Pile behaviour due to adjacent tunnel excavation

C. F. Leung[1], A. L. Saw[1], R. F. Shen[1], Y. K. Chow[1], Y. S. Ng[2], H. T. Tan[2] and C. J. Hua[2]

[1]Centre for Soft Ground Engineering, National University of Singapore, Singapore 117676; PH (65) 6516 2281; email: ceelcf@nus.edu.sg
[2]Building & Infrastructure Group, Housing and Development Board, Singapore 310480; PH (65) 6490 2546; email: Hcj1@hdb.gov.sg

ABSTRACT

With ever increasing population in many urban cities, the construction of underground subways is often conducted close to existing buildings. Several subway lines are currently under construction in Singapore and tunnel excavation has been carried out very close to an existing building. This paper presents a case study with tunnel excavation conducted beneath the base of relatively short piles. The horizontal distance between the tunnel perimeter and the closest pile is only about 1 m. Before construction, three-dimensional finite element analysis was conducted to evaluate the surface and subsurface ground movements as well as the axial and lateral responses of the piled foundation. The latter includes the pile settlement and deflection as well as the bending moment profile along the pile and the axial load transfer along the pile shaft comprising the loss of pile end bearing resistance due to tunnel excavation. Scenarios including large ground volume loss due to potential problems in tunnelling are considered in the analysis as well. Extensive field instrument program was initiated to monitor the ground and building movements before, during and after tunnel excavation. The instrument results are presented and compared with predictions from finite element analysis. As the ground volume loss has been well controlled during tunnel excavation, the movement and tilt of the building is well within the desired limits and comparable with the numerical predictions. This illustrates the importance of ground volume loss control for tunnel excavation to minimise its damage to adjacent foundation. Details of the numerical and field studies are presented in this paper.

Keywords: 3-D finite element analysis, tunnel excavation, volume loss, piled foundation, building movement

1 INTRODUCTION

1.1 Background

Singapore is in the midst of expanding its rail network to support the transit operation among the community. In some area, the proposed tunnel alignments have no choice but to be built in close proximity to existing structures. According to Burland (1997), tunnel excavation inevitably causes ground movement. In other words, this construction may pose damage to existing structure if not handled properly. In view of this, preliminary finite element analysis (FEA) was conducted prior to construction to evaluate the ground and piled foundation responses due to tunnel excavation. The preliminary prediction provides monitoring guidance for the site activity and serves as primary safety barrier. The site monitoring is supported by a comprehensive field instrument program to observe the ground and structure responses before, during and after tunnel excavation. Nevertheless, it is almost impossible to instrument the existing piles to monitor the performance subjected to adjacent tunnel excavation. Therefore, some (e.g. Pang, 2006) carry out finite element back-analyses based on the measured ground movements to predict the pile behaviour subjected to tunnelling process.

One of the tunnel constructions in the eastern part of Singapore is studied herein. Extensive soil investigation work was conducted to establish the in-situ soil profile and design parameters. The site is sitting on the Old Alluvium (OA) formation of Singapore with a thin layer of peaty clay in the range of 1.3 m to 6 m observed in a few boreholes. Two tunnels, named as West Bound (WB) and East Bound (EB), have passed beneath an existing residential building with EB moving ahead of WB. Both tunnels are approximately 6.3 m in diameter and separated by 11.5 m (center to center) with their crown levels located at 31.1 m below ground surface. The piled foundation configuration of this existing building was derived from the as-built drawing. The cast in-situ concrete bored piles toes are located approximately 7 m above the tunnel crown and varies from 600 mm to 900 mm in diameter. Figure 1 shows the location of piles with respect to the two tunnels.

