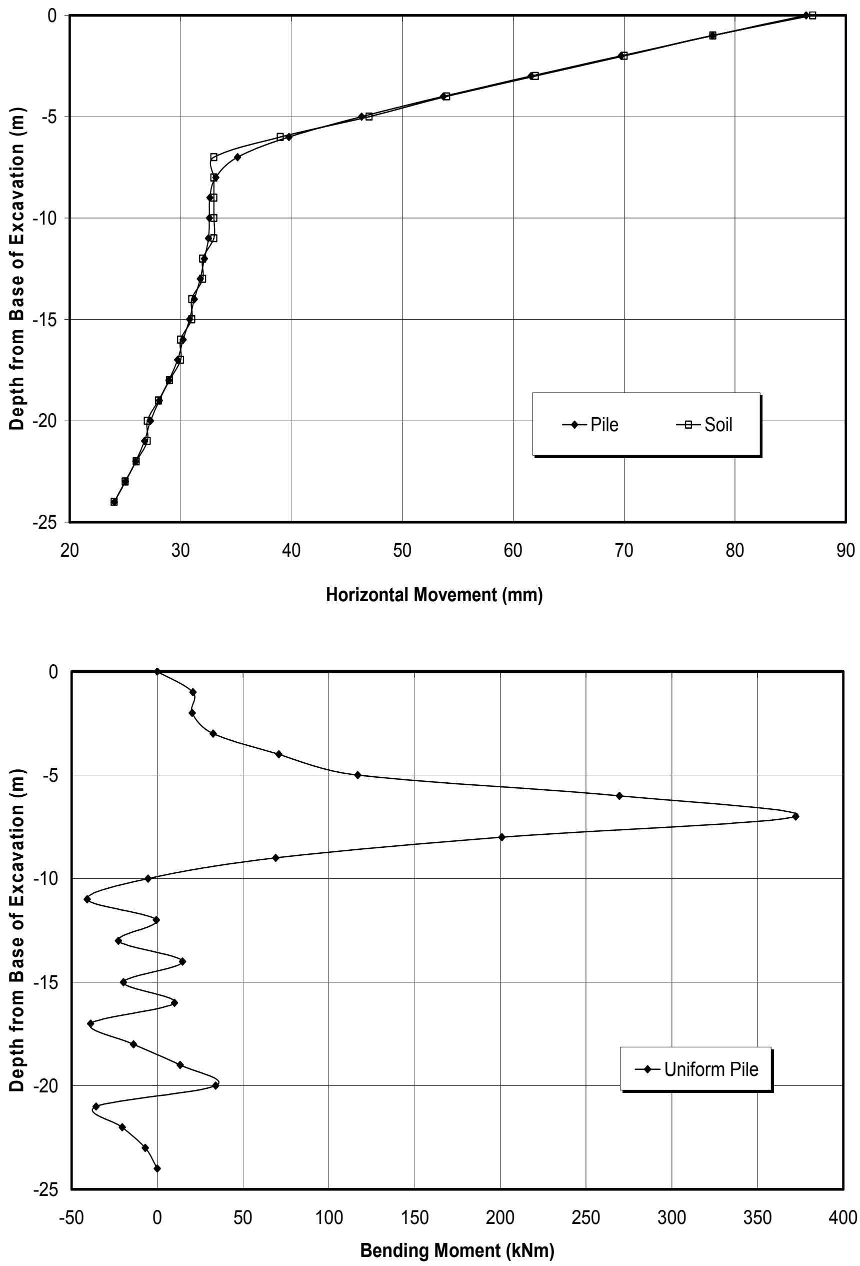
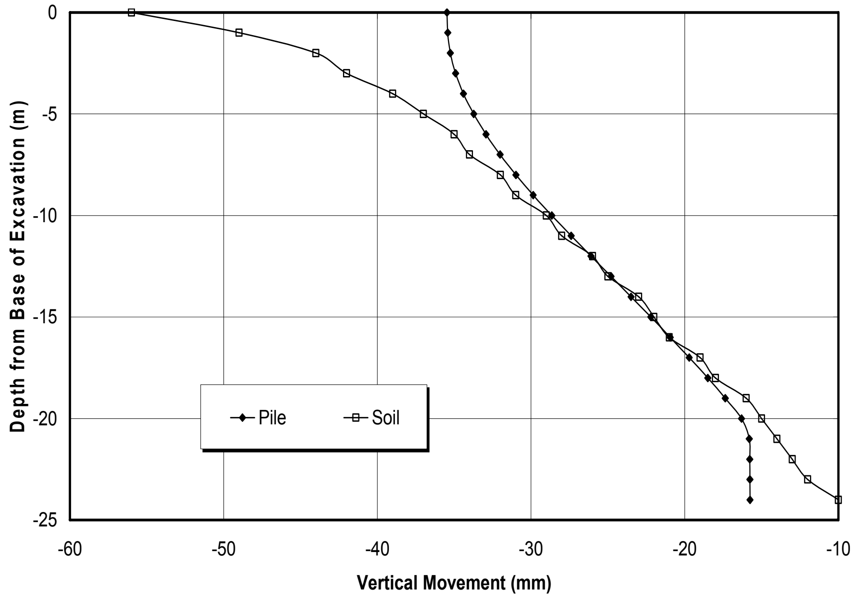