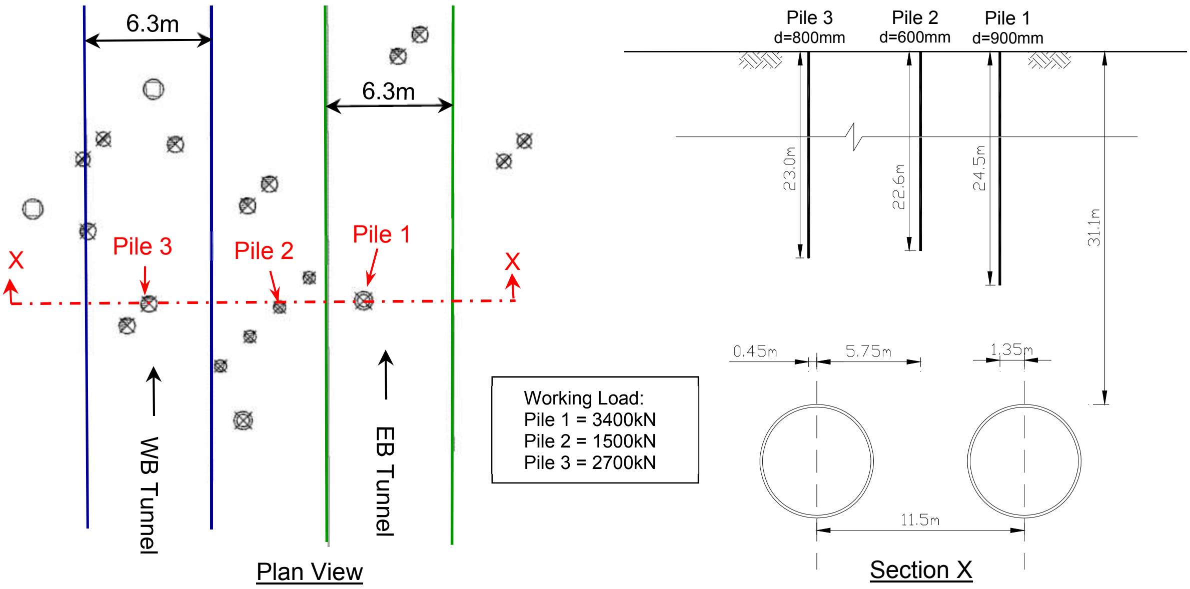


Figure 1. Location of piles with respect to tunnels

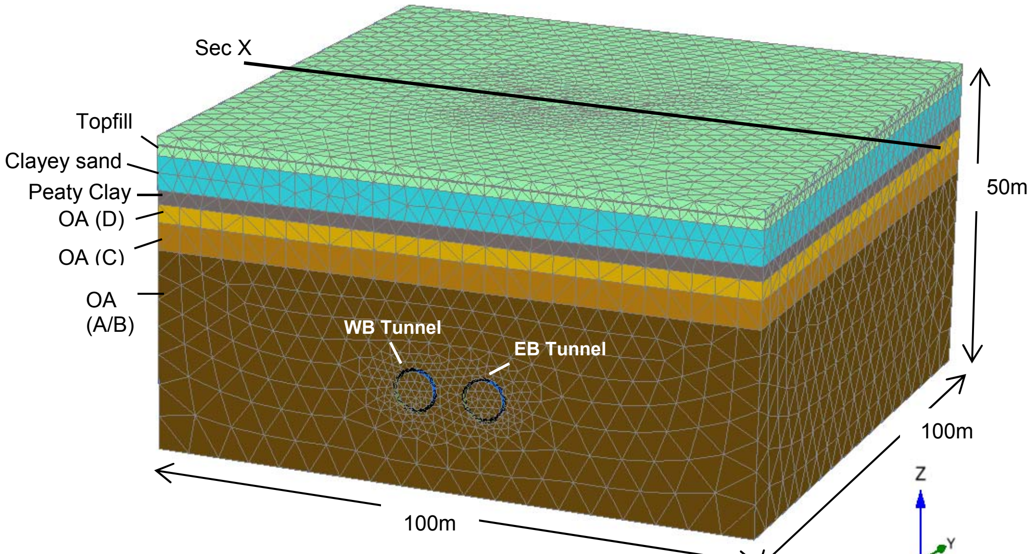
2 PRELIMINARY FINITE ELEMENT ANALYSIS

Prior to tunnel construction, three-dimensional FEA was conducted to evaluate the ground and pile responses due to tunnelling. Past local experiences for tunnel excavation in this type of soil had reported volume losses $\leq$ 1% as documented by Shirlaw et al. (2003) and Zhang et al. (2011) for North East Line and Circle Line, respectively. However, OA contains confined aquifer with water under sub-artesian pressure which can become flowing ground soon after exposure (Knight et al., 2001). As such, larger ground volume loss = 2% was considered in the analysis as well.

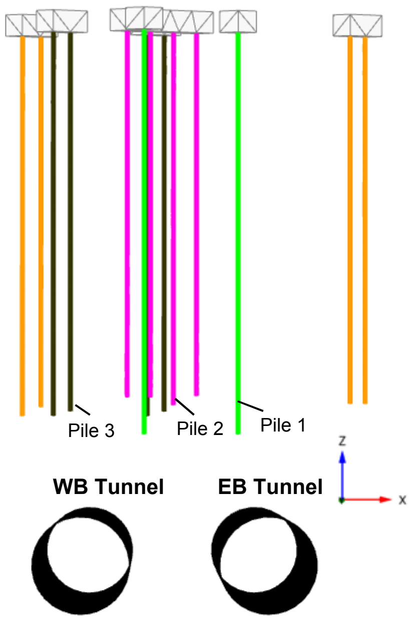
2.1 Model Set-up

The finite element analysis was carried out using Plaxis 3-D with quadratic tetrahedral 10-node elements. The boundary for the finite element model was 100 m wide and 100 m long in order to keep 50 m away from the interest section in both transverse and longitudinal directions. As shown in Figure 2, a total of 138,241 elements was used to build up this 3-D model. Figure 1 illustrates the locations of tunnels and piles. In this study, three piles named as Piles 1, 2 and 3 were selected for the tunnel-foundation interaction analysis.

The simulation of the tunnel excavation was simplified to one single step, i.e. remove soil element, activate tunnel lining and apply contraction for the whole tunnel length. The tunnel volume loss was simulated using the gap method proposed by Lee and Rowe (1991). Since there is no face pressure data in this preliminary stage, it is too crude to model various volume loss components such as face loss, shield loss and tail loss. Nonetheless, as the tunnelling effect on adjacent piles encompasses all components of volume loss, they were simplified and modelled as a single total volume loss component in the present study. The modelling of this single component was done by applying a uniform contraction with tunnel tip fixed throughout the length of tunnel.



Overall 3-D view



Front view for piles and tunnels in 3-D model

Figure 2. 3-D finite element model

2.2 Material Models

The soil behaviour in this study was described by Mohr Coulomb (MC) model. The tunnel excavation process was modelled as undrained condition since most of the ground response occurred immediately within a short period of time during the passing of the tunnelling boring machine (Cham, 2009). The soil yielding criteria is described using effective soil parameters as required by the Plaxis3D program and given in Table 1. As the mobilized stress would not reach the peak state in this small strain problem, the adoption of effective parameters is deemed acceptable.

Table 1: FEA Input Parameters for Soils

Soil	Top fill	Clayey sand	Peaty clay	OA (D)	OA (C)	OA (A/B)
SPT-N	-	-	-	10<N<30	30<N<50	> 50
γ_{unsat} (kN/m^3)	17	16	13	17	18	18
γ_{sat} (kN/m^3)	20	18	16	19	20	20
E' (MPa)	10.0	8.0	5.0	30.0	70.0	100.0
ν	0.3	0.3	0.3	0.3	0.3	0.3
c' (kPa)	5	1	0	10	15	20
ϕ' (^{o})	28	30	22	32	34	36

R_{inter} = 0.8

The tunnel lining was modelled with plate element with following input parameters: thickness = 0.3 m, γ = 24 kN/m^3, ν = 0.25, and E = 30 GPa. For this design stage, the crack conditions for various concrete elements have not been considered. On the other hand, the existing piled foundations were modelled as embedded pile elements. An embedded pile is made of beam elements embedded with interface elements to describe the interaction between the soil and pile along the shaft as well as at the toe. Each pile group was tied to a 1.5 m thick concrete pile cap. The corresponded working load as shown in Figure 1 was applied as surcharge on top of the pile cap. The FEA consists of 4 simulation stages as follows,

1. Initial ground condition.
2. Activate piles, pile caps and surcharge on pile caps.
3. EB tunnel excavation.
4. WB tunnel excavation.