Figure 8: Response of an edge pile (1m dia.) to lateral soil movement.



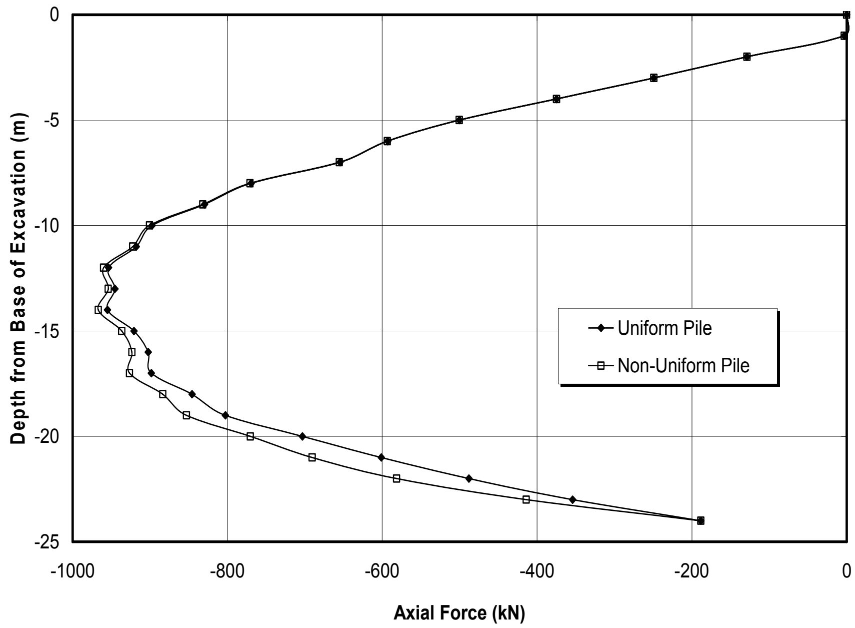


Figure 9: Response of a central pile (1m dia.) to vertical soil movement.

Camelot condominium is a 5 years old residential property, comprising a block of 20 storey flats and a block of flats from 3 storeys to 15 storeys complete with a club house, a swimming pool and a tennis court at the location as shown in Figure 2. The minimum distance of the building to the tunnel alignment is 30 m. The perimeter wall/fencing of Camelot condominium in relation to the temporary works is shown on the instrumentation and monitoring plan in Figure 10. The excavated area here is up to 84 m wide and 22.5 m deep.