2.3 Finite Element Prediction for Pile Behaviours Due to Tunnel Excavation

The prediction results for volume loss = 1% as shown in Figures 3 to 5 will be firstly discussed. Figure 3 presents the predicted pile settlements for Piles 1, 2 and 3 due to EB and WB tunnel excavations. For the first tunnel (EB) excavation, differences in settlements for these 3 piles are merely due to their relative location from EB tunnel. Pile 1 recorded the maximum settlement during EB tunnel excavation as it is located just above the tunnel hence experienced the direct effect from the excavation work. Nonetheless, this difference is minimized after the second tunnel has passed through. A maximum settlement of 20.9 mm was observed in Pile 2; and Pile 1 experienced an approximate similar maximum settlement.

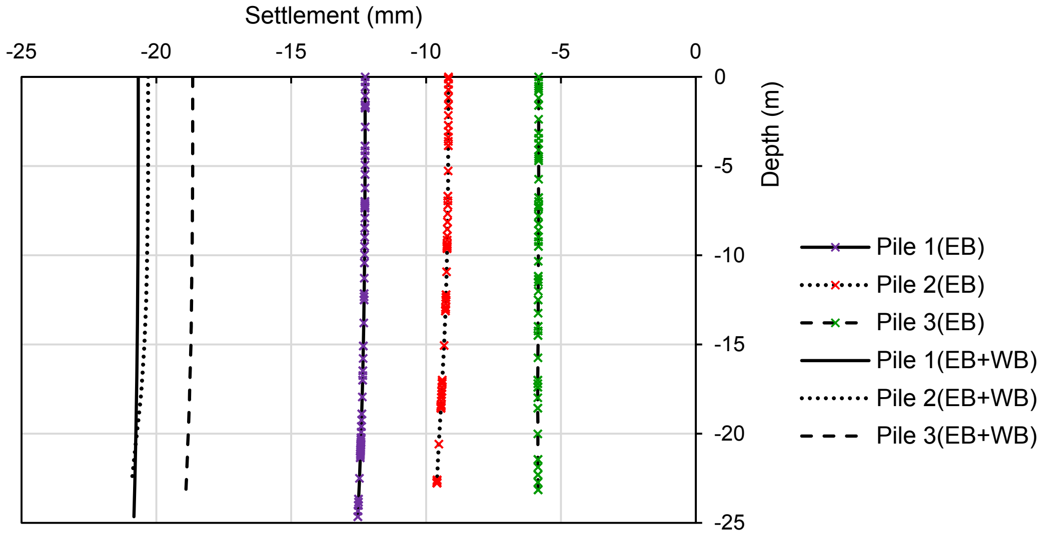


Figure 3. Pile settlements due to twin tunnelling

For the transverse displacement of piles, it is found that the piles are generally displaced towards the direction of the tunnel passing through. Pile 3 first moved towards EB tunnel when it passed through and subsequent WB tunnel construction imposed little transverse displacement as the tunnel was directly beneath Pile 3. On the other hand, Pile E behaved the same pattern but in a mirrored manner. For Pile 2 located at the center of two tunnels, the pile resolved at zero transverse displacement after two tunnels passed through. Figure 4 shows the predicted transverse displacements of Piles 1, 2 and 3 due to twin tunnelling.