The building structure is founded on driven precast pre-tensioned spun concrete piles with pointed pile shoes. The pile size consists of 450 mm, 500 mm, 600 mm diameter. The recorded pile length is 35 m from the pile cut off level. Based on the longitudinal geotechnical section, it is considered that the piles are founded in the OA stratum. An extensive 250 mm thick reinforced concrete slab constitutes the basement floor, which connects the buildings and extends under the tennis court. The near side of the tennis court is 31 m from the KPE excavation. The 20 storey building is located on the far side of the tennis court, approximately 35 m away from the excavation. The piles selected for damage assessment are located under the tennis court due to their close proximity to the excavation and sensitivity to ground movement.

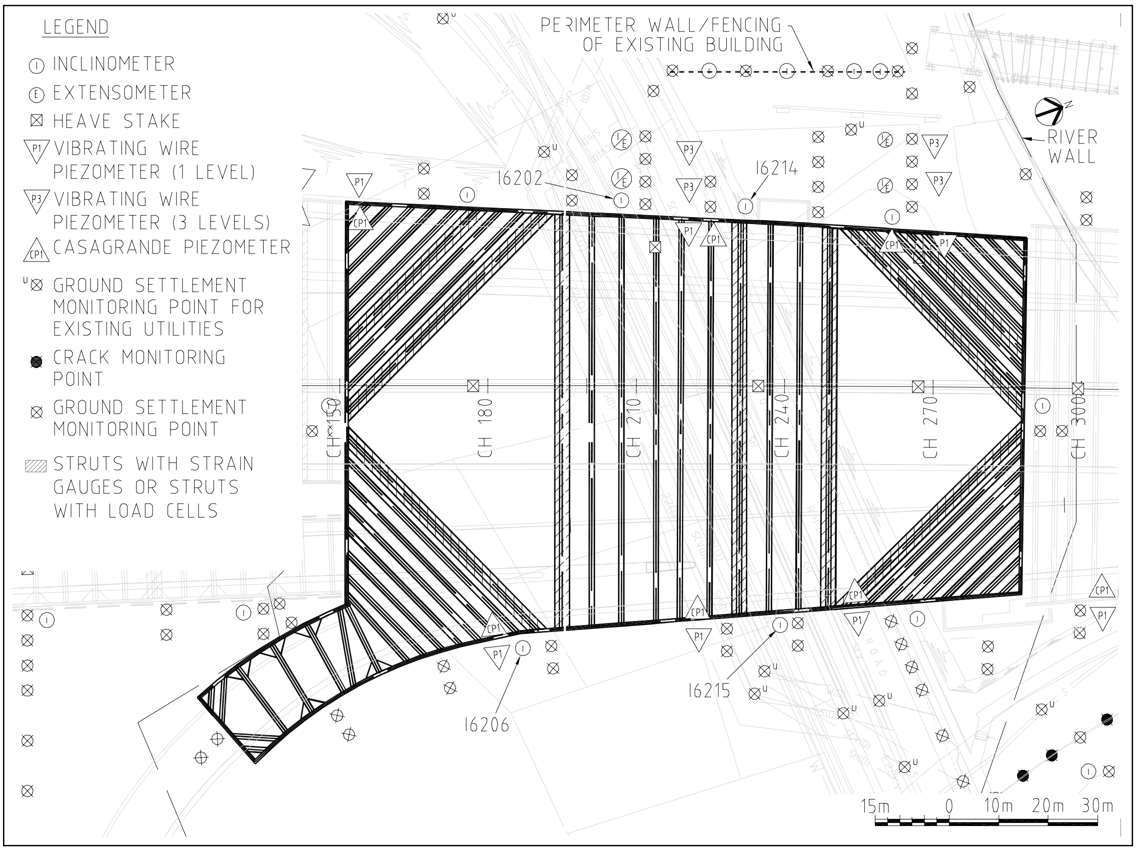


Figure 10: Instrumentation and monitoring plan of KPE C421 alignment adjacent to Camelot.

The excavation for the KPE tunnel involves a 5 m pre-cut prior to sheet piling. This pre-cut causes some horizontal ground movement before any bracing is installed to the sheet pile walls. The design is developed to control any further horizontal movement within the total limits for the prevention of damage to the Camelot building piles.

4.2 DESIGN CONSIDERATIONS

Due to the compressible and low strength nature of the underlying soils, most structures along the project alignment are founded on piles. As the project site is located in a reclaimed area where the whole area is undergoing consolidation settlement, the pile design for the existing structures is expected to have allowed for additional loading from negative skin friction (NSF). The excavation works will only speed up the settlement process and will not add additional vertical loads onto the piles of existing structures.

However, excavation would also result in lateral soil movements which would induce additional bending moments on the piles, compromising the pile performance. The impacts of lateral soil movement on the piles are considered in design of the temporary sheetpile walls.

Another important consideration is the plastic flow of the Marine Clay "squeezing" between the piles, i.e. horizontal ground movement can occur without this translating to a similar degree of movement of the piles. Lateral loads applied to the piles due to lateral soil movement will be distributed over the length of the whole pile group beneath the buildings.

4.3 ASSESSMENT METHODOLOGY

The horizontal ground movement at pile locations caused by the adjacent excavation is estimated using the program PLAXIS. The induced bending moments at pile locations are calculated using the program PALLAS. The induced bending moment is compared with the flexural capacity of the pile to determine the possibility of pile damage during excavation. This approach captures the complexity of the subsurface soil-structure interactions and is similar to that described in Section 3 above.

4.4 RESULTS OF ANALYSIS

PLAXIS modelling reveals that lateral soil movements of approximately 30-50 mm around the basement are expected at the position of piles closest to, and located at approximately 35 m away from, the excavation. This is indicated in Figure 11, in which green field horizontal movements are shown. The basement is considered to be fully restrained from horizontal and vertical movement. The associated bending moment induced by this lateral soil movement does not exceed the pile capacity based on the results of PALLAS and will not adversely affect the performance of the piles. It is further assessed that the pile is able to tolerate a lateral soil movement up to 100 mm without its structural capacity being exceeded.

An observational approach has also been implemented based on site monitoring data to assess the performance of tunnel excavation. The support system for the tunnel excavation is designed so that the resulting ground movements will not induce bending moments in the pile that exceed the pile capacity. Inclinometers are installed along the perimeter wall of Camelot (see Figure 10) and the measured movements for each pre-excavation stage are reviewed before excavation to the next level can proceed. Should the measured values exceed the pile tolerance and indicate future movements could damage the piles, a contingency system would be implemented to prevent further ground movements from impacting the piles.