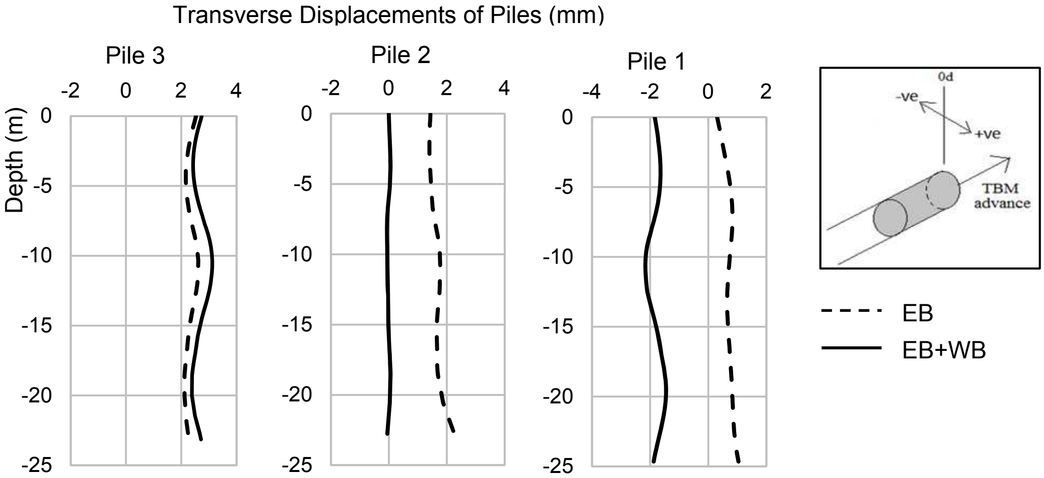


Figure 4. Transverse displacements of Piles 1, 2 and 3 due to twin tunnelling

The ground loss surrounding tunnel results in a loss of pile bearing resistance as the piles are located above the tunnel excavation. This is clearly demonstrated in Figure 5 where the pile bearing resistances after twin tunnelling are significantly reduced compared to initial condition. Upon loading on the piles, the piles underwent larger settlements to further mobilize the shaft friction to hold the applied loads. Figure 5 also shows the maximum induced transverse bending moment for 1% volume loss. The induced transverse bending moment is closely related to the transverse lateral movement as discussed earlier. All the piles register accumulate transverse bending moment after twin tunnelling except Pile 2 which records maximum transverse bending moment during EB tunnel excavation. Pile 1 experienced larger bending moment as compared to Pile 3 although they both posed almost similar transverse lateral movements. This is because the stiffness of Pile 1 (diameter = 900 mm) is higher than Pile 3 (diameter = 800 mm). In view that Pile 1 experienced greater impact due to adjacent tunnel excavation, the pile responses for 1%, 1.5% and 2% volume losses are summarized in Table 2 for structural integrity assessment.

Table 2: Summary of Pile 1 Responses for Volume Loss = 1%, 1.5% & 2%

Volume Loss	1%		1.5%		2 %	
Tunnel	EB	EB+WB	EB	EB+WB	EB	EB+WB
Settlement (mm)	12.55	20.84	19.56	31.52	26.89	42.52
Transverse Displacement (mm)	1.17	2.16	1.68	3.52	2.14	5.09
Max. BM due to tunnelling (kNm)	138		177		221	