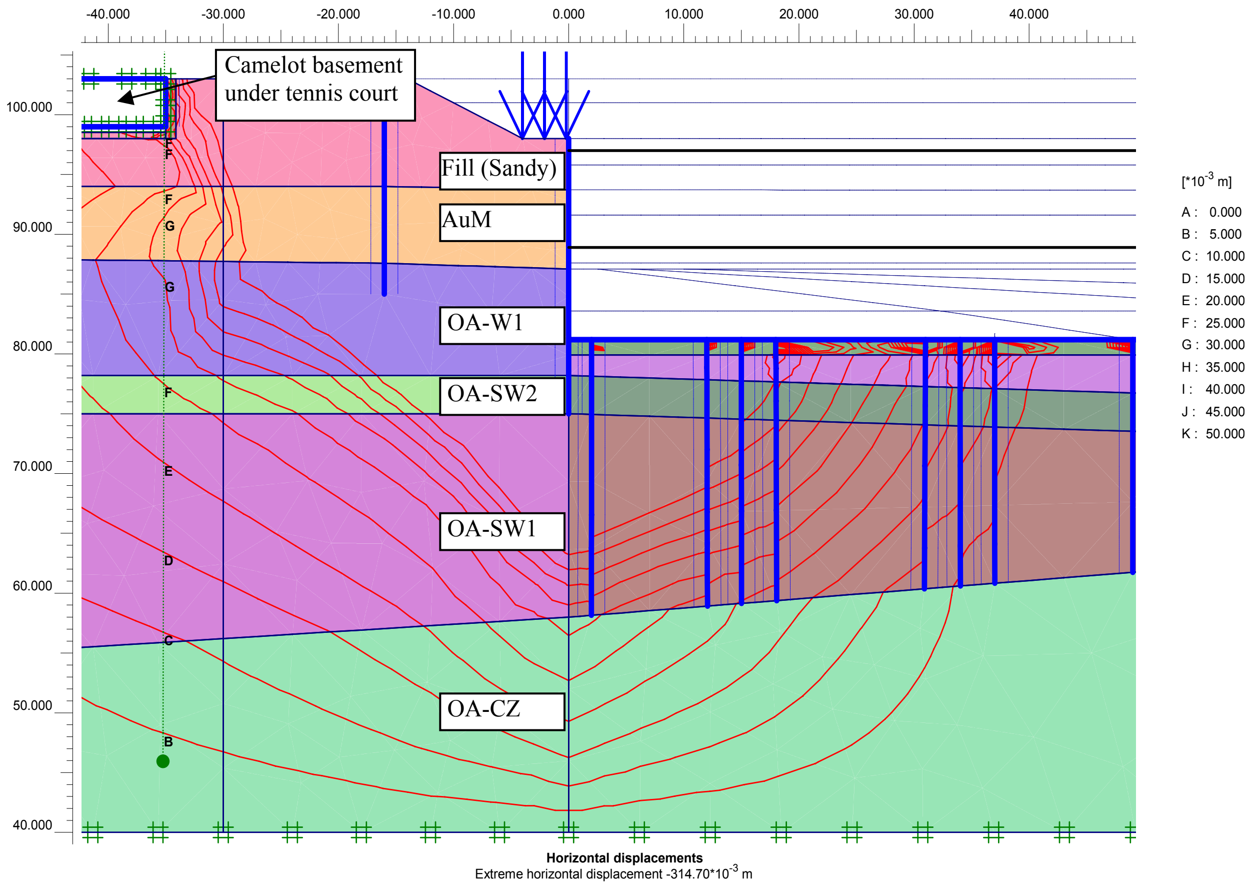


Figure 11: Horizontal soil movements at Ch +230.

4.5 PERFORMANCE

Four inclinometers have been installed at the perimeter of the Camelot condominium and have measured lateral ground movements up to 80 mm resulting from the tunnel excavation, as shown in Figure 12. These movements are within the structural capacity of the piles and are considered acceptable.

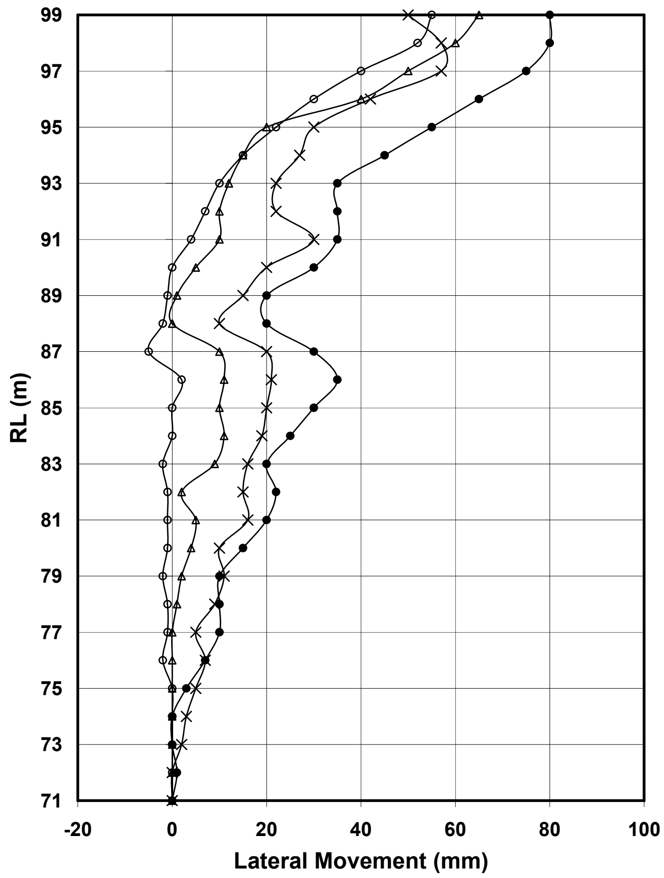


Figure 12: Measured lateral movements at perimeter wall after last excavation stage.