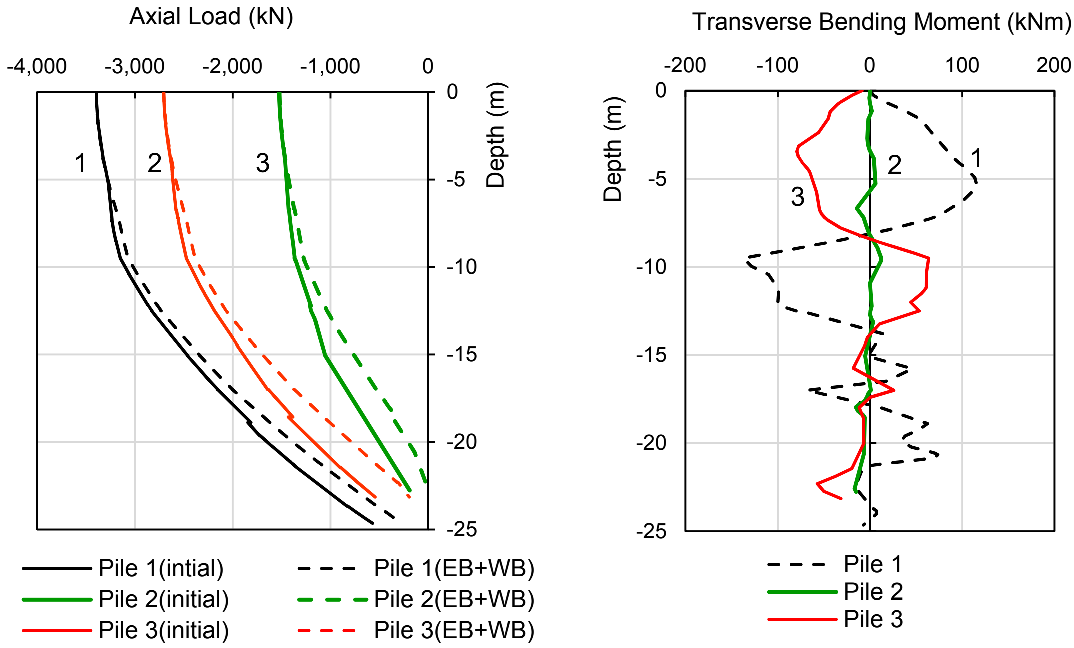
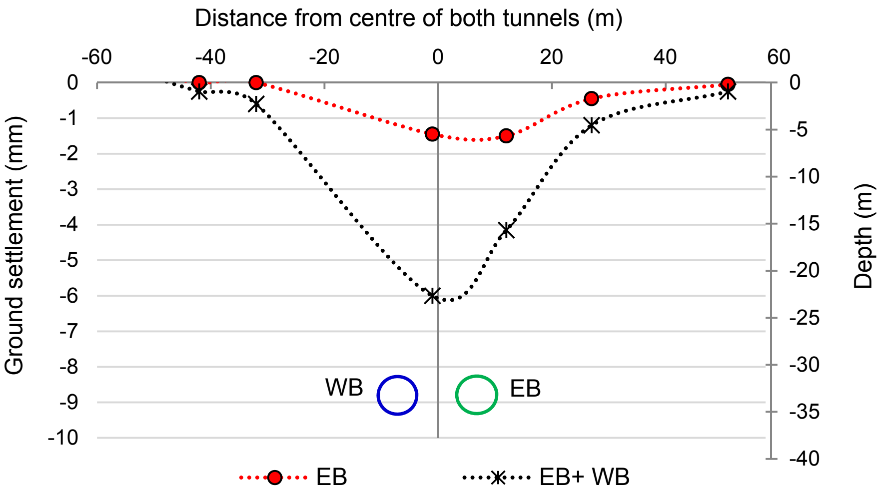