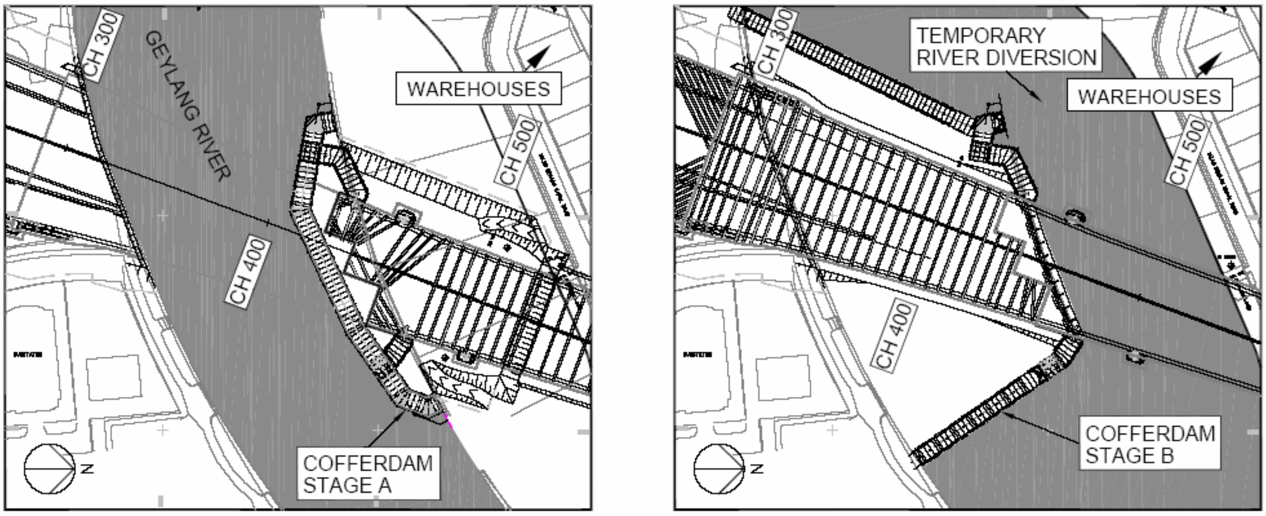
5 WAREHOUSES ON SHALLOW FOUNDATIONS

5.1 GENERAL CONDITIONS

Warehouses supported on shallow foundations are located to the north of the Geylang River as shown in Figures 1 and 2. In addition to the excavation works for the main tunnel at a distance of approximately 22 m away from the warehouses, a temporary river (Stage B) is to be excavated approximately 13 m in front of the warehouses as shown in Figure 13. This diversion of the Geylang River is required for the construction of the KPE section from Ch +280 to Ch +430.

5.2 DESIGN CONSIDERATIONS

It is envisaged that ground movements associated with excavation works carried out at such close proximity to the warehouses may induce damaging tensile strains in the structure. Analysis is carried out to determine the potential damage to the warehouses as a result of the ground movements caused by the adjacent excavation works.



(a) Stage A (1st stage) (b) Stage B (2nd stage)

Figure 13: River crossing works.

5.3 ASSESSMENT METHODOLOGY

The vertical and horizontal ground displacements due to excavation are predicted using PLAXIS. Vertical and horizontal displacements are calculated for the main tunnel and temporary river excavations. The displacements are superimposed to obtain the vertical and horizontal ground movements due to both excavations.

The methodology used to assess the category of potential damage to the warehouses is based on limiting tensile strains (Burland *et al.*, 1977). The building is treated as an idealized beam and assumed to have no stiffness. The strains in the building due to the predicted relative settlements are calculated over its hogging and/or sagging spans.

The horizontal ground strain is combined with the bending and diagonal strains to give the total tensile strains using superposition suggested by Boscardin and Cording (1989).

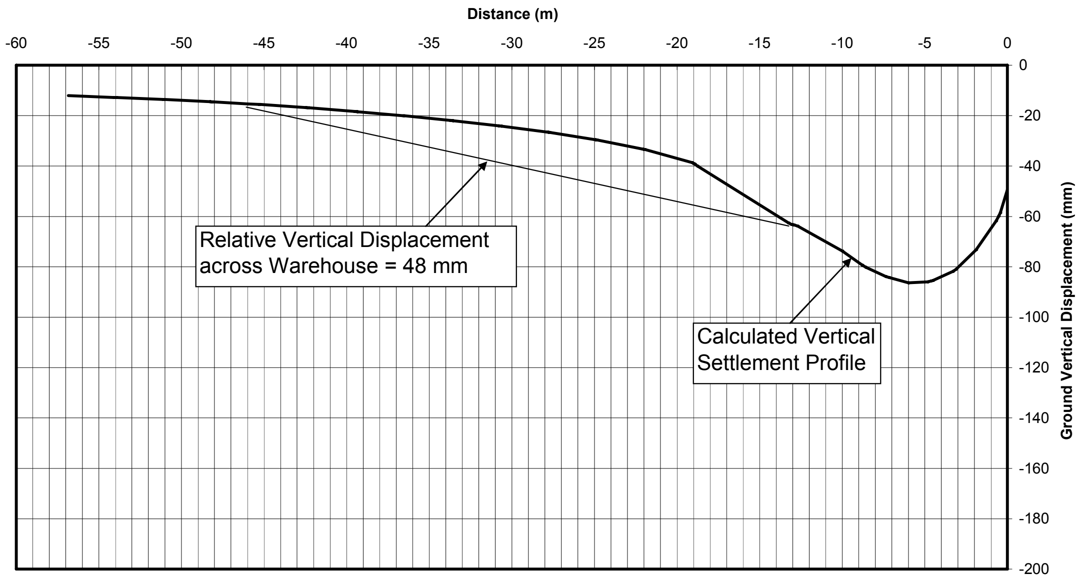
The maximum tensile strains calculated are used to assess the potential damage to the building according to the relationship between tensile strains and categories of damage developed by Burland and Wroth (1974), Boscardin and Cording (1989) and Mair *et al*. (1996).

5.4 RESULTS OF ANALYSIS AND PERFORMANCE

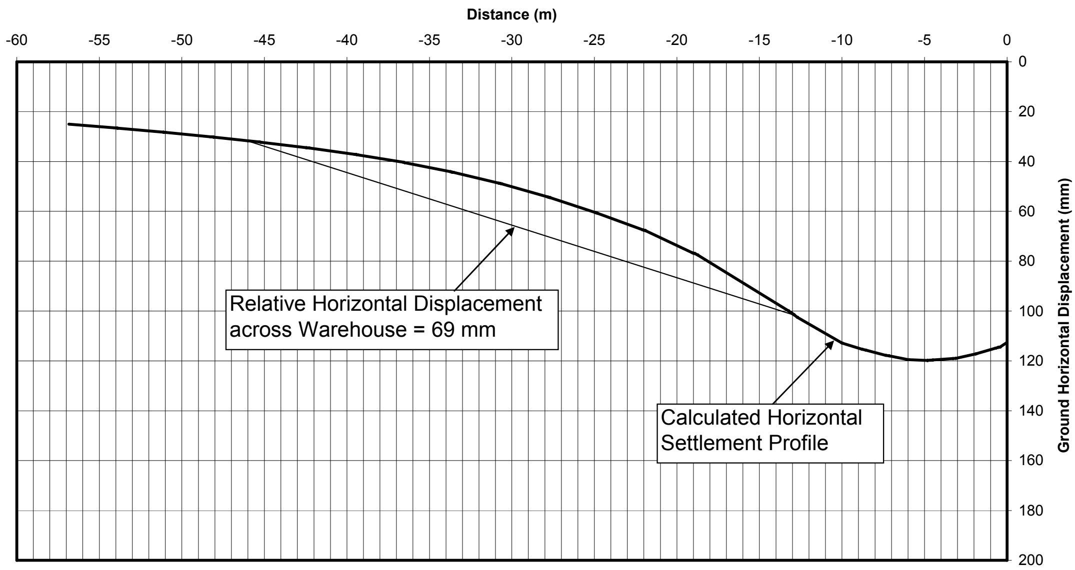
The calculated vertical and horizontal displacements below the warehouse from PLAXIS are presented in Figure 14. Based on the methods described above, the damage category to the warehouses due to excavation works is assessed to be severe.