Figure 5. Pile responses due to twin tunnelling

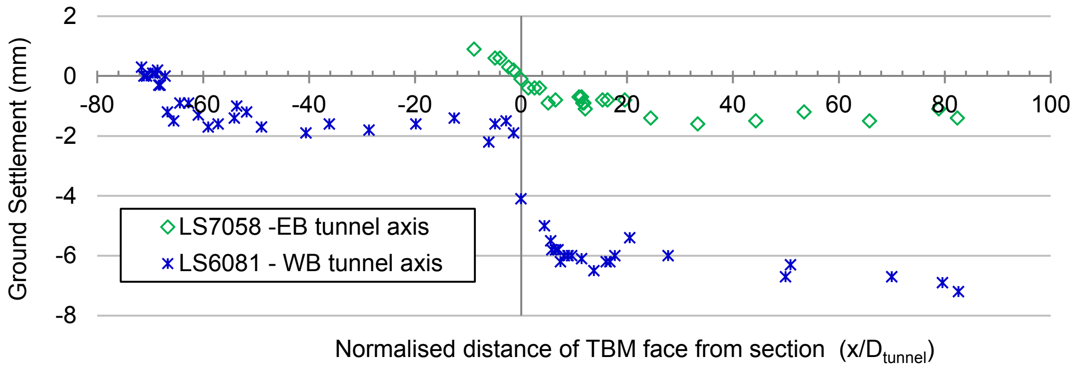
3 FIELD MONITORING RESULTS

Based on field record, the typical applied tunnel boring machine (TBM) face pressure for stability purpose at greenfield area was in a range of 1.5 to 2.5 bars. However, the face pressure was increased to a range of 2.6 to 3.6 bars (which is closer to existing earth pressure) in this section in order to minimize the tunnelling effect on existing buildings. Due to the well-controlled operation on field, the ground volume loss due to tunnelling work was less than 0.25 % in this section. Figure 6a shows the field measured tunnelling-induced transverse settlement trough at this section. The settlement troughs did not fit Gaussian curve shape due to the existing pile-soil interaction. It is observed that the induced ground settlement due to 2nd tunnel excavation (WB) was comparatively much more as compared to 1st tunnel excavation (EB). This observation is consistent with study reported by Pang (2006). This observation is better revealed in longitudinal settlement plot as shown in Figure 6b. One possible explanation to this scenario is that the soil above the EB tunnel has been strained by earlier tunnel excavation; larger movements would be induced with this reduced soil stiffness (Chapman et al., 2004).

The settlement recorded by building settlement markers was in a range of 0.5 mm to 1.2 mm. As expected, the building will experience much lesser settlement than ground since it is supported by piled foundation. Figure 7 plots the building settlement marker readings from other sections in this site against distance from tunnel centre. This is to compare against the study conducted by Cham and Goh (2013) where they found that for volume losses less than 1%, most data fall within a region bounded by a maximum of 0.05% normalised building settlement and 1.5 normalised distance from tunnel centre (as denoted by the dotted line across Figure 8). An observation made from Figure 8 is the consistency of this field data with the prediction made by Cham and Goh (2013).



a) Transverse ground settlement trough



b) Longitudinal settlement plot against normalised distance from interest section

Figure 6. Field measurements of ground surface settlement

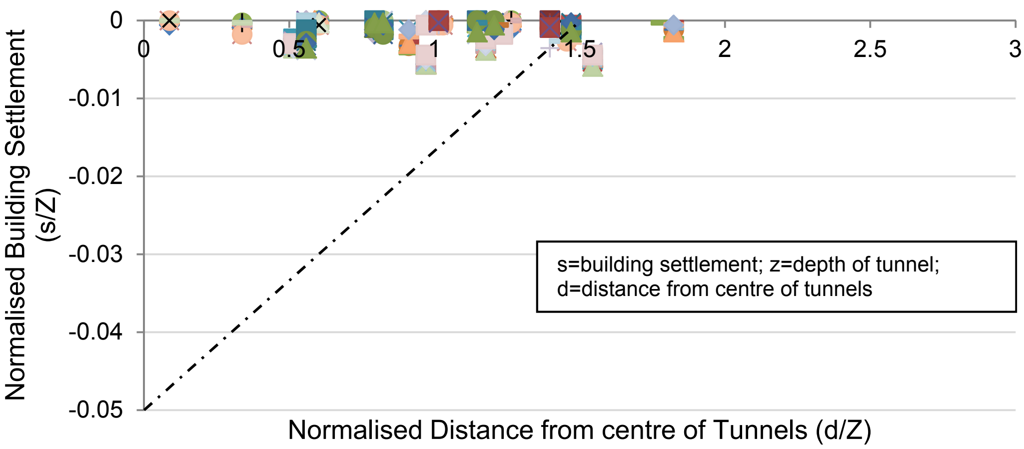


Figure 7. Building settlement against distance from tunnel centre

A total of 14 tilt-plates was installed at the columns of this building to further monitor the building response on top of the building settlement markers. The readings were monitored for angular distortion limit to 1/500 (tilt = 0.11459 degree) as this is the safe limit for building where cracking is not permissible according to damage criteria by Bjerrum (1963). Table 3 summarizes the maximum readings for 14 tilt-plates. From Table 3, this building did not tilt beyond the allowable degree. Based on the buildings settlement markers and tilt-plate readings, it can be said that the effect of tunnelling on this building is negligible. When back-analysed this section based on the measured volume loss and face pressure, the authors found that the effect of these two tunnel excavations on the piled foundation is negligible. As such, this study illustrates the importance of ground volume loss control for tunnel excavation to minimise its damage to adjacent foundations.