Due to the proximity of the warehouses to the excavations and the magnitude of predicted ground movements, it is considered uneconomical and impractical to minimize ground movements as a means to reduce the damage category to the warehouses. In reality the inherent stiffness of the warehouse will reduce the strains and the resulting damage of category will be less severe. A site construction procedure is developed that maintains the structural integrity and serviceability of the warehouse, during and following the occurrence of ground movement.

The procedure involves close monitoring of the amount of ground movement and structural movement of the warehouse and close inspection of the warehouse structure for evidence of any unacceptable loss of strength and serviceability. If such loss of strength and serviceability occurs, prompt remedial action is taken to recover this loss. Upon completion of the river diversion and excavation works, no severe distress of the building structure has been observed.



(a) Vertical displacement



(b) Horizontal displacement

Figure 14: Calculated vertical and horizontal displacement of warehouse foundation.

6 CONCLUSIONS

It is prudent to properly predict the likely ground movements associated with large scale excavations in order to adequately assess the resulting impacts on the adjacent structures. The additional vertical loading and bending moment induced in the bored piles constructed below the tunnel due to excavation are considered during the design. An observational approach comprising prediction and monitoring of ground movements is adopted to evaluate the performance of foundation piles of a high rise building located close to a deep excavation. The presented building damage assessment procedure for warehouses sitting on shallow foundations follows well established practice.

7 REFERENCES

Boscardin, M.D., and Cording, E.G. (1989). Building Response to Excavation Induced Settlement. Journal of Geotechnical Engineering. ASCE, Volume 115, pp.1-21

Burland, J.B., Broms, B.B., and De Mello, V.F.B. (1977). Behaviour of Foundations and Structures. State of the Art Report, Session 2, Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo.

Burland, J.B., and Wroth, C.P. (1974). Settlement of Buildings and Associated Damage. State of the Art Review. Conference on Settlement of Structures, Cambridge, Pentech Press, London.

Hull, T.S. (1998). Piles And Lateral Loading AnalysiS (PALLAS). Center for Geotechnical Research, Department of Civil Engineering, University of Sydney.

Mair, R.J., Taylor, R.N. and Burland, J.B. (1996). Prediction of Ground Movements and Assessment of Risk of Building Damage due to Bored Tunnelling. Geotechnical Aspects of Underground Construction in Soft Ground, Mair and Taylor (eds), 1996 Balkema, Rotterdam.

PLAXIS, Version 7, (1998). Finite Element Code for Soil and Rock Analysis, Brinkgreve and Vermeer (eds.), A.A. Balkema/Rotterdam/Brookfield.

Poulos, H.G. (1989). PIES. Centre for Geotechnical Research, Department of Civil Engineering, University of Sydney.

Poulos, H.G. (1998). The Effects of Ground Movements on Pile Foundations. Prof. R. Katzenbach and Prof U. Arslan (eds.), International Conference on Soil-Structure Interaction in Urban Civil Engineering 8-9 October 1998. Darmstadt Geotechnics No. 4 Vol 2. Darmstadt University of Technology, Germany.