Table 3: Summary Maximum Readings for 14 Tilt-Plates (TM)

Tilt-plate	TM-1	TM-2	TM-3	TM-4	TM-5	TM-6	TM-7
Max. degree	0.0046	0.008	0.0011	0.0057	0.0069	0.0034	0.0172
Tilt-plate	TM-8	TM-9	TM-10	TM-11	TM-12	TM-13	TM-14
Max. degree	0.0126	0.0103	0.008	0.0149	0.0092	0.0069	0.0103

4 CONCLUSION

This paper presents the preliminary finite element analysis and field monitoring results for twin tunnelling beneath an existing piled foundation in the eastern part of Singapore. The preliminary FEA was conducted to evaluate the response of piled foundation subjected to the tunnel excavation at volume losses = 1%, 1.5% and 2%. The FE prediction revealed the potential loss of pile bearing resistance as the piled are located above the tunnel excavation. As a result, the piles settled to further mobilize the shaft resistance in order to resist the applied load on top. Three piles were selected for discussion in this study due to their distances away from tunnel. Generally, the piles displaced towards the direction where the tunnel passed through. Centre pile (Pile 2) resolved at almost zero transverse displacement for it was located in the centre of the EB and WB tunnels. Nonetheless, due to the well-controlled operation during tunnel excavations, the measured field ground volume loss was less than 0.25%. The building settlement markers and tilt-plates reported negligible readings which are well within the desired limits. This demonstrates the importance of ground volume loss control for tunnel excavation on site.

5 ACKNOWLEDGEMENTS

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REFERENCES

Bjerrum, L. (1963). Contribution to the discussion. Sect. 6, Proc. of 6th E.C.S.M.F.E., Wiesbaden, 2, 135-137.

Burland, J. B. (1997). “Assessment of risk of damage to buildings due to tunnelling and excavation.” Earthquake Geotechnical Engineering, 3, 1189-1201.

Cham, W. M. (2009). “The response of ground and piled structures to tunnelling.” Proc. of Underground Singapore, Singapore, 60-72.

Cham, W. M. and Goh, K. H. (2013). “Estimation of pile settlement and influence zone due to tunnelling.” Int. Symp. On Advances in Foundation Engineering, Singapore, 507-513.

Chapman, D. N., Rogers, C. D. F. and Hunt, D. V. L. (2004). “Predicting the settlements above twin tunnels constructed in soft ground.” Tunnelling and Underground Space Technology, 19 (4/5), 378-380.

Knight H. C. K., Rosser, H. B. and Eng, W. C. (2001). “Difficult ground conditions encountered during construction of a cross passage in Old Alluvium.” Proc. of Underground Singapore 2001, Singapore, 24-31.

Lee, K. M. and Rowe, R. K. (1991). “An analysis of three-dimensional ground movements: the Thunder Bay tunnel.” Canadian Geotechnical Journal, 28 (1), 25-41.

Pang, C. H. (2006). “The effects of tunnel construction on nearby pile foundation.” PhD thesis, National University of Singapore.

Shirlaw, J. N., Ong, J. C. W., Rosser, H. B., Tan, C. G., Osborne, N. H. and Heslop, P. E. (2003). “Local settlements and sinkholes due to EPB tunnelling.” Proceedings of the ICE-Geotechnical Engineering, 156 (4), 193-211.

Zhang, Y. H., Cham, W. M. and Kumarasamy, J. (2011). “Volume loss caused by tunnelling of circle line projects.” Proc. of Underground Singapore 2011, Singapore, 114-121